Report of Geotechnical Exploration

Dry Fly Ash Stack
Bottom Ash Disposal Area 2
Ash Disposal Area J
John Sevier Fossil Plant
Rogersville, Tennessee

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

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

February 8, 2010

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

Stantec

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

Re: Report of Geotechnical Exploration
Dry Fly Ash Stack
Bottom Ash Disposal Area 2
Ash Disposal Area J
John Sevier Fossil Plant
Rogersville, Tennessee

Dear Mr. Snider:

Stantec Consulting Services Inc. (Stantec) has completed a geotechnical exploration of the Dry Fly Ash Stack, Bottom Ash Pond Area 2, and Ash Disposal Area J at the John Sevier Fossil (JSF) Plant. The purpose of the exploration was to perform a general engineering assessment of the stability of the three JSF ash disposal facilities. Our final report, transmitted herewith, includes discussions of general site conditions, scope of work performed, subsurface conditions, results of laboratory testing and our engineering analyses. The report also includes a review of historical documentation provided by TVA, and our conclusions and recommendations relative to the conditions and monitoring of the facilities. These services were performed under Engineering Service Request ESR/TAO 700 in accordance with the terms and provisions established in our System-Wide Services Agreement dated December 22, 2008.

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

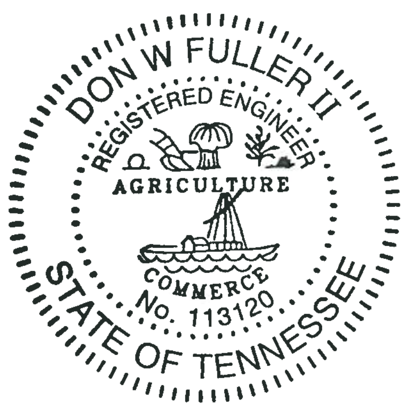
Sincerely,

STANTEC CONSULTING SERVICES INC.

Adam Davis, EIT
Project Engineer

Hugo R. Aparicio, PE
Principal

Don W. Fuller II, PE
Principal



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Executive Summary

Stantec has completed a geotechnical exploration of the Dry Fly Ash Stack, Bottom Ash Disposal Area 2, and Ash Disposal Area J at John Sevier Fossil Plant. The scope of work consisted of reviewing pertinent historical documentation provided by TVA, field observations, a geotechnical exploration, engineering analyses and providing conclusions and recommendations relative to the general stability conditions and monitoring of the three ash disposal facilities.

The Dry Fly Ash Stack is approximately 90 acres in area, rises about 110 feet above a nearby river and receives 215,000 tons of dry fly ash annually. The Bottom Ash Disposal Area 2, which receives 20,000 dry tons of sluiced bottom ash annually, is a 40-acre facility enclosed by an 8,600 foot long dike. The dike is the highest along its north side where it measures about 37 feet. Opened in 1955, the dry stack area was originally a series of ash ponds that stored sluiced ash. In 1979 all sluicing to the stack was stopped and the Bottom Ash Disposal Area 2 went online. The original ponds were closed and the stack area received only compacted, dry ash. Ash Disposal Area J, located west of the dry stack area, was the last ash pond to be constructed and operated from 1982 until 1999. It extends over 22 acres enclosed by an earthen dike that is 35 feet high along its north side.

There are reasonably complete design and as-built drawings of the dikes that form the two smaller facilities and the starter dike built originally along the north and east sides of the Dry Fly Ash Stack. However, practically no as-built information is available relative to the vertical expansion of the starter dike of the Dry Fly Ash Stack, which is the only facility where the starter dike was raised. This information is important because wet ash was deposited above the starter dike and dry ash was later stacked on top of the sluiced ash. Design plans for the dry ash stacking are available. Historical documents note a number of cases where disturbance occurred along the lower north dike slope of the Dry Fly Ash Stack before 2008.

The geotechnical exploration consisted of advancing 93 borings at the project site. The subsurface investigation included standard penetration testing (SPT) in most of the borings, and vane shear testing, cone penetration tests (CPT) and undisturbed soil sampling in selected borings. A total of 45 piezometers and 15 slope inclinometers were installed in selected borings. Several of the borings were advanced along the lower west side of the Dry Fly Ash Stack in an effort to obtain more information relative to the upward expansion of the starter dike after finding sluiced ash above the design top elevation of the starter dike.

The stability of the various dikes was evaluated using two-dimensional limit equilibrium methods of analysis, assuming static, long-term and fully drained conditions within the existing dikes. Stability analyses were performed for several cross sections using soil properties selected based on in-situ as well as laboratory testing results and phreatic levels obtained from piezometer readings. This evaluation was limited to existing conditions and does not address future operations.

The slope stability calculations produced factors of safety against sliding predominantly at or above 1.5, the minimum acceptable value that current USACE criteria requires for long-term loading conditions on similar dikes. Less than acceptable factor of safety values were obtained near the toe of the Dry Fly Ash Stack and Ash Disposal Area J north dike slopes. The low factors of safety for the dry stack are a result of high phreatic levels and steep river bank slope conditions. Steep toe slope conditions resulted in low factors of safety along

certain areas of Ash Disposal Area J. In addition, scouring along the river bank has left near vertical surfaces near the toe of the north dike slope of Area J, which in the past has caused slumps of tree areas that separated the toe of the dike from the river bank before the slumping. The slumps occurred toward the west end of the north dike slope. Similar slumps can potentially occur along the rest of the north dike slope of Area J unless corrective measures are implemented.

There are work plans currently being prepared to install a sub-drain along the toe of the Dry Fly Ash Stack north dike slope to lower the high phreatic surface. The sub-drain and placing additional riprap along the river bank should provide acceptable factors of safety for long term loading conditions. It is recommended that sufficient riprap be placed along the scoured river shoreline below Ash Disposal Area J to prevent potential future slumps adjacent to the toe of the dike as well as improve the stability of the dike.

The profile of the cross sections used in the stability analyses of the Dry Fly Ash Stack slopes was prepared based on the limited information exploratory borings provide and assumptions made relative to subsoil horizon boundaries. Understanding how these profiles were prepared is important in formulating measures to monitor the long term stability of the dike slopes located below elevation 1110 feet. It is recommended that the stability of these slopes be evaluated periodically through a rigorous instrumentation monitoring program. Depending on the results of the periodic evaluations and further analyses of corrective measures toward closure of the facilities, it is possible and it should be expected that additional geotechnical work, including installing more instrumentation, will need to be performed.

Report of Geotechnical Exploration

Dry Fly Ash Stack
Bottom Ash Disposal Area 2
Ash Disposal Area J
John Sevier Fossil Plant
Rogersville, Tennessee

1. Introduction

1.1. General

Tennessee Valley Authority (TVA) retained Stantec Consulting Services Inc. (Stantec) to perform facility assessments at eleven (11) active fossil plants and one closed fossil plant near the Watts Bar Nuclear Power plant. Specifically, Stantec was requested to assess the coal combustion by-product (CCB) disposal facilities at these plants. In general the facilities consisted of ash ponds, scrubber sludge (gypsum) ponds, wet ash dredge cells, dry ash stacks and gypsum stacks. A number of facilities were abandoned (having completed their design life), while majority of them were actively receiving by-products at the time of this project.

1.2. Facilities Assessment Project

Stantec's scope of work for the facilities assessment project was divided into four (4) main phases designated as Phases 1 through 4. Phase 1 was sub-divided into two phases, 1A and 1B. A brief description of Stantec's scope of work for each of the phases is presented in the following paragraphs.

- Phase 1A – Review most recent TVA inspection reports, observe critical disposal features accompanied by TVA personnel, develop a list of primary concerns and recommend immediate action or engineering assessment as considered necessary.

- Phase 1B – Review available historical documentation, visit sites for more detailed observations and measurements, complete dam safety checklists adapted from standard dam safety protocols, recommend immediate action as judged necessary and recommend sites/features that should undergo further evaluation.

- Phase 2 – Evaluate TVA facilities based on current dam safety criteria adopted by the state where the plant is located, conduct geotechnical explorations and engineering analyses at sites recommended in Phase 1 as well as prepare conceptual designs to address identified issues.

- Phase 3 – Design of repairs for sites recommended in Phase 2, plans and specifications for construction as well as permit/planning documents.

- Phase 4 – Dam safety training for TVA Staff and preparation of operation manuals.

At the time of this writing, Phase 1 of the assessment was completed at all fossil plants and Phase 2 was being implemented at several facilities located within the different plants. Phase 1 report recommended that Phase 2 evaluations include geotechnical exploration and hydraulic/hydrologic assessment. This report addresses the results of a geotechnical exploration of the Dry Fly Ash stack, Bottom Ash Disposal Area 2 and Ash Disposal Area J of the John Sevier Fossil Plant.

1.3. Facility Layout and Power Generation

The John Sevier Fossil Plant is located in eastern Tennessee along the southern flank of the Holston River near Rogersville. Figure 1 below shows the approximate location of the plant.

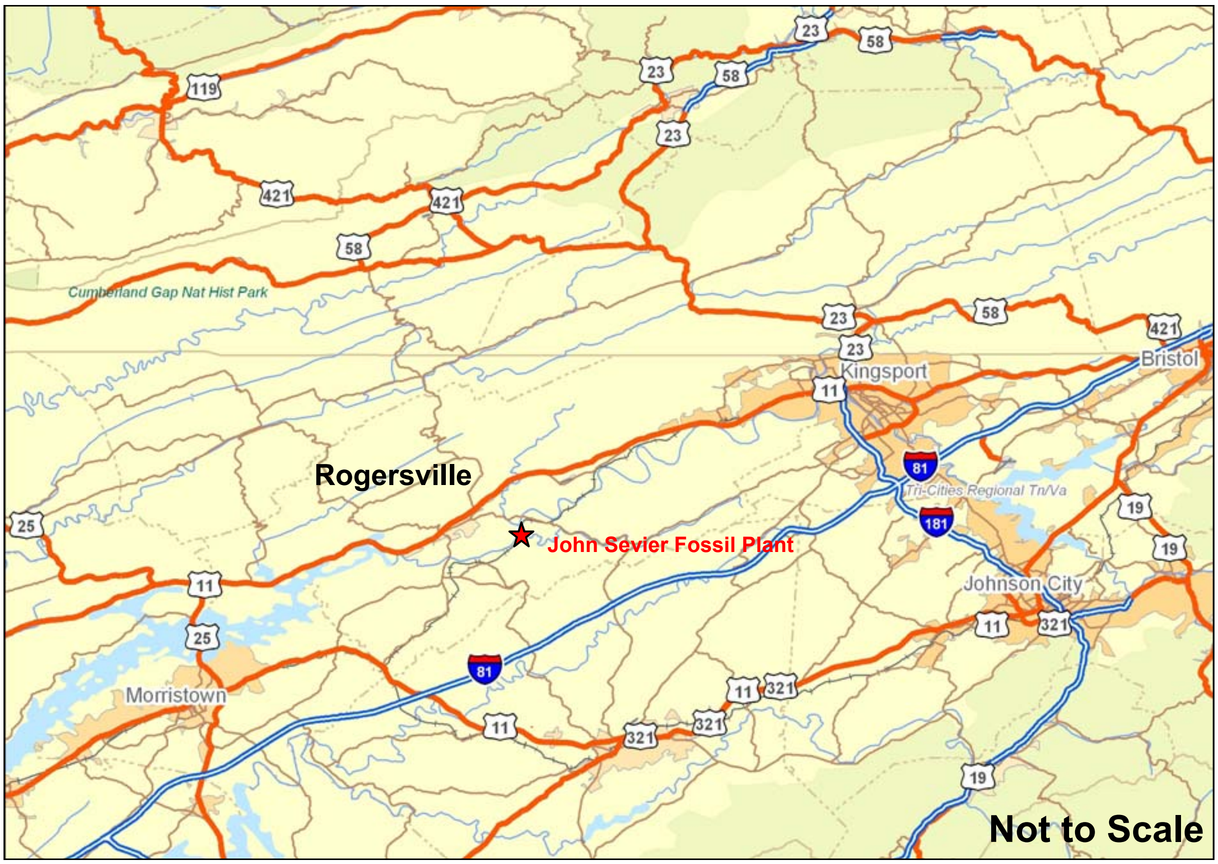


Figure 1. Vicinity Map

Construction of the John Sevier Fossil Plant began in 1952 and was completed in 1957. The plant has four coal-fired generating units, consumes approximately 5,700 tons of coal per day and generates 5 billion kilowatt-hours of electricity a year, enough to supply more than 350,000 homes. The winter net dependable generating capacity is 712 megawatts.

There are three disposal facilities which TVA has operated or is currently operating: (1) Dry Fly Ash Stack, (2) Bottom Ash Disposal Area 2 and (3) Ash Disposal Area J (closed). Figure 2 below shows the layout of the three facilities along with other smaller structures.

Approximately 215,000 tons of dry fly ash is collected in silos each year and hauled to an onsite permitted dry stack disposal area (Dry Fly Ash Stack). Approximately 100,000 dry tons of fly ash is marketed offsite to the concrete industry. Approximately 20,000 dry tons per year of bottom ash is wet-sluiced to Bottom Ash Disposal Area 2. At the Bottom Ash Disposal Area 2, bottom ash is collected and sent offsite by Appalachian Products.

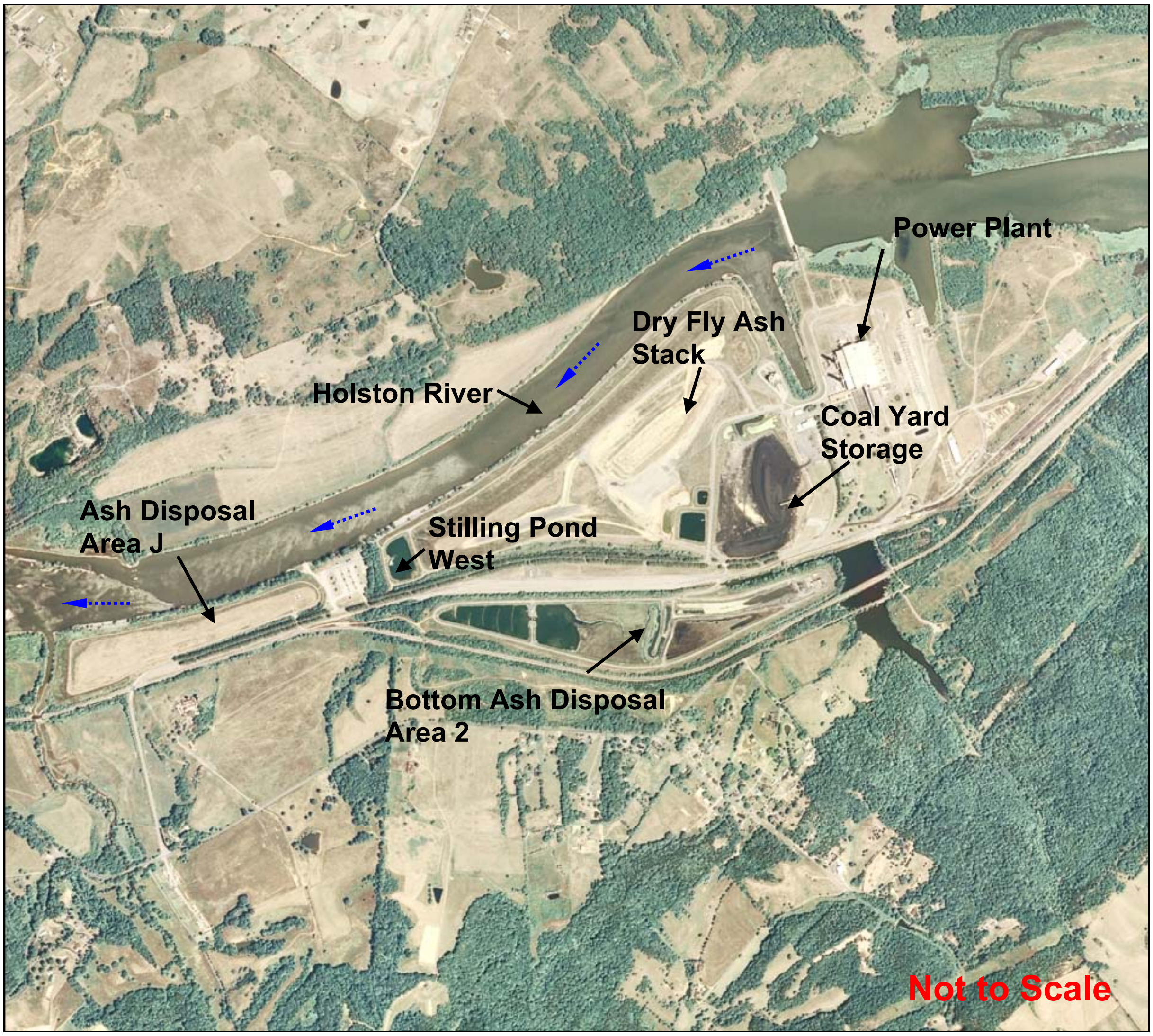


Figure 2. Location Map

2. Scope of Work

The scope of the geotechnical exploration was divided into the following tasks.

a. Review of general site geology

b. Review of historical Information

c. Disturbance features observed in 2009

d. Subsurface Exploration

e. Field Instrumentation and Monitoring

f. Surveying

g. Laboratory Testing

h. Engineering Analyses

i. Repair and Maintenance Work Completed in 2009

j. Conclusions and Recommendations

k. Closure

The work performed as part of these tasks is described in the following paragraphs

3. General Site Geology

The John Sevier Fossil Plant is located in the eastern portion of Tennessee along the southern flank of the Holston River just east (upstream) of the confluence of the river and Dodson Creek. This portion of Tennessee is underlain by sedimentary rock formations which were folded and fractured by ancient tectonic events. More specifically, the general area of the plant is underlain by two distinct formations, the Sevier Shale and the Newala Formation of the Knox Dolomite Group. It is probable that the contact between these formations occurs along or just north of where the Holston River crosses the plant area, with the Sevier Shale outcropping south of the river.

Most of the plant reservation was developed on a floodplain of the Holston River. As such, much of the site is underlain by alluvium and terrace deposits varying in thickness from less than 5 feet along the tributary stream banks to more than 30 feet adjacent to the river. Typical of alluvium in this region of the state, these soils consist of sands, silts, and gravels with few interspersed cobbles. The underlying bedrock consists of the Ordovician age Sevier Shale Formation which consists of bluish gray, a silty to sandy calcareous shale with thin limestone layers and lenses of siltstone and sandstone.

According to a description presented in plant historical information (see Reference 10 listed in Table 1), massive shale outcrops in a quarry located southeast of the plant indicate that the folded Sevier Shale dips at angles ranging from 45 to 80 degrees to the southeast. Joints were observed running sub-parallel to the strike and dipping near vertical. Reference 10 also states that the Newala Formation is exposed north of the river where a significant level of solution activity was noted.

Sevier Shale outcrops are visible along the Polly Branch Creek adjacent to the existing Bottom Ash Disposal Area 2 and along the Holston River adjacent to the closed Ash Disposal Area J. Solution activity within the plant reservation south of Holston River was not reported in previous geotechnical studies nor was it encountered during Stantec's geotechnical exploration.

4. Review of Historical Information

4.1. General

During the Phase 1 of the facility assessment, Stantec engineers reviewed all documents provided by TVA pertaining to the development of the different ash disposal facilities. The documents reviewed for this report include mostly design drawings and reports. Other documents reviewed consisted of correspondence (letters, emails and faxes) and photos. A complete list of the documents provided by TVA for review is presented with the Phase 1 Facility Assessment Report. Table 1 presents a list of the documents considered more relevant to the geotechnical study of the different disposal areas as part of Phase 2 of the facility assessment.

Table 1. List of Documents Reviewed for Geotechnical Exploration

Reference No.*	Document Name	Type of Document	Dated	Agency	TVA Reference No.
1	Ash Disposal Area	Design Drawing	April 1953 (revised 1958)	TVA	10N410
2	Ash Disposal Area "E" Dike Repair	Design Drawing	July 1973 (As-Built, March 1975)	TVA	10N290
3	Fly Ash Disposal Area "G"	Design Drawings	February 1974 (As-built, August 1980)	TVA	10N295 10N296
4	Ash Disposal Area No. 2	Design Drawings	August 1977 (As-Built, August 1980)	TVA	10W293 1 through 3
5	Fly Ash Disposal Area "J"	Design Drawings	July 1982 (revised 1984)	TVA	10W286 1 through 7
6	Dry Fly Ash Stack Existing Contours (East)	Design Drawing	September 1994 (revised 1997)	LAW	10H291-3
7	Ash Disposal-Stack Area	Design Drawings	March 2001 (revised 2002)	Parsons	10W206 1 through 11
8	Ash Disposal Area Soils Exploration & Testing	Report	June 1981	TVA	NA
9	Ash Disposal Area Proposed Dry Stacking	Report	July 1986	TVA	NA
10	Report of Hydrogeologic and Engineering Evaluation	Report	October 1994	LAW	NA
11	Dike Exploration and Testing Program	Report	October 1999	LAW	NA
12	Fly Ash Pond Dike Slope Stability Evaluation	Report	December 1999	Parsons	NA

*Presented as attachments in this order in Appendix A

4.2. Development of Disposal Facilities

4.2.1. Dry Fly Ash Stack

The Dry Fly Ash Stack was originally a series of ash ponds when the plant went online in 1955. The ponds were labeled as 'Areas' and lettered from A to G, with Area A being the most eastern pond and Area G being the most western (west half of Area G is now the Stilling Pond West). There is practically no information available relative to the construction of the dikes that divided these areas, except for the construction of Area G as discussed later. Reference Nos. 2 and 3 include a Key Plan of the disposal site showing the relative location of the different areas. This Key Plan is also presented in Figure 3.

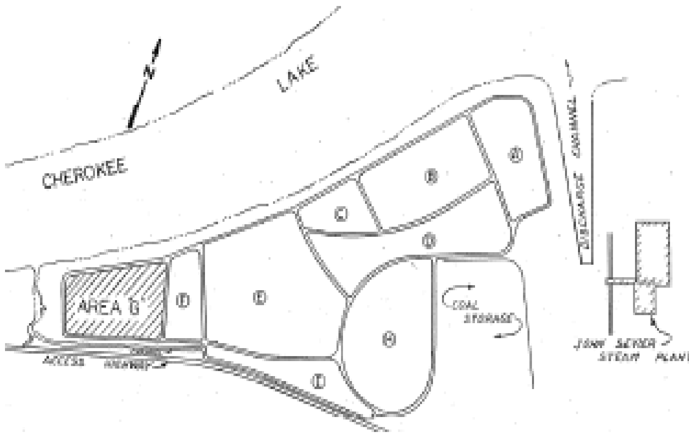


Figure 3.   Original Disposal Pond Areas

At the beginning of the plant operations, only Areas A, B and C were active and water was discharged to the river through a spillway in Area C. In 1971, Areas A, B, and C were abandoned and ash was sluiced to Areas D, E and F (spillway in Area F discharged to river).

In 1973, sluicing stopped to Areas D, E, and F due to the dike failure in Area E (though spillway was still active) and Areas H and I were activated (spillway in I to drainage channel along main plant road). In 1974, Areas A, B, C, D, E, and F were used as disposal areas for dredged bottom ash. In 1976, Area G was activated in the west end of the current Dry Fly Ash Stack, and received all sluiced fly ash while Areas H and I received all sluiced bottom ash.

In 1979, the Bottom Ash Disposal Area 2 was activated and all sluicing stopped to the Dry Fly Ash Stack area. At this same time, Areas A through I were designated for dry ash disposal and Area G was filled and abandoned. In 1982, a Bathtub Area was constructed in the eastern portion of the Dry Fly Ash Stack. In 1984, the Bathtub Area began receiving dredged bottom ash from the Bottom Ash Disposal Area 2. In 1990, all bottom ash was sluiced to the Bathtub Area as Bottom Ash Disposal Area 2 was offline. In 1993, dry fly ash began being stacked in the Bathtub Area, which extended approximately over Areas A through E and H. A plan view drawing of the Dry Fly Ash Stack site showing the approximate location of Areas A through I and the Bath Area is presented in Appendix B**.**

In 2001-2002, the eastern two thirds of the north slope of the Dry Fly Ash Stack, below approximate elevation 1100 feet, were re-graded after surface sliding and tension cracks developed in this area of the slope. A sub-drainage collection system (with two pumps) was constructed in the vicinity of two old clay pipes in the northeast corner in 2000 and expanded as part of the re-grading in 2001-2002. This system is shown on the plan view drawing presented in Appendix B.

4.2.2. Ash Disposal Area J

Ash Disposal Area J went online in late 1982 and was used as a fly ash settlement pond. Ash was sluiced to the east end of the area. The west side of the disposal area acted as a stilling pond and contained two concrete riser structures which discharged into the Holston River. In 1985, riprap was placed along 700 feet of the river bank to protect the toe of the dike on the west end of the north dike slope, after a treed area next to the toe slumped into the river. At the same time the exterior slope of the west side of the dike was changed from 2:1 to 4:1. Sluicing was stopped in 1988 and the pond was dewatered and used as a dry stacking area. Ash Disposal Area J was inactive starting in early 1990's and officially closed in 1999.

4.2.3. Bottom Ash Disposal Area 2

The Bottom Ash Disposal Area 2 came online in 1979 to receive all sluiced bottom ash and infrequent sluiced fly ash. A stilling pond exists in the west end of the area, accessed through a rock weir in an internal dike. Bottom ash was stacked in the southeastern portion of the area starting in 1981. In 1987, sluicing stopped at Area 2 and the ash was dry hauled offsite for disposal. Ash was again sluiced to this area starting sometime between 1990 and 1993. In 1999, a bottom ash collection facility was constructed at the east end of Area 2 and run by Appalachian Products, for offsite marketing of bottom ash. Currently, the Bottom Ash Disposal Area 2 receives sluiced bottom ash, intermittent fly ash (sluiced to separate trench for settlement), and discharges from the Coal Yard Runoff Pond and Chemical Treatment Pond - Iron.

4.3. Design and Record Drawings

4.3.1. Reference No. 1 – Dry Ash Disposal Area Starter Dike

Reference No. 1 (listed in Table 1) is a design drawing titled "*Ash Disposal Area",* originally dated April, 1953 and revised for the third time in April, 1958. This drawing was prepared by TVA and is declared the "Final Field Revision". The drawing is believed to have been used for constructing the starter dike along the northern and eastern edges of the site to form the main barrier of the initial ponds, which is now the location of the existing Dry Fly Ash Stack. This drawing also appears to illustrate the original ground contours prior to any development of the ash disposal facility, as well as the "Begin Dike" and "End of Dike" locations. Based on the "End of Dike" location, it appears the original intent was to end the starter dike short of Area G.

The single page drawing shows several design cross sections of the starter dike. According to these sections, the starter dike was constructed with 3:1 slopes on the river side and 1.5:1 slopes on the ash fill side. The top of the dike was constructed at an elevation of 1087 feet and having varying crest widths. A typical section from reference drawing No. 1 is shown below in Figure 4.

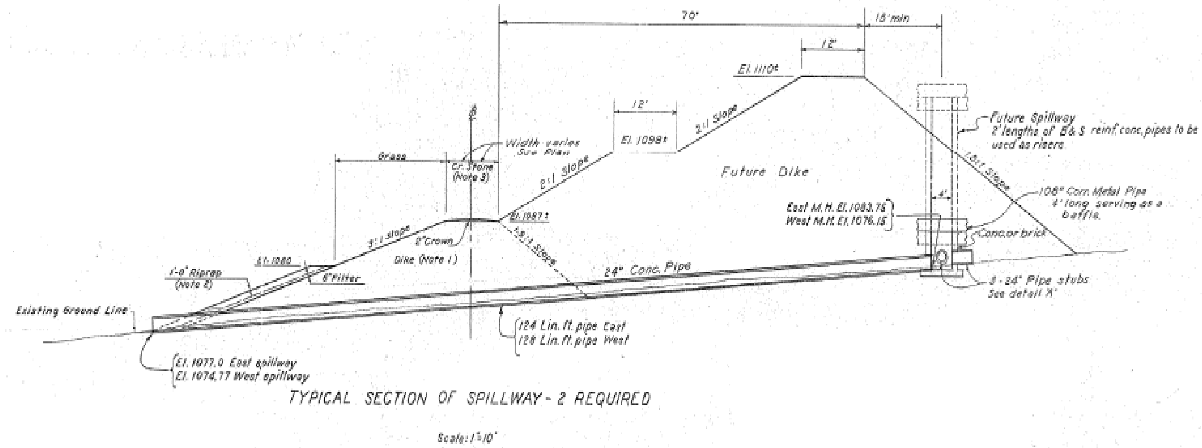


Figure 4. Starter Dike Typical Section 1953 (Revised 1958)

The section in Figure 4 also shows a proposed (future) vertical expansion of the dike which would have raised the top of the starter dike from elevation 1087 feet to elevation 1110 feet±. The expansion was to include 2:1 exterior slopes and a 12-foot wide intermediate bench at elevation 1098 feet±.

4.3.2. Reference No. 2 – Pond "E" Dike Repair (1973)

Reference No. 2 (listed in Table 1) is a drawing titled "Ash Disposal Area "E" - Dike Repair," originally dated July, 1973 and signed," Record Drawing As Constructed" in March, 1975. This drawing, shown in Figure 5, illustrates conditions prior to the May, 1973 dike failure. The break in the northern dike occurred near the divider dike between Areas E and F and was approximately 300 feet long.

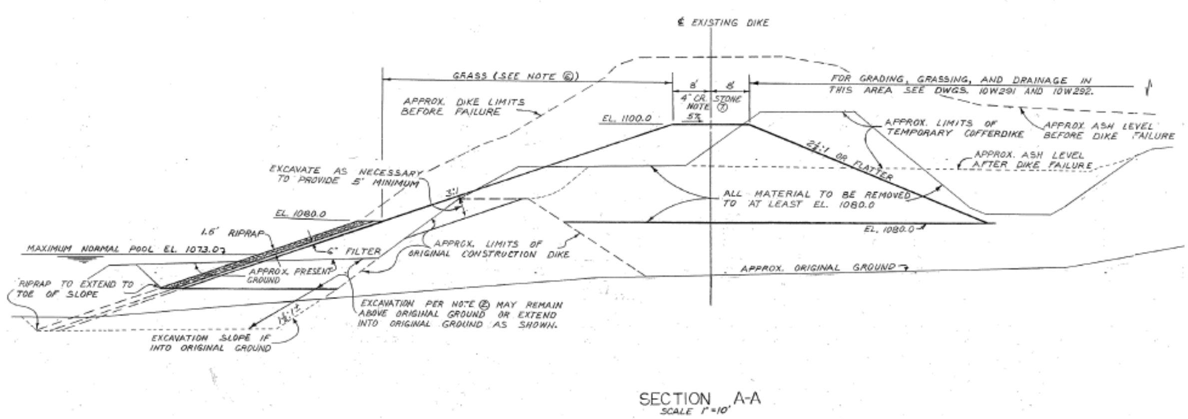


Figure 5. Ash Disposal Area "E" – Dike Repair 1973 (As Built 1975)

The drawing depicts several features not shown in the 1958 typical section (Figure 4). It appears that at least in Area E the original dike was not expanded following the original intended design. The following items provide some insight to the conditions of this area in 1973 and repair work proposed at that time.

- Approximate ash level before the dike failure, with the top of the ash located near elevation 1130 feet+, indicates ash was placed on top of the starter dike and extending into the river bank, with no intermediate benches.

- Ash level after the dike failure at approximately elevation 1098 feet, or a drop of about 40 feet from the top elevation prior to failure.

- Removal of all material to at least elevation 1080 feet as part of the repair work.

- Construction of a temporary coffer-dike and upward expansion of the starter dike by reestablishing the exterior 3:1 slope of the starter dike straight up to elevation 1100 feet.

- Lining the riverbank with an 18-inch thick layer of riprap.

4.3.3. Reference No. 3 – Fly Ash Disposal Area G

According to Reference No. 3, Area G was the last of the contiguous areas developed for sluiced ash disposal purposes. The as-built drawings include notes indicating that changes to Area G were implemented as recently as August, 1980. Figure 6 shows a cross section of Area G dike extracted from Reference No. 3.

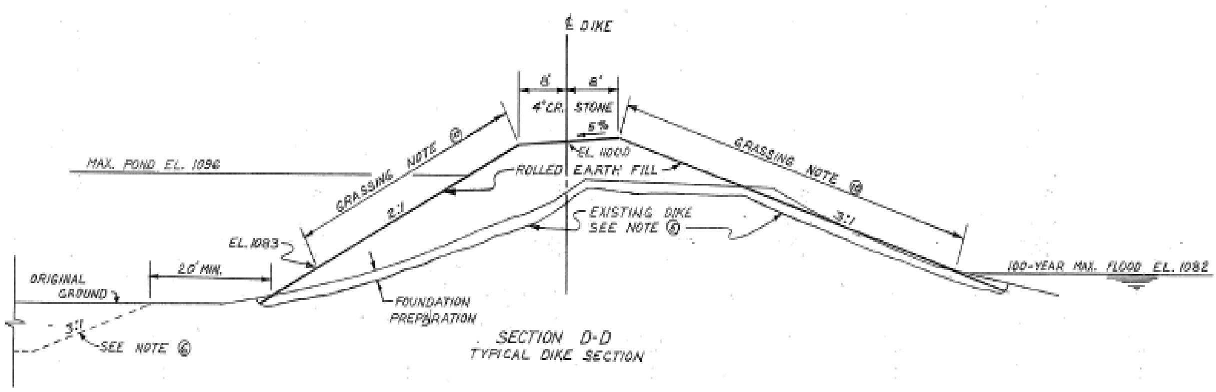


Figure 6. Dry Fly Ash Stack- Typical West Section (As Built 1975)

This cross section appears to indicate that an initial or starter dike had already been extended into Area G and constructed up to near elevation 1090 feet prior to the final development of Area G. The section also shows the Area G dike crest set at elevation 1100 feet.

According to Reference No. 10 (dated October, 1994), the plant disposed ash in a stacking procedure over the western portion first. Consequently, the western portion of the site had risen to approximately 20 feet above the level of the impoundment dikes. These drawings along with historic inspection reports were used to develop the original pond limits for plan drawings presented in Appendix B.

4.3.4. Reference No. 4 – Bottom Ash Disposal Area 2 Dike

Reference No. 4 (listed in Table 1) is a set of design drawings titled "*Ash Disposal Area No. 2",* originally dated August, 1977 and signed as a "Record Drawing as Constructed," August, 1980. These drawings are believed to have been used for constructing the dike along the entire border of what is now the location of the existing Bottom Ash Disposal Area 2. These drawings are believed to illustrate the original ground contours prior to any development of the disposal facility as well as design of the dikes, spillway, and drainage ditches. According to the available tables and sections illustrated on the drawing, the dikes were constructed with slopes varying between 2:1 and 3:1. The top of the dike was constructed at an elevation of 1145 feet and having a uniform width throughout of sixteen feet. A typical dike section from Reference No. 4 is shown below in Figure 7. According to this section, impervious earth fill was placed in a cutoff trench and toe area of interior dike slope to control seepage through the foundation soil.

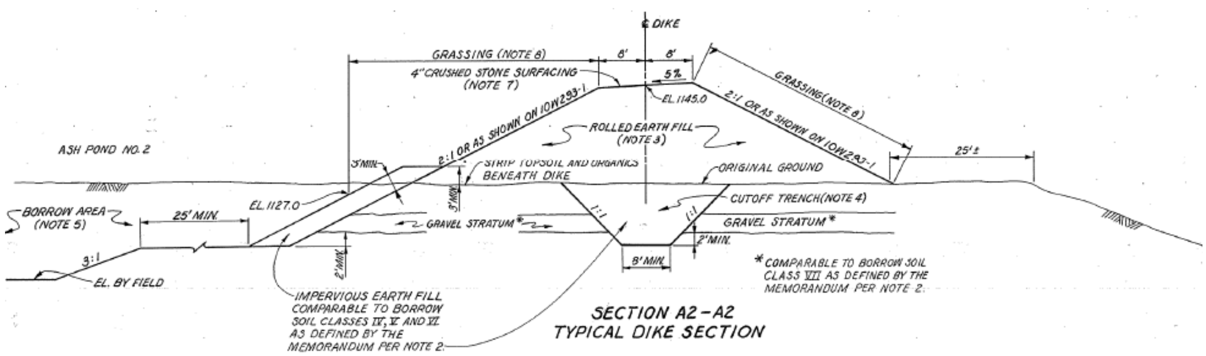


Figure 7. Bottom Ash Disposal Area 2 – Typical Section 1977 (As Built 1980)

4.3.5. Reference No. 5 – Fly Ash Disposal Area "J"

Reference No. 5 (listed in Table 1) is a set of design drawings titled "*Fly Ash Disposal Area J,"* originally dated July, 1982 and revised December, 1984. These drawings are believed to have been used for constructing the dike along the entire border of what was originally Ash Disposal Area "J" and reflect some modifications to the original dike configuration.

These drawings appear to illustrate the original ground contours prior to any development of the disposal facility as well as typical cross sections of the dike, spillway, and drainage ditches. According to these drawings, the dike of Ash Disposal Area "J" was constructed with slopes of 2:1 interior slopes and 2.5:1 exterior slopes. The top of the dike was constructed at an elevation of 1105 feet and a uniform bench width of sixteen feet.

Sheet 4 of the drawings illustrates some repair work performed toward the west end of the north dike slope to stabilize the river bank. A relatively narrow tree area located between the toe of the dike and a steep (near vertical) river bank apparently slumped into the river compromising the toe of the dike. Similar pre-slump conditions currently exist east of this area of the north dike slope. A typical dike cross section from a Reference No. 5 drawing is shown in Figure 8. This section also shows the measures taken to stabilize the river bank area discussed above.

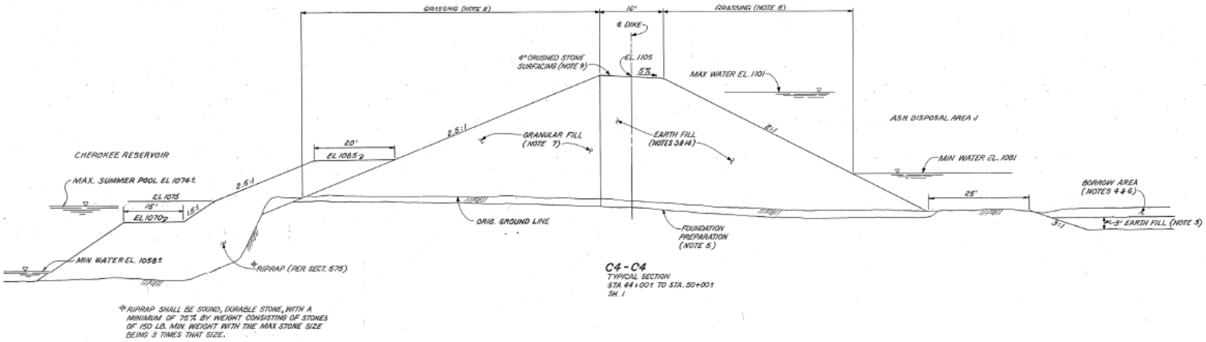


Figure 8. Ash Disposal Area "J" – Typical Section 1984

Sheets 5, 6 and 7 of the drawings in Reference No. 5 also include a closure plan revised in January of 1995.

4.3.6. Reference No. 6 – Dry Fly Ash Stack – Bathtub Area

Reference No. 6 (listed in Table 1) is a drawing created by Law Engineering, Inc. and Tribble & Richardson Inc., titled “Dry Fly Ash Stack Existing Contours,” originally dated September, 1994 and revised March, 1997. This drawing illustrates existing 1992 contours including the Bathtub Area. This drawing was used to determine the limits of the Bathtub Area for the drawing titled, “Original Disposal Facilities” presented in Appendix B.

4.3.7. Reference No. 7 – Ash Disposal - Stack Area

Reference No. 7 (listed in Table 1) is a set of design drawings created by Parsons Energy & Chemical Group Inc. In 1999, Parsons conducted a slope stability analysis on a total of seven cross sections through the northern and eastern dikes. The results of the study concluded that the east two-thirds of the north slope of the disposal area was only marginally stable and needed to be repaired. The west one-third of the slope was deemed to have an adequate factor of safety against sliding, therefore it needed no repairs. As a result, the drawings showed 3:1 re-grade slopes to be applied to the marginally stable areas up to the intermediate bench located near elevation 1110 feet.

This set of drawings illustrates previous existing site features as of February, 2001 as well as design plans for re-grading and additions of riprap near the base of the slopes. These drawings were used by Stantec to develop the profile of the river bank area immediately below the toe of the Dry Fly Ash Stack, in preparation of the slope stability analysis. Specifically, the drawings were used to estimate the thickness or geometric configuration of the riprap layer placed along the base of the dike.

4.4. As-Built Drawings

The title blocks for Reference Nos. 2, 3 and 4 drawings contain the description "Record Drawings As Constructed," and date of original signing. These drawings also include revisions to the original drawings and their corresponding new dates.

The title block of the earliest ash disposal area drawing (Reference No. 1) is dated April 30, 1953. A revision note above the title block for April 23, 1958 reads "Final Field Revision." The John Sevier Fossil Plant came online in 1955 and therefore, it is assumed that this drawing is also considered an as-built drawing.

4.5. Geotechnical Studies

Historical documentation for review included reports of subsurface investigations, hydro-geologic studies, and dike stability evaluations studies and investigations performed for the fly ash disposal area. Documents in Reference Nos. 8, 9, 10, 11 and 12 include information and data used for review purposes.

4.5.1. Reference No. 10 – Hydrogeologic and Engineering Evaluations (Law 1994)

In 1994, Law Engineering, Inc. based out of Atlanta, Georgia performed a hydrogeologic and engineering evaluation at the John Sevier Fossil Plant in general accordance with requirements of the Tennessee Department of Environment and Conservation. The study utilized previous subsurface explorations to augment its own findings from four (4) soil test borings. Supplemented data came from Reference No. 9, "John Sevier Fossil Plant-Ash Disposal Area-Proposed Dry Stacking," an internal report produced by TVA. The data collected from all findings provided Law engineers with information to form technical reviews of groundwater recharge, discharge, and flow as well as soil parameters that were used to perform slope stability analysis. The analysis was completed on two typical cross sections, perpendicular to the river and perimeter dike.

4.5.2. Reference No. 11 – Dike Exploration and Testing Program (Law 1999)

In 1999, Law Engineering, Inc conducted a subsurface investigation which included seven (7) soil borings along the top of the existing dike and six (6) soil borings along the perimeter road near the base of the dike. Laboratory testing was conducted on Standard Penetration Test (SPT) samples and undisturbed samples obtained from recovered Shelby tubes. Testing included natural moisture content determinations, Atterberg limits, grain size distribution, and tri-axial shear tests. The exploration was used to supply general subsurface conditions at John Sevier to a third party for purposes of conducting a slope stability analysis.

4.5.3. Reference No. 12 – Fly Ash Pond Dike Slope Stability Evaluation (Parsons 1999)

In 1999, following the Law Engineering report, Parsons Energy & Chemical Group Inc. conducted a dike slope stability evaluation. The evaluation, using data collected from the 1994 and 1999 Law reports, focused on seven (7) widely spaced cross-sections believed to represent typical geometry and conditions along the northern and eastern dikes. Parsons reported existing factors of safety values varying between 0.87 and 1.61, and recommended re-grading the dike sections with a factor of safety less than 1.3 to a uniform slope of 3H:1V.

4.6. O&M Manual

The only operations and management document supplied by TVA is titled “John Sevier Fossil Plant By-Products Operations Manual.” Within this document is the Pond & Ash Management JSF.TI.05.014.019 which briefly discusses the duties and obligations of TVA personnel at the plant. Management procedures are broken into Yard Ops Duties, Yard Ops Engineering, Plant Ops Duties, PAE Duties, and Fossil Engineering Services. Procedures include routine inspections which are assumed to be visual only. Fossil Engineering Services is required to prepare, once each year, “a Dike Stability Report based on inspections of all pond dikes (ash, yard drainage, red water & fines) for leaks, erosion, rooted trees and red water seeps.”

4.7. Annual and Quarterly Reports

Annual reports reviewed by Stantec include the “JSF-Annual Stability Inspection of Waste Disposal Areas,” conducted by Fossil Engineering Services accompanied by plant personnel. Inspections were conducted for the Fly Ash Disposal Area, Ash Disposal Area 2, Ash Disposal Area J, Chemical Treatment Ponds, and Coal Yard Drainage Basin.

Quarterly Reports reviewed by Stantec include the “Quarterly Red Water Seep Inspection,” conducted by plant personnel. Visual inspections were conducted for the Ash Stack River Dike, Exterior Slopes-Ash Stacking Area, Pond 2 Active Ash Pond Dike, and J-Pond Inactive Ash Pond Dike.

4.8. Summary of Disturbance Events

The documents listed in Table 2 were used to gain an understanding of key disturbance events that occurred at the John Sevier Fossil Plant facilities. These events were used to identify areas of possible concern. The events listed in Table 2 are in chronological order.

Table 2. Summary of Disturbance Events

Date	Event	Document Source
May 1973	North Dike Failure	1973-Annual Ash Disposal Area Inspection
September 1989	North Dike Toe Slide	1990-Annual Fossil and Hydro Engineering Inspection of the Ash Disposal Areas
December 1990	North Dike Slides	1991-Original Ash Disposal Area – Dike Slope Stability
July 1990	North Dike Tension Cracks	1990-Annual Fossil and Hydro Engineering Inspection of the Ash Disposal Areas
February 1991	North and East Dike Sloughing	1991-Original Ash Disposal Area – Dike Slope Stability
February 1994	Dike Sloughing at Toe of Stilling Pond West	1994-Annual Fossil Engineering Report Inspection of Ash Disposal Areas
April 1995	North Dike Shallow Surface Slide	1995-Annual Fossil Engineering Report Inspection of Ash Disposal Areas
March 1997	Minor Surface Sloughing	1997-Annual Inspection of Waste Disposal Areas
April 1999	Northwest Stack Corner Surface Slide (adjacent to riprap down drain)	2000-Annual Ash Pond Dike Inspection
September 2007	North Dike Sloughing	2008-Annual Stability Inspection of Waste Disposal Areas
November 2007	North Dike Erosion Ditch	2008-Annual Stability Inspection of Waste Disposal Areas

*-All event locations listed are approximate based on Stantec's review of available documents

5. Disturbance Features Observed in 2009

Table 3 presents a summary of disturbance features observed during Phase 1 of the facilities assessment completed in January and February of 2009. Items 3, 4 and 6 through 10 have been addressed through repair and maintenance work performed since the Phase 1 of the assessment was completed, as described in Section 12 of this report.

Items 1 and 2 appear to have been present since prior to 1999. According to the historical documents, and based on recommendations presented in Reference No. 12, the lower east two-thirds of the north slope of the Dry Fly Ash Stack were re-graded to stabilize the area extending from the toe of the starter dike up to elevation 1110 feet. However, the same lower area of the slope located west of the ramp that connects the lower and upper perimeter roads was left unchanged. Today this area has an irregular surface with an apparent slump immediately below the crest of the slope and some isolated humps. In addition, TVA personnel inspecting the plant facilities report periodically the presence of wet areas along the toe of the slope and the lower perimeter road.

Table 3. Disturbance Features Noted during Phase 1 of Facilities Assessment

No.	Structure	Location	Type of Disturbance
1	Dry Fly Ash Stack	North dike exterior slope west of northern access ramp	Slumping approx. 400 FT long
2	Dry Fly Ash Stack	North dike exterior slope west of northern access ramp	Raised area approx. 2 FT above neighboring ground
3	Bottom Ash Pond Area 2	West exterior slope of stilling pond	Minor slumps, slides and depressions.
4	Bottom Ash Pond Area 2	Southwest corner exterior slope of stilling pond	33FT x 51FT area of multiple depressions and mounds
5	Ash Disposal Area J	North river embankment	Several areas of erosion
6	Stilling Pond West	West interior slope near outlet structures	Minor slump
7	Stilling Pond West	East interior slope	Small slumps and depressions
8	Sediment Pond East	North interior slope	Four erosion gullies
9	Iron Chemical Treatment Pond	Northeast corner interior slope	Minor sloughing, irregular slopes, and depression
10	Coal Yard Drainage Pond	Southeast interior bank	Bank erosion

6. Subsurface Exploration

6.1. General

Fieldwork for the geotechnical exploration was performed by Stantec during March 23, 2009 through June 5, 2009. The field work consisted of advancing 93 borings at the project site. Boring locations were chosen by Stantec and staked and surveyed by TVA. The subsurface exploration included standard penetration testing (SPT) in selected borings, the installation of 45 piezometers advanced using 3¼ inch (ID) hollow stem augers, 15 slope inclinometers advanced using 4¼ inch (ID) hollow stem augers, 12 vane shear tests, and 5 cone penetration tests (CPT). The locations of the borings and their corresponding elevations are shown on the boring layout drawing provided in Appendix B.

An automatic hammer was utilized to perform SPT testing in the borings advanced as part of this exploration A standard penetration test consists of dropping a 140-pound hammer to drive a split-barrel sampler 18 inches. The consistency or relative density of the soil material is estimated by the number of blows it takes to drive the split spoon the last 12 inches. 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. In addition, undisturbed samples (Shelby Tubes) were also obtained from selected depth intervals within fly ash and foundation clay. Upon completion of the drilling and sampling procedures, the boreholes were either backfilled with auger cuttings or well backfill materials (cement, sand and/or bentonite) depending on the type of instrumentation the borehole received.

A geotechnical engineer or geologist was present on-site throughout the drilling and sampling operations. The engineer/geologist directed the drill crew, logged the subsurface materials encountered during the exploration, and collected soil samples. Particular attention was given to the subsurface material's color, texture, moisture content and consistency or relative density. Following the field exploration, the SPT samples, Shelby tube and bulk samples were transported to our laboratory. The samples will be available for review up to thirty days following the submittal of this report, at which time the samples will be discarded unless prior arrangements for storage have been made.

6.2. Summary of Borings

Typed boring logs are presented in Appendix C. Results of laboratory testing on selected samples are included in Appendix F. The boring layout is presented on a drawing included in Appendix B. A summary of the boring information is presented in Table 4, where all measurements are expressed in feet.

Table 4. Summary of Borings

Boring No.	Surface Elevation (ft)	Northing (ft)	Easting (ft)	Depth to Bottom of Hole (ft)	Elevation of Bottom of Hole (ft)
BA-1	1145.4	734343.87	2893639.94	39.4	1106.0
BA-2	1145.9	734229.93	2893695.53	50.5	1095.4
BA-3	1145.3	733939.03	2893286.73	37.1	1108.2
BA-4	1145.2	733486.11	2890407.91	42.5	1102.7
BA-5	1144.9	733604.48	2889750.33	56.4	1088.5
BA-6	1145.1	733808.75	2889830.63	48.9	1096.2
BA-7	1144.3	733872.97	2890492.40	39.6	1104.7
BA-8	1145.2	733946.71	2891566.83	40.2	1105.0
BA-9	1144.7	734027.41	2892632.01	41.2	1103.5
JP-1	1105.4	733930.64	2888187.78	36.0	1069.4
JP-2	1105.7	733703.71	2887641.90	36.0	1069.7
JP-3	1105.8	733483.09	2886974.16	35.4	1070.4
JP-4	1105.6	733323.27	2886393.14	47.7	1057.9
JP-4A	1105.3	733325.38	2886401.23	30.0	1075.3
JP-5	1104.5	732679.06	2886045.57	45.7	1058.8
JP-6	1106.3	732862.78	2886526.80	42.0	1064.3
JS-10	1085.0	736877.33	2892782.32	23.2	1061.8
JS-11	1115.3	736817.60	2892703.95	61.0	1054.3
JS-12	1114.8	736796.96	2892666.90	52.5	1062.3
JS-13	1132.5	736741.69	2892570.62	69.0	1063.5
JS-14	Boring Cancelled				
JS-15	1084.1	737186.07	2892539.85	25.5	1058.6
JS-16	1115.7	737079.51	2892528.69	61.5	1054.2
JS-17	1114.5	737004.19	2892496.33	54.5	1060.0
JS-18	1136.3	736848.84	2892429.18	76.5	1059.8
JS-19	1077.3	736913.99	2891993.30	20.0	1057.3
JS-20	1113.8	736826.84	2892070.81	61.5	1052.3
JS-21	1111.0	736784.15	2892107.96	51.8	1059.2
JS-22	1134.7	736662.66	2892209.60	74.7	1060.0
JS-23	1075.1	736562.81	2891652.34	17.1	1058.0
JS-24	1113.4	736463.59	2891743.40	58.7	1054.7
JS-25	1108.1	736417.96	2891781.01	48.5	1059.6
JS-26	1141.8	736300.23	2891894.54	90.0	1051.8

Table 4. Summary of Borings

Boring No.	Surface Elevation (ft)	Northing (ft)	Easting (ft)	Depth to Bottom of Hole (ft)	Elevation of Bottom of Hole (ft)
JS-27	1158.3	736239.87	2891944.24	97.5	1060.8
JS-28	1074.5	736010.84	2891176.23	18.3	1056.2
JS-29	1111.5	735935.78	2891247.73	52.0	1059.5
JS-30	1105.6	735899.72	2891288.23	49.2	1056.4
JS-31	1151.1	735755.45	2891418.56	98.8	1052.3
JS-32	1150.6	735766.70	2891431.00	67.0	1083.6
JS-33A	1152.4	735606.69	2891839.2	72.1	1080.3
JS-33B	1155.3	735313.55	2891533	72.8	1082.5
JS34A	1156.4	735400.64	2891943.1	74.6	1081.8
JS-34B	1156.3	735161.98	2891694.1	72.3	1084.0
JS-34C	1120.4	735045.58	2892079.28	36.9	1083.5
JS-35	1078.9	735547.59	2890689.83	22.3	1056.6
JS-36	1108.5	735478.03	2890742.60	52.0	1056.5
JS-36A	1106.2	735355.98	2890578.53	53.0	1053.2
JS-36B	1110.8	735703.43	2891025.07	56.6	1054.2
JS-37	1103.8	735429.18	2890784.99	43.2	1060.6
JS-37X	1104.4	735425.46	2890782.69	25.0	1079.4
JS-38	1151.5	735263.83	2890906.40	93.0	1058.5
JS-39	1181.3	735175.12	2890973.42	105.5	1075.8
JS-40	1170.2	735048.86	2891066.57	90.2	1080.0
JS-41	1154.6	734877.81	2891195.60	75.2	1079.4
JS-42	1138.2	734710.66	2891295.11	49.5	1088.7
JS-43	1081.5	735279.02	2890354.76	23.8	1057.7
JS-44	1103.2	735219.55	2890399.56	49.0	1054.2
JS-45	1101.3	735171.68	2890440.72	41.4	1059.9
JS-45X	1101.5	735168.74	2890438.03	24.5	1077.0
JS-46	1144.7	735006.11	2890560.28	82.0	1062.7
JS-47	1078.2	735013.36	2890001.65	18.0	1060.2
JS-48	1101.3	734956.57	2890044.99	35.0	1066.3
JS-49	1098.8	734898.66	2890091.75	27.1	1071.7
JS-50	1138.7	734760.24	2890196.57	66.3	1072.4
JS-51	Boring Cancelled				
JS-52	1136.8	734518.95	2890384.61	54.1	1082.7
JS-53	1081.4	734742.01	2889577.25	13.9	1067.5
JS-54	1100.2	734685.87	2889594.68	35.0	1065.2
JS-55	1097.4	734611.13	2889621.92	27.5	1069.9
JS-56	1131.0	734506.50	2889656.35	58.0	1073.0
JS-57	1130.1	734277.92	2889720.99	54.9	1075.2
JS-58	1100.2	734222.32	2889559.16	27.3	1072.9
JS-58X	1100.1	734224.38	2889557.53	27.5	1072.6
JS-59	1099.3	734047.10	2889202.69	31.1	1068.2
CPT-1	1109.5	735528.42	2890809.86	46.2	1063.3
CPT-2	1108.3	735472.49	2890736.90	47.8	1060.5
CPT-3	1107.1	735419.93	2890663.93	43.2	1063.9
CPT-4	1101.8	735439.57	2890778.44	37.8	1064.0
CPT-5	1100.0	735182.18	2890431.15	38.7	1061.3
JS-36-SV	1108.4	735481.63	2890746.85	42.0	1066.4

Table 4. Summary of Borings

Boring No.	Surface Elevation (ft)	Northing (ft)	Easting (ft)	Depth to Bottom of Hole (ft)	Elevation of Bottom of Hole (ft)
JS-37-SV	1102.3	735436.98	2890782.91	37.0	1065.3
JS-36A-SV	1106.4	735359.66	2890582.51	41.5	1064.9
JS-45-SV	1100.1	735181.14	2890438.31	31.5	1068.6
JS-60A	1089.5	736513.29	2891697.31	28.5	1061.5
JS-60B	1089.5	736515.46	2891699.27	28.0	1062.0
JS-61A	1089.7	735980.74	2891206.58	30.0	1059.7
JS-61B	1089.1	735978.47	2891204.07	17.0	1072.1
JS-62A	1090.0	735318.64	2890444.05	30.0	1060.0
JS-62B	1090.0	735316.23	2890442.25	30.0	1060.0
JS-62C	1088.2	735339.49	2890481.47	28.5	1059.7
JS-63A	1089.4	734985.29	2890020.63	27.0	1062.4
JS-63B	1089.4	734987.89	2890023.29	27.0	1062.4
JS-64	1082.3	735402.40	2890528.11	22.5	1059.8
JS-65A	1095.1	735271.28	2890430.29	36.5	1058.6
JS-65B	1094.7	735269.06	2890426.10	15.0	1079.7

6.3. Undisturbed Sampling

A total of thirty-one (31) undisturbed Shelby tube samples were obtained containing the fly ash and clay soils from ten (10) offset borings immediately adjacent to the standard penetration test borings. The undisturbed samples were retrieved using a 2 7/8-inch inside diameter, 30-inch long thin walled tubes and a piston sampler. The undisturbed soil samples were performed in general accordance with the procedures outlined in ASTM D1587, "Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes."

All Shelby tube samples were sealed with caps in the field and transported to either Stantec's laboratory in Lexington, Kentucky or Geocomp Corporation/Geotesting Express in Alpharetta, Georgia for testing. Testing of the recovered samples included unconsolidated undrained triaxial tests, consolidated undrained triaxial tests, unconfined compression tests, and falling head permeability tests. An inventory of recovered samples, including sample depth and percent recovery is presented in Table 5 below. Results including unit weight wet, unit weight dry, and normal moisture content are presented in Table 11 of the Laboratory Testing section of this report.

Table 5. Summary of Undisturbed Shelby Tube Samples

Boring No.	Sample	Depth (ft)	Sample Recovery (%)
JS-36-SV	ST-1	18.5-20.5	100
JS-36-SV	ST-2	28.5-30.5	50
JS-36-SV	ST-3	40.0-42.0	100
JS-37-SV	ST-1	18.0-20.0	85
JS-37-SV	ST-2	24.5-26.5	95
JS-37-SV	ST-3	35.0-37.0	65
JS-36A-SV	ST-1	34.5-36.5	100
JS-36A-SV	ST-2	39.5-41.5	85
JS-45-SV	ST-1	18.5-20.5	50
JS-45-SV	ST-2	24.5-26.5	100
JS-45-SV	ST-3	29.5-31.5	100
JS-60B	ST-1	5.0-6.3	65
JS-61B	ST-1	8.0-8.5	25
JS-61B	ST-2	12.0-13.0	50
JS-61B	ST-3	15.0-17.0	100
JS-62B	ST-1	3.0-4.0	50
JS-62B	ST-2	7.0-8.2	60
JS-62B	ST-3	14.0-16.0	100
JS-62B	ST-4	20.0-22.0	100
JS-62B	ST-5	23.0-25.0	100
JS-63B	ST-1	1.0-2.9	95
JS-63B	ST-2	5.0-7.0	100
JS-63B	ST-3	8.0-10.0	100
JS-63B	ST-4	11.0-13.0	100
JS-63B	ST-5	15.0-16.9	95
JS-65A	ST-1	28.5-30.5	100
JS-65B	ST-1	5.0-7.0	100
JS-65B	ST-2	10.0-11.5	75
JS-65B	ST-3	15.0-16.0	50
JP-4A	ST-1	10.0-12.0	100
JP-4A	ST-2	20.0-21.0	50

6.4. Vane Shear Testing

Four (4) vane shear test borings were advanced on the northern side of the Dry Fly Ash Stack adjacent to previously drilled sample borings JS-36, JS-36A, JS-37x, and JS-45x (see boring plan presented in Appendix B). The previous sample logs were used to estimate depths for each target soil horizon to determine where to advance the vane. The tests were performed in accordance with ASTM D 2573-08, “Standard Test Method for Field Vane Shear Test in Cohesive Soil.” Each boring had three vane shear tests conducted at various depths. These tests were performed to determine in-situ undrained shear strength of soils determined to be soft during previous standard penetration testing. Upon the conclusion of each vane shear test, an undisturbed Shelby tube sample was obtained below the vane shear test interval to conduct in-situ strength tests. The results from the vane shear tests were compared with laboratory shear strength tests from the undisturbed samples obtained during testing. Vane shear test results are presented on the drawings titled, “Logs of Borings” in Appendix B and on the borings logs in Appendix C. The summary of the vane shear testing is presented below in Table 6.

Table 6. Summary of Vane Shear Testing

Boring	Depth, (ft)	Soil Tested	Maximum Measured Torque, (In-lbs)	Vane Size	Undrained Shear Strength, (psi)	Residual Shear Strength, (psi)	Sensitivity
JS-45-SV	19.0	Sluiced Fly Ash	475	S	16.37	4.48	3.65
	25.0	Sluiced Fly Ash	60	S	2.07	1.38	1.50
	30.0	Sluiced Fly Ash	225	S	7.76	4.31	1.80
JS-36-SV	19.0	Sluiced Fly Ash	340	M	6.10	0.18	34.0
	29.1	Sluiced Fly Ash	480	M	8.62	1.31	6.58
	40.6	Alluvial Clay	620	M	11.13	3.77	2.95
JS-36A-SV	28.5	Sluiced Fly Ash	380	S	13.10	4.31	3.04
	35.0	Alluvial Clay	520	S	17.92	3.79	4.73
	40.0	Alluvial Clay	450	S	15.51	3.79	4.09
JS-37-SV	18.5	Sluiced Fly Ash	420	M	7.54	0.90	8.40
	25.0	Sluiced Fly Ash	390	M	7.00	0.90	7.80
	34.0	Sluiced Fly Ash	>600	M	Unknown	Unknown	Unknown

6.5. Cone Penetration Testing

Five (5) cone penetration test (CPT) borings were performed on the northern side of the Dry Fly Ash Stack adjacent to previously drilled sample borings JS-36, JS-37x, and JS-45x (see boring plan presented in Appendix B.) The previous sample logs were used to estimate/calibrate the depths for each soil horizon as the CPT testing was being performed. The CPT testing was performed in accordance with ASTM Standard D 5778, “Standard Test Method for Performing Electronic Cone and Piezocone Penetration Testing of Soils.” Cone penetration testing is used to determine soil properties and delineate soil stratigraphy by measuring tip resistance, sleeve friction, and dynamic pore pressure. Soil parameters determined by a CPT include, pore pressure, effective angle of internal friction, and un-drained shear strength. CPT test results were used to compare to laboratory shear strength test results. The results of the CPT testing can be found in Appendix H.

7. Field Instrumentation and Monitoring

7.1. General

As part of the geotechnical exploration, Stantec devised and implemented a slope monitoring program. The program started by installing instrumentation in the boreholes drilled for the geotechnical exploration. After taking initial or baseline instrumentation readings the monitoring of the dike slope conditions continued by obtaining periodic readings. The monitoring through the information obtained from the readings will continue until actions are implemented to provide adequate long term stability of the structure and beyond. Some of the instrumentation readings were also used to arrive to the results of the engineering analysis presented in this report. The following paragraphs provide additional details regarding the instrumentation and monitoring program.

7.2. Instrumentation

A total of forty three (43) borings were instrumented with 10 foot slotted screen piezometers (PZ) and two (2) borings were instrumented with a 5 foot slotted screen piezometers to monitor pore pressures at the specific depths and locations shown on the piezometer logs in Appendix D and on the graphical boring logs in Appendix B. In general, each piezometer screen was surrounded by an eleven foot thick sand filter pack, followed by a minimum two-foot thick bentonite seal, and then the annulus of the borehole was grouted to the surface with a bentonite/portland cement mix. Piezometer instrumentation logs can be found in Appendix D and piezometer readings can be found in Appendix E. Table 7 represents all piezometers installed at the John Sevier Fossil Plant.

Table 7. Summary of Piezometers Installed

Boring No.	PZ Tip Depth (ft)	PZ Tip Elevation (ft)	Cover Type
BA-1	37.1	1108.3	Flush Mount
BA-2	40.1	1105.8	Flush Mount
BA-3	34.8	1110.5	Flush Mount
BA-5	40.0	1104.9	Flush Mount
BA-8	34.5	1110.7	Flush Mount
JP-3	34.9	1070.9	Flush Mount
JP-4	46.0	1059.6	Flush Mount
JP-5	45.7	1058.8	Flush Mount
JP-6	40.6	1065.7	Flush Mount
JS-10	23.8	1061.2	Steel Riser
JS-12	52.2	1062.6	Steel Riser
JS-13	68.0	1064.5	Steel Riser
JS-15	24.7	1059.4	Flush Mount
JS-17	53.0	1061.5	Steel Riser
JS-18	66.1	1070.2	Steel Riser
JS-19	19.5	1057.8	Flush Mount
JS-21	45.0	1066.0	Steel Riser
JS-22	74.3	1060.4	Steel Riser
JS-23	16.0	1059.1	Flush Mount
JS-25	40.0	1068.1	Steel Riser
JS-27	80.0	1078.3	Temporary
JS-28	16.8	1057.7	Steel Riser
JS-30	30.0	1075.6	Steel Riser

Table 7. Summary of Piezometers Installed

Boring No.	PZ Tip Depth (ft)	PZ Tip Elevation (ft)	Cover Type
JS-32	66.0	1084.6	Temporary
JS-34C	21.5	1098.9	Steel Riser
JS-35	21.5	1057.4	Steel Riser
JS-37	24.0	1079.8	Steel Riser
JS-39	92.5	1088.8	Temporary
JS-42	46.5	1091.7	Flush Mount
JS-43	22.8	1058.7	Flush Mount
JS-45	24.5	1076.8	Steel Riser
JS-47	14.4	1063.8	Flush Mount
JS-49	25.5	1073.3	Steel Riser
JS-50	62.0	1076.7	Steel Riser
JS-52	45.0	1091.8	Steel Riser
JS-53	13.4	1068.0	Flush Mount
JS-55	17.0	1080.4	Steel Riser
JS-56	57.0	1074.0	Steel Riser
JS-57	48.3	1081.8	Steel Riser
JS-58	27.5	1072.7	Steel Riser
JS-59	31.1	1068.2	Flush Mount
JS-60B	27.0	1062.5	Steel Riser
JS-61A	25.5	1064.2	Steel Riser
JS-62B	29.3	1060.7	Flush Mount
JS-63B	24.2	1065.2	Steel Riser

A total of fifteen (15) borings were instrumented with 2.75 inch OD slope inclinometer (SI) casing to monitor potential subsurface lateral movement. Stantec has been taking inclinometer readings once a month since their installation. The displacement curves for the slope inclinometer readings and the maximum displacement observed for each of the slope inclinometers are presented in Appendix E. Table 8 represents all slope inclinometers installed at the John Sevier Fossil Plant.

Table 8. Summary of Slope Inclinometers Installed

Boring No.	Bottom of Casing Depth (ft)	Bottom of Casing Elevation (ft)	Cover Type
JS-11	59.8	1055.5	Flush Mount
JS-16	61.5	1054.2	Flush Mount
JS-20	61.5	1052.3	Flush Mount
JS-24	58.7	1054.7	Flush Mount
JS-26	89.5	1052.3	Steel Riser
JS-29	52.0	1059.5	Flush Mount
JS-31	98.5	1052.6	Steel Riser
JS-36	52.0	1056.5	Flush Mount
JS-36A	53.0	1053.2	Flush Mount
JS-36B	56.6	1054.2	Flush Mount
JS-38	91.5	1060.0	Steel Riser

Table 8. Summary of Slope Inclinometers Installed

Boring No.	Bottom of Casing Depth (ft)	Bottom of Casing Elevation (ft)	Cover Type
JS-44	49.0	1054.2	Flush Mount
JS-46	81.3	1063.4	Steel Riser
JS-48	34.3	1067.0	Flush Mount
JS-54	35.0	1065.2	Flush Mount

7.3. Monitoring of Dike Slope Conditions

Stantec began a monitoring program upon installation of instruments listed above. The purpose of the monitoring program was to obtain periodic water level readings from piezometers and slope movement data from slope inclinometers. Piezometer readings were taken using a water level indicator and slope inclinometer readings were obtained using a portable traversing inclinometer probe designed for this purpose. The first slope inclinometer survey established the initial profile of the casing and subsequent surveys measured changes in the profile of the casing if movement has occurred around the casing.

Stantec's schedule for monitoring program is presented in Table 9. Results of monitoring program are presented in Appendix E in the following order:

- Attachment 1 – PZ Readings, and
- Attachment 2 – SI Readings

Table 9. Monitoring Program Schedule

Reading Number	Date of PZ Reading	Date of SI Reading
1	May 19, 2009	June 4, 2009
2	May 21, 2009	June 16, 2009
3	June 3, 2009	June 29, 2009
4	June 17, 2009	July 13, 2009
5	June 29, 2009	July 31, 2009
6	July 13, 2009	August 12, 2009
7	July 30, 2009	September 8, 2009
8	August 13, 2009	October 13, 2009
9	September 8, 2009	November 11, 2009
10	October 13, 2009	December 12, 2009
11	November 12, 2009	January 12, 2010
12	December 9, 2009	
13	January 12, 2010	

7.4. Slug Testing

In addition to obtaining water level readings at frequent intervals, Stantec also performed slug testing on piezometers. The slug tests were performed in general accordance with ASTM D 4044 titled, "Standard Test Method for (Field Procedure) for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers." A pressure transducer with a data recorder manufactured by In-Situ, Inc. was used to collect water level information from wells with a riser pipe of sufficient diameter to accommodate the instrument.

All wells were tested by taking an initial measurement of static water level and then the pressure transducer was placed into the well. Approximately, a half gallon of water was then poured into the well to cause a nearly instantaneous change in the water level. The water levels were then recorded at regular intervals until reaching near static levels. The results were recorded electronically and downloaded into a data collector. Raw data was checked in the field for any discrepancies prior to demobilizing from the site.

The field data, once collected and returned to the office, was entered into AQTESOLV software program to estimate the hydraulic conductivity of the in-situ soils. The software utilized the Bouwer-Rice solution for a slug test in an unconfined aquifer to estimate the hydraulic conductivity of the subsurface soil. The hydraulic conductivity is estimated for the strata of soil that the piezometer screen is set in. Results from the slug testing data are presented in Appendix E.

8. Surveying

8.1. General

Topographic mapping of the John Sevier Fossil Plant (developed from aerial photographs) and contour mapping of the river bank along the plant facility (developed from a hydrographic field survey) were provided by TVA. Stantec's scope of work included a field topographic survey of selected areas located on the Dry Fly Ash Stack and Ash Disposal Area J. A summary of survey data obtained is presented in the following paragraphs.

8.2. Aerial Survey

TVA provided topographic mapping developed by Tuck Mapping Solutions, Inc. of the overall John Sevier Fossil Plant based on aerial photographs taken in March, 2009. The results of aerial survey can be seen on the base map presented in Appendix B.

8.3. Topographic Survey

Stantec requested a field topographic survey in July, 2009 of the north dike of the Dry Fly Ash Stack extending from the river bank to the perimeter road at approximately elevation 1105. A second field topographic survey was completed in October, 2009 of the north dike of the Ash Disposal Area J extending from the river bank to sixty feet south of the existing dike centerline. The objective of this work was to supplement the aerial mapping with a more accurate survey of the following features:

(i) Slopes
(ii) Embankments

(iii) Bench dimensions
(iv) Drainage ditches,
(v) Pipe inverts, and
(vi) Obscured aerial mapping areas

The results of Stantec's topographic surveys were applied to the cross section profiles used for stability analysis. Selected cross sections are presented in Appendix B.

8.4. Hydrographic Survey

At the request of Stantec, TVA Surveying and Project Services also performed a hydrographic survey of the river banks along the Dry Ash Disposal Stack and Ash Disposal Area J in September, 2009 to supplement land and aerial survey data. The combined survey information was used to aid in slope stability analyses and support site repair recommendations.

9. Laboratory Testing

9.1. General

The soil samples obtained during the geotechnical exploration were subjected to laboratory tests by Stantec in Lexington, Kentucky and by GeoComp Corporation/Geotesting Express Inc. in Alpharetta, Georgia. The laboratory tests were performed in accordance with ASTM standard testing procedures. Detailed results of laboratory testing are presented in Appendix F.

9.2. Laboratory Tests Performed

Stantec performed laboratory testing of all materials encountered to estimate their engineering properties. Geotesting Express Inc. was subcontracted by Stantec to assist in performing laboratory testing on specific undisturbed and disturbed soil samples. A summary of laboratory tests performed is presented in Table 10.

Table 10. Laboratory Tests Performed

Group *	Testing for	Standard
1	Natural Moisture Content	ASTM D 2216
2	Classification	ASTM D 2487
	Particle Size Analysis	ASTM D 422
	Density	ASTM D 2937
	Atterberg Limits	ASTM D 4318
	Specific Gravity	ASTM D 854
3	Standard Proctor	ASTM D 699
4	Falling Head Permeability	ASTM D 5084
5	Consolidated Undrained Triaxial (CU)	ASTM D 4767
6	Unconfined Undrained Triaxial (UU)	ASTM D 2850
7	Unconfined Compression Test (UC)	ASTM D 2166

* Results Presented in this order in Appendix F.

9.3. Natural Moisture Content

Natural moisture content tests were performed on all split-spoon samples, disturbed bulk samples, and undisturbed Shelby tube samples. For fly ash samples, an oven drying temperature of 60ºC was used and for all other soils encountered, an oven temperature of 110ºC was used to determine the natural moisture content. The results of moisture content determinations are presented in Attachment 1 of Appendix F.

9.4. Specific Gravity

Specific gravity tests at 20 degrees Celsius were performed on selected undisturbed Shelby tube samples and disturbed bulk samples. The results of these tests were used during soil classification.

9.5. Particle Size Analysis

Particle size distribution tests were performed on seventy one (71) total bulk samples. Fifty one (51) bulk samples of soils encountered at the Dry Fly Ash Stack were analyzed; sixteen (16) bulk samples from auger cuttings of clay were analyzed from the Ash Disposal Area J; and two (2) composite samples from SPT samples of clay were analyzed from the Bottom Ash Disposal Area 2. The tests were performed in accordance with ASTM D 422, "Particle Size Analysis of Soils," using sieve analysis for the soil fraction greater than 0.074 mm (No. 200 sieve size) and hydrometer analysis for the fraction smaller than 0.074 mm. The tests were performed on the predominant soil types to supplement the visual classifications made by the engineer/geologist in the field. The individual grain size distribution curves generated from these tests are presented as Attachment 2 of Appendix F.

9.6. Density

The undisturbed Shelby tube samples obtained from the subsurface exploration were extruded and trimmed into six-inch specimens in the laboratory. The trimmings from each specimen were used to determine the natural moisture content and the sample size and weight. The respective dry density for each sample was then calculated from the total density, the moisture content measurement, and sample dimensions.

9.7. Shear Strength

Thirty six (36) consolidated undrained (CU) triaxial tests were performed on undisturbed Shelby tube samples and disturbed bulk remolded samples from the Dry Fly Ash Stack, five (5) CU triaxial test were performed on undisturbed Shelby tube samples and disturbed bulk samples from the Ash Disposal Area J, and six (6) CU triaxial test were performed on disturbed composite bulk remolded samples from the Bottom Ash Disposal Area No. 2. These tests were performed in accordance with ASTM D 4767. Nine (9) unconsolidated undrained triaxial tests were performed on undisturbed soil specimens from the Dry Fly Ash Stack, in accordance with ASTM D 2850. One (1) unconfined compression test was performed on an undisturbed soil sample from the Dry Fly Ash Stack, in accordance with ASTM D2166. All tests were performed to obtain shear strength parameters for use in stability analysis. The test results are presented in Attachments 5, 6, and 7 of Appendix F. The summary of unit weight and moisture content values obtained from undisturbed Shelby tube samples is presented below in Table 11.

Table 11. Unit Weight and Moisture Content for Undisturbed Shelby Tube Samples

Boring No.	Depth (ft)	Unit Weight Dry (pcf)	Unit Weight Wet (pcf)	Normal Moisture Content (%)
JP-4A	20.0-20.6	96.7	116.7	20.6
JP-4A	11.3-11.9	98.4	122.3	24.3
JP-4A	10.7-11.3	107.0	122.9	14.9
JP-4A	10.1-10.7	119.4	126.9	6.3
JS-36 SV	19.1-19.6	55.4	93.7	69.2
JS-36 SV	29.0-29.5	59.0	94.8	60.7
JS-36 SV	18.5-19.0	70.9	105.1	48.2
JS-36 SV	19.9-20.4	59.3	92.7	56.4
JS-36 SV	40.5-41.0	94.6	121.1	28.0
JS-36 SV	41.0-41.5	89.9	116.5	29.6
JS-36 SV	41.5-42.0	90.1	118.9	32.0
JS-36A SV	40.4-40.9	87.5	116.8	33.5
JS-36A SV	39.7-40.2	86.6	115.3	33.1
JS-36A SV	34.5-35.0	103.9	124.0	19.3
JS-37 SV	35.0-35.5	112.8	132.2	17.2
JS-45 SV	30.6-31.5	57.5	92.0	60.0
JS-45 SV	18.5-19	73.0	106.1	45.3
JS-45 SV	24.5-25	71.8	98.0	36.5
JS-45 SV	25.2-25.7	55.9	92.2	64.9
JS-45 SV	25.8-26.3	51.5	89.4	73.6
JS-45 SV	29.5-30.0	65.8	98.0	49.0
JS-45 SV	30.1-30.6	55.6	94.2	69.4
JS-60B	5.0-5.6	105.0	127.3	21.3
JS-61B	15.5-16.0	99.2	123.9	24.9
JS-61B	16.0-16.5	100.1	126.3	26.2
JS-61B	16.5-17.0	105.4	129.0	22.4
JS-61B	8.0-8.5	114.4	134.8	17.9
JS-62B	14.1-14.7	110.2	131.5	19.4
JS-62B	15.4-16.0	114.2	133.8	17.1
JS-62B	14.8-15.4	114.4	136.1	19.0
JS-62B	24.4-24.9	88.0	116.7	32.7
JS-62B	23.8-24.4	91.9	119.4	30.0
JS-62B	23.3-23.8	99.9	123.4	23.5
JS-62B	20.7-21.3	104.7	128.4	22.6
JS-62B	21.3-21.9	109.2	131.2	20.1
JS-62B	20.1-20.6	111.6	133.0	19.1
JS-62B	7.7-8.2	111.3	131.2	17.9
JS-62B	7.0-7.7	113.1	130.8	15.6
JS-63B	1.7-2.3	104.4	120.6	15.5
JS-63B	5.5-6.0	105.6	126.5	19.8
JS-63B	6.0-6.5	106.8	128.8	20.5
JS-63B	1.2-1.7	109.0	126.3	15.8

Table 11. Unit Weight and Moisture Content for Undisturbed Shelby Tube Samples

Boring No.	Depth (ft)	Unit Weight Dry (pcf)	Unit Weight Wet (pcf)	Normal Moisture Content (%)
JS-63B	6.5-7.0	110.8	131.3	18.5
JS-63B	2.3-2.8	112.1	128.6	14.7
JS-63B	11.3-11.8	103.2	124.4	20.6
JS-63B	8.8-9.4	104.7	126.6	21.0
JS-63B	11.8-12.3	105.0	124.8	18.9
JS-63B	15.1-15.7	107.7	129.4	20.1
JS-63B	12.3-12.9	108.2	128.8	19.1
JS-63B	8.2-8.8	109.1	131.8	20.8
JS-63B	9.4-10.0	109.8	133.6	21.7
JS-65A	28.6-29.2	102.1	127.2	24.6
JS-65A	29.2-29.8	103.7	127.5	23.0
JS-65A	29.8-30.4	105.3	128.1	21.7
JS-65B	5.7-6.3	110.3	131.0	18.7
JS-65B	6.3-6.9	113.6	137.4	20.9
JS-65B	15.1-15.8	102.9	123.8	20.4
JS-65B	10.2-10.8	99.2	120.3	21.2
JS-65B	10.8-11.3	108.4	131.2	21.0

9.8. Permeability

Falling head permeability tests were performed on one undisturbed fly ash sample and one alluvial clay sample from the Dry Fly Ash Stack. The tests were performed in tri-axial cells in general accordance with ASTM D 5084, "Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials using Flexible Wall Permeameter. Confining pressures ranging from 5 to 10 psi were used during the testing and a back pressure of 65 psi was used to achieve saturation. The summary of permeability tests conducted is presented below in Table 12 and complete test results are provided in Attachment 4 of Appendix F.

Table 12. Summary of Falling Head Permeability Test Results

Facility	Boring	Soil Horizon	Test Interval (feet)	Initial Conditions				Coefficient of Permeability Kv (cm/sec)
				Dry Density (pcf)	Moisture Content (%) @ 20° C	Void Ratio, e	Specific Gravity, Gs	
Dry Stack	JS-45-SV	Fly Ash	30.6-31.5	57.5	60.0	1.519	2.32	5.44E-05
Dry Stack	JS-36-SV	Alluvial Clay	41.5-42.0	90.1	32.0	0.864	2.69	3.27E-07

9.9. Classification Testing and Proctor Testing

Soil classification testing consisting of Atterberg limits, particle-size analysis, specific gravity, and standard proctor testing were performed on select undisturbed Shelby tube samples and disturbed bulk samples. These tests are used specifically for classifying the different soil strata. The results can be found in Attachments 2 and 3 of Appendix F.

10. Results of Field Exploration & Laboratory Testing

10.1. Dry Fly Ash Stack

10.1.1. Subsurface Soil Conditions

The subsurface conditions encountered during the geotechnical exploration of the Dry Fly Ash Stack were dependent on the vertical location of the borings. In general, borings advanced above elevation 1110 feet encountered three or more of seven predominant soil types. These included clay fill (cap material), compacted fly ash fill, bottom ash fill, sluiced fly ash fill, alluvial clay, alluvial gravel and alluvial sand. Borings advanced below elevation 1110 feet (upper perimeter road) but above the lower perimeter road encountered a clay fill layer (cap material) underlain by what is believed to be original starter dike clay, alluvial clay, and alluvial gravel and sand. Borings advanced along the lower perimeter road encountered mostly alluvial materials consisting of clay, sand and gravel. Logs of sample borings are presented in Appendix C. Table 13 below presents a summary of laboratory classification test for the Dry Fly Ash Stack.

Clay fill (Soil 1) or cap material, typically located above ash deposits, was visually classified in the field as clay with sand and gravel, light brown to brown, soft to stiff, moist, with occasional silty zones. Bulk samples of this material were classified in the laboratory as sandy lean clay (CL) having an average plasticity index of 14 and specific gravity of 2.6. N-values (determined from SPT blow counts) ranged from 2 to greater than 30. The moisture content (determined from SPT samples) ranged from 11 to 28 percent.

Compacted or dry fly ash (Soil 4) was visually classified in the field as fly ash, gray to dark gray and black, dry to wet, very loose to very dense, with occasional clay seams, gravel, coal fragments, and traces of bottom ash. Bulk samples of this material were classified in the laboratory as silt with sand (ML), non-plastic, having an average specific gravity of 2.4. N-values (determined from SPT blow counts) ranged from less than 4 to greater than 51.

Sluiced fly ash (Soil 5) was found to typically exist below elevation 1095 feet and between the compacted fly ash and alluvial clay soil horizons. Sluiced fly ash was visually classified in the field as very loose and saturated fly ash. N-values for this material were typically less than four (<4) including intervals where only the weight of rod (WOR) or weight of hammer (WOH) were needed to advance the spoon.

Bottom ash (Soil 3) was visually classified in the field as bottom ash, dark gray to black, dry to wet, very loose to very dense, medium to very coarse grained, and angular. Bulk samples were classified in the laboratory as silty sand (SM), non-plastic, having an average specific gravity of 2.4. N-values (determined from SPT blow counts) ranged from less than 4 to greater than 50.

Dike material (Soil 8) was visually classified in the field as lean clay with sand and silt, light brown to brown and gray, medium stiff to very stiff, moist, with traces of gravel and manganese concretions. A bulk sample was classified in the laboratory as lean clay with sand (CL), having a plasticity index of 20 and specific gravity of 2.7. The N-values (determined from SPT blow counts) ranged from 4 to 30 with an average of 14. The moisture content, (determined from SPT samples) ranged from 11 to 25 percent and having an average of 19 percent.

Alluvial clay (Soil 2) was visually classified in the field as clay with sand, brown to tan, soft to stiff, moist to wet, with occasional manganese concretions, silty zones, and gravel. Bulk samples of this material were classified in the laboratory as lean clay with sand (CL), having an average plasticity index of 18 and specific gravity of 2.7. Alluvial clay was also identified as Soil 9 in a limited number of sample borings. This material was visually classified in the field as clay with silt, dark brown to dark gray, very soft to stiff, with occasional manganese concretions and gravel. N-values for alluvial clays (determined from SPT blow counts) ranged from less than 2 to greater than 30. The moisture content (determined from SPT samples) ranged from 16 to 40 percent.

Alluvial sand (Soil 7) and gravel (Soil 6), were typically encountered in thin zones above the shale bedrock. No laboratory classifications were performed on these materials. The sand was visually classified in the field as brown and tan, medium grained, moist, and loose to very dense. The gravel was visually classified in the field as brown and tan, medium grained, dry to wet, loose to very dense, poorly graded with sand. The N-values for both sand and gravel (determined from SPT blow counts) ranged from 4 to greater than 50. No laboratory classifications were performed on these materials.

Table 13. Summary of Laboratory Test Results – Dry Fly Ash Stack

Soil Type	Boring	Depth (feet)	Unified Class	Plasticity Index	Specific Gravity	Gravel & Sand (%)	Silt & Clay (%)
Alluvial Clay (Soil 2)	JS-11	31.5-43.5	CL	20	2.66	38.6	61.4
Alluvial Clay (Soil 2)	JS-12	28.5-46.5	CL	17	2.69	25.2	74.8
Alluvial Clay (Soil 2)	JS-60A	13.5-21.0	CL	17	2.70	26.1	73.9
Bottom Ash (Soil 3)	JS-33A	40.5-46.5	SM	NP	2.21	52.7	47.3
Bottom Ash(Soil 3)	JS-36B	13.5-15.0	SM	NP	2.52	55.1	44.9
Clay Fill (Soil 1)	JS-36A	10.5-18.0	CL	15	2.68	39.9	60.1
Clay Fill (Soil 1)	JS-36B	4.7-7.5	CL	11	2.58	31.2	68.8
Clay Fill (Soil 1)	JS-36B	18.0-27.0	CL	15	2.67	31.3	68.7
Dry Fly Ash (Soil 4)	JS-11	13.5-31.5	ML	NP	2.36	22.2	77.8
Dry Fly Ash (Soil 4)	JS-12	2.8 - 7.5	ML	NP	2.43	21.8	78.2
Dry Fly Ash (Soil 4)	JS-12	13.5 - 18.0	ML	NP	2.25	16.9	83.1
Dry Fly Ash (Soil 4)	JS-13	3.0-9.0	ML	NP	2.38	9.7	90.3
Dry Fly Ash (Soil 4)	JS-13	18.0-21.0	ML	NP	2.32	25.3	74.7
Dry Fly Ash (Soil 4)	JS-16	16.5-22.5	ML	NP	2.32	15.4	84.6
Dry Fly Ash (Soil 4)	JS-17	4.0-13.5	ML	NP	2.37	18.7	81.3
Dry Fly Ash (Soil 4)	JS-17	18.0-22.5	ML	NP	2.25	11.5	88.5
Dry Fly Ash (Soil 4)	JS-20	7.5-22	ML	NP	2.37	24.2	75.8

Table 13. Summary of Laboratory Test Results – Dry Fly Ash Stack

Soil Type	Boring	Depth (feet)	Unified Class	Plasticity Index	Specific Gravity	Gravel & Sand (%)	Silt & Clay (%)
Dry Fly Ash (Soil 4)	JS-21	2.5-7.5	ML	NP	2.33	18.4	81.6
Dry Fly Ash (Soil 4)	JS-25	2.6-11.7	ML	NP	2.43	48.3	51.7
Dry Fly Ash (Soil 4)	JS-25	11.7-21.0	ML	NP	2.30	14.1	85.9
Dry Fly Ash (Soil 4)	JS-30	3.0-7.5	ML	NP	2.41	41.3	58.7
Dry Fly Ash (Soil 4)	JS-30	19.5-24.0	ML	NP	2.23	11.2	88.8
Dry Fly Ash (Soil 4)	JS-31	13.5-18.0	ML	NP	2.44	29.3	70.7
Dry Fly Ash (Soil 4)	JS-31	48.0-51.0	ML	NP	2.37	36.6	63.4
Dry Fly Ash (Soil 4)	JS-33A	0.0-15.0	ML	NP	2.22	11.9	88.1
Dry Fly Ash (Soil 4)	JS-33B	0.0-15.0	ML	NP	2.28	13.3	86.7
Dry Fly Ash (Soil 4)	JS-34A	0.0-15.0	ML	NP	2.27	18.1	81.9
Dry Fly Ash (Soil 4)	JS-34B	0.0-15.0	ML	NP	2.25	22.7	77.3
Dry Fly Ash (Soil 4)	JS-37	4.5-10.0	ML	NP	2.36	26.2	73.8
Dry Fly Ash (Soil 4)	JS-38	7.5-13.8	ML	NP	2.30	14.7	85.3
Dry Fly Ash (Soil 4)	JS-38	45.0-48.0	ML	NP	2.33	33.5	66.5
Dry Fly Ash (Soil 4)	JS-39	22.5-30.0	ML	NP	2.34	15.0	85.0
Dry Fly Ash (Soil 4)	JS-40	0.0-15.0	ML	1	2.51	24.6	75.4
Dry Fly Ash (Soil 4)	JS-41	0.0-15.0	ML	NP	2.29	12.0	88.0
Dry Fly Ash (Soil 4)	JS-42	0.0-15.0	ML	NP	2.43	11.8	88.2
Dry Fly Ash (Soil 4)	JS-45	3.6-7.0	ML	NP	2.39	35.4	64.6
Dry Fly Ash (Soil 4)	JS-46	12.0-18.0	ML	NP	2.41	41.0	59.0
Dry Fly Ash (Soil 4)	JS-50	0.0-24.0	ML	NP	2.37	21.8	78.2
Dry Fly Ash (Soil 4)	JS-52	6.0-18.0	ML	NP	2.71	11.9	88.1
Dry Fly Ash (Soil 4)	JS-56	0.0-18.0	ML	NP	2.41	11.1	88.9
Dry Fly Ash (Soil 4)	JS-57	6.0-13.2	ML	NP	2.31	19.5	80.5
Dry Fly Ash (Soil 4)	JS-58	4.0-15.0	ML	NP	2.36	8.1	91.9
Sluiced Ash (Soil 5)	JS-34C	7.5-13.5	ML	NP	2.38	16.0	84.0
Sluiced Ash (Soil 5)	JS-45	7.0-15.0	ML	NP	2.31	19.9	80.1
Sluiced Ash (Soil 5)	JS-49	12.0-18.0	ML	NP	2.30	13.4	86.6
Dike (Soil 8)	JS-63A	9.0-15.0	CL	20	2.70	18.8	81.2

10.1.2. Bedrock Conditions

Rock coring was performed in two (2) borings advanced during this exploration. All other borings were terminated before encountering auger refusal. The underlying bedrock consists of the Ordovician age Sevier Shale Formation. The shale was visually classified as brown to gray, very thin to laminated bedding on high (45°) dip, with few seams of limestone, and weathered near the bedrock surface.

10.1.3. Subsurface Water

Forty Five (45) borings advanced at the Dry Fly Ash Stack were instrumented with slotted screen piezometers to measure subsurface water conditions over time. The presumed water level reading was initially recorded during the inspection of SPT samples. These depths to water are shown on the boring logs presented in Appendix C. Since their installation, water level readings in the piezometers have been obtained several times as summarized in Table 9, "Monitoring Program Schedule". On average the water elevation along the north side of the dry stack ranges from approximately elevation 1070 feet in the east to elevation 1076 feet in the west. Subsurface water elevations were observed to be higher on the southern side of the stack and ranged from 1086 feet in the east to 1089 feet in the west. This is consistent with the hydro-geological conditions of the site, which are influenced by the location of Holston River.

10.2. Bottom Ash Disposal Area 2

Nine (9) SPT borings were advanced at the Bottom Ash Disposal Area 2 and positioned on top of the existing dike near the exterior crest. The typical top of dike elevation was 1145 feet. These borings encountered two distinct clay zones above shale bedrock. The two clay zones were identified as either dike fill material (Soil 1) or foundation residual clay (Soil 10). Dike material was visually identified in the field as clay with sand and gravel, light brown to brown with occasional gray mottling, medium stiff to hard, moist, with occasional manganese concretions and silty zones. This material was classified in the laboratory as lean clay (CL) having a plasticity index of 26, specific gravity of 2.7, maximum dry density of 106.4 pcf, and an average moisture content (determined from SPT samples) of 22 percent. The N-value (determined from SPT blow counts) ranged from 6 to 43 with an average of 18.

Residual clay material, located below the clay dike, was visually identified in the field as clay, light brown to brown, stiff to hard, moist, to wet, with some manganese concretions. This material was classified in the laboratory as a lean clay (CL) having a plasticity index of 25, specific gravity of 2.7, maximum dry density of 101.5 pcf, and an average moisture content (determined from SPT samples) of 29 percent. The N-value (determined from SPT blow counts) ranged from 10 to 52 with an average of 21. Table 14 below presents a summary of laboratory classification test for the Bottom Ash Disposal Area 2 subsurface soil.

Table 14. Summary of Laboratory Test Results – Bottom Ash Disposal Area 2

Soil Type	Max Dry Density (pcf)	Optimum Moisture (%)	Unified Class	Plasticity Index	Specific Gravity	Gravel & Sand (%)	Silt & Clay (%)
Dike (Soil 1)	106.4	19.7	CL	26	2.70	11.4	88.6
Residual Clay (Soil 10)	101.5	20.5	CL	25	2.70	11.4	88.6

Although rock coring was not performed in borings located at the Bottom Ash Disposal Area 2, samples obtained from auger cuttings and standard penetration tests that penetrated the underlying bedrock suggest this area is underlain by the same shale formation encountered below the Dry Fly Ash Stack. This is confirmed by rock outcrop observed along Polly Branch Creek, which traverses immediately below the north slope of the area. Based on the SPT samples, the upper portion of the shale appears to be weathered to different depths. The top of the weathered zone was described as the top of rock during this geotechnical exploration. The top of rock ranges from elevation 1108 feet near the eastern side of the facility to elevation 1118 feet borings located near the western side of the facility.

Five (5) of the sample borings advanced at the Bottom Ash Disposal Area 2 were instrumented with slotted screen piezometers to measure subsurface water conditions over time. The presumed water level reading was initially recorded during the inspection of SPT samples. These water levels are shown on the boring logs presented in Appendix C. Since their installation, water level in these piezometers has been monitored, as summarized in Table 9, "Monitoring Program Schedule". The water elevation ranges from approximately 1111 feet to 1126 feet.

10.3. Ash Disposal Area J

Six (6) SPT borings were advanced at the Ash Disposal Area J and positioned on top of the existing dike near the exterior crest where the typical ground surface elevation is 1105 feet. These borings encountered four distinct soils above shale bedrock consisting of dike fill material and alluvial clay, sand, and gravel.

Two clay zones were identified as either dike fill material (Soil 1) or alluvial clay (Soil 2). The dike material was visually classified in the field as clay, light brown to brown, tan, with occasional gray mottling, medium stiff to hard, moist, with sand and gravel. This material was classified in the laboratory as a lean clay with sand (CL) having an average plasticity index of 25, specific gravity of 2.7, and an average moisture content (determined from SPT samples) of 18 percent. The N-value (determined from SPT blow counts) ranged from 6 to 43 with an average of 19.

The alluvial clay, one of the dike foundation materials, was visually identified in the field as clay, brown to dark brown, soft to very stiff, moist, with manganese concretions and sand. The material was classified in the laboratory as lean clay with sand (CL) having an average plasticity index of 19, specific gravity of 2.7, and an average moisture content (determined from SPT samples) of 22 percent. The N-value (determined from SPT blow counts) ranged from 4 to 28 with an average of 11.

Granular materials, alluvial sand and gravel, were discovered to typically exist in thin zones above the shale bedrock. No laboratory classifications were performed on these materials. The sand was visually classified in the field as brown and tan, medium grained, dry to wet, and loose to medium dense. The N-value (determined from SPT blow counts) ranged from 5 to 16. The gravel was visually classified in the field as brown and tan, medium grained, medium dense to very dense, poorly graded with sand. The N-value (determined from SPT blow counts) ranged from 20 to 95. Table 15 below presents a summary of laboratory classification tests for samples obtained at the Ash Disposal Area J.

Table 15. Summary of Laboratory Classifications – Ash Disposal Area J

Soil Type	Boring	Depth (feet)	Unified Class	Plasticity Index	Specific Gravity	Gravel & Sand (%)	Silt & Clay (%)
Dike (Soil 1)	JP-1	1.5-7.5	CL	28	2.73	30.3	69.7
Dike (Soil 1)	JP-1	19.5-28.5	CL	26	2.69	17.9	82.1
Dike (Soil 1)	JP-2	0.0-9.0	CH/CL	26	2.77	27.6	72.4
Dike (Soil 1)	JP-2	22.5-24.0	CL	24	2.70	21.6	78.4
Dike (Soil 1)	JP-3	6.5-11.5	CL	18	2.70	21.1	78.9
Dike (Soil 1)	JP-3	26.5-30.0	CL	21	2.67	27.7	72.3
Dike (Soil 1)	JP-4	0.0-11.5	CL	21	2.67	30	70.0
Dike (Soil 1)	JP-4	20.0-25.0	CL	26	2.72	28	72.0
Dike (Soil 1)	JP-5	6.5-16.5	CL	25	2.73	34.1	65.9
Dike (Soil 1)	JP-5	26.5-32.0	CH	33	2.73	42.9	57.1
Dike (Soil 1)	JP-5	36.5-40.0	CL	25	2.68	37	63.0
Dike (Soil 1)	JP-6	6.5-15.0	CH	29	2.76	38.5	61.5
Dike (Soil 1)	JP-6	26.5-34.5	CL	26	2.78	24.1	75.9
Alluvial Clay (Soil 2)	JP-4	25.7-30.0	CL	22	2.69	16.8	83.2
Alluvial Clay (Soil 2)	JP-4	37.5-45.0	CL	16	2.68	23.8	76.2

Rock coring was not performed in borings advanced at the Ash Disposal Area J. However, samples obtained from auger cuttings and standard penetration tests that extended into the underlying bedrock indicate that Area J is underlain by the same formation encountered at the Dry Fly Ash Stack. Also, this shale formation outcrops along the south flank of Holston River, immediately below the northern dike of Area J. The top of the weather zone was described as the top of rock during this geotechnical exploration. The top of rock ranges from approximately elevation 1073 feet along the northeastern side of the facility to elevation 1060 feet at the southwestern end of Area J.

Four (4) of the sample borings advanced at the Ash Disposal Area J were instrumented with slotted screen piezometers to measure subsurface water conditions over time. The presumed water level reading was initially recorded during the inspection of SPT samples. These depths to water are shown on the boring logs presented in Appendix C. Since their installation, the water level in the piezometers has been measured several times as noted in Table 9, “Monitoring Program Schedule”. The water elevation ranges on from approximately 1070 feet below the northern dike to 1085 feet below the southwestern dike.

11. Engineering Analyses

11.1. General

Based on the review of available information, results of geotechnical exploration and results of laboratory testing, Stantec performed engineering analyses of the three principal structures at John Sevier Fossil Plant. This included seepage and stability analyses of one

(1) cross section at the Bottom Ash Disposal Area 2 and slope stability analysis of eight (8) cross sections at the Dry Fly Ash Stack and four (4) cross sections of the Ash Disposal Area J. The procedure and results of the analyses are presented in the following paragraphs.

11.2. Seepage Analysis

11.2.1. Background

The objective of the seepage analysis was to understand the total head (and pore water pressure) distribution within a given cross section of the Bottom Ash Disposal Area 2 dike. Seepage analysis was performed using SEEP/W, a numerical software tool developed by Geo-Slope International Inc. SEEP/W is a finite element software product for analyzing groundwater seepage and excess pore-water pressure dissipation problems within porous materials such as soil and rock.

The first step in the seepage analysis was to develop several cross sections of the dike and select a typical one for the analysis. Stantec utilized a combination of boring logs, piezometer data, historic drawings and topographic and hydrographic survey information to estimate the dimensions of the cross section and build its geometry. SEEP/W uses the concept of regions and points to define the geometry of a problem and to facilitate discretization (or meshing) of the problem. Upon defining the geometry of the model (with automatic mesh generation), material properties were assigned using the *Saturated/Unsaturated Model* available in SEEP/W. The next step in the process was to define boundary conditions. All boundary conditions were applied directly on geometry items such as region faces and region lines. Upon defining the boundary conditions, the model was analyzed using the *Steady State* seepage option available in SEEP/W based on the assumption that the boundary conditions are constant over time. Specific details regarding the analysis procedure are presented in the following sections.

11.2.2. Cross Sections

Seepage analysis was performed for existing ground conditions of cross section I-I’, where boring BA-7 was advanced (see Figure 9).

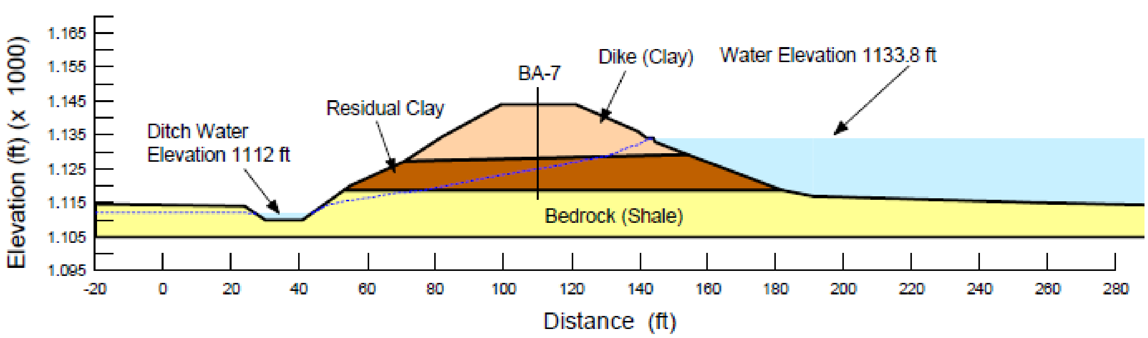


Figure 9. Cross Section I-I’

11.2.3. Material Properties

After developing a representative subsurface profile, material properties were estimated based on available laboratory data, slug testing, and typical values for similar soils. Material properties used in the seepage analysis are summarized below in Table 16.

Significant engineering judgment is needed to select appropriate hydraulic properties for earth materials. Unlike other key properties, hydraulic conductivity can vary over several orders of magnitude for a range of soils, often with substantial anisotropy for seepage in horizontal versus vertical directions. Laboratory test samples often do not represent important variations within a larger soil deposit. For the Bottom Ash Disposal Area 2, 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 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 residual lean clay soil horizon. An isotropic material (sands and gravels) would have $k_h/k_v = 1$, while deposits of horizontally layered soils (silt, fly ash) might have values as high as $k_h/k_v = 100$. For the Bottom Ash Pond Area 2, a ratio of 20 was assumed for the lean clay fill and residual lean clay to represent both naturally deposited material and material that would have been placed and compacted in lifts. A ratio of 10 was assumed for the shale bedrock material to represent the tight horizontal bedding planes.

Table 16. Material Properties used for Seepage Analysis

Material	K_h (ft/sec)	K_v/ K_h	K_h/K_v	Gs	e	w-sat (%)	w-res (%)
Dike (Clay) (Soil 1)	1E-8	0.05	20	2.7	0.7	25	2
Residual Clay (Soil 10)	9.234E-7	0.05	20	2.7	0.67	25	2
Bedrock (Shale)	8.166E-6	0.10	10	2.6	0.25	20	1

Where,

- k_v is the vertical hydraulic conductivity
- k_h is the horizontal hydraulic conductivity
- G_s is the specific gravity
- e is the void ratio
- w_{sat} is the saturated water content, and
- w_{res} is the residual water content

Horizontal Hydraulic Conductivity (K_h): The K_h values of Soil 2 (Residual Lean Clay) and Shale materials were estimated using slug test performed on similar soils in proximity to cross section I-I'. Slug testing was performed at all piezometers installed at the Bottom Ash Disposal Area 2. The results of the slug testing are presented in Appendix E. The K_h value for Soil 1-Lean Clay were assumed based on similar soil characteristics examined at another TVA facility. It was thus determined that Soil 1 was approximately two orders of magnitude higher than the underlying Soil 2.

Vertical Hydraulic Conductivity (K_v): The K_v values of all materials were based on the estimated ratio of K_v to K_h. The ratios of K_h to K_v for Soil 1- Lean Clay and Soil 2- Lean Clay were assumed based on similar soil characteristics examined at another TVA facility. The ratio for the shale was selected to be consistent with the general bedding nature of this material.

Specific Gravity (G_s): The G_s values of the two clay materials were estimated based on the laboratory test results presented in Appendix F. The G_s value for shale was assumed based on published values for similar material and upon values used for shale at other TVA facilities.

Void Ratio (e): The *e* values of the two clay materials were estimated based on the laboratory test results presented in Appendix F. The *e* value shale was assumed based upon published values of similar materials and consistent with values used at other TVA facilities.

Saturated Water Content (w-sat): The *w-sat* values of all materials were based upon phase relationships for fully saturated materials augmented by published values for similar materials.

Residual Water Content (w-rest): The *w-res* values of all materials were assumed using the reference Rawls et al.'s "Estimation of Soil Water Properties".

After the initial seepage parameters were estimated, results from the SEEP/W model were compared to pore water pressures measured in a nearby piezometer. Nodes were placed in the model at the same location as the piezometer tip was installed in the field, and then the total head predicted at the node was compared to the piezometer reading.

The material parameters listed in Table 16 vary slightly from the originally assumed values so that the final soil parameters resulted in a general agreement between the measured total head within the piezometer and the total head calculated from SEEP/W/

11.2.4. Results

Detailed results of seepage analysis are presented in Appendix I. A discussion of the results is presented in the following paragraphs.

The total head distribution for cross section I-I' is presented in Figure 10. Table 17 presents a comparison of the SEEP/W results (total head) with the average measurements taken from piezometer BA-8.

Table 17. Total Head Measurements

Cross-Section	Piezometer	Tip Elevation (feet)	Pond Pool Elevation (feet)	Drainage Ditch Pool Elevation (feet)	SEEP/W Phreatic Elevation (feet)	Average Field Measurement Phreatic Elevation (feet)
I-I'	BA-8	1110.7	1133.8	1112.0	1125.5	1126.0

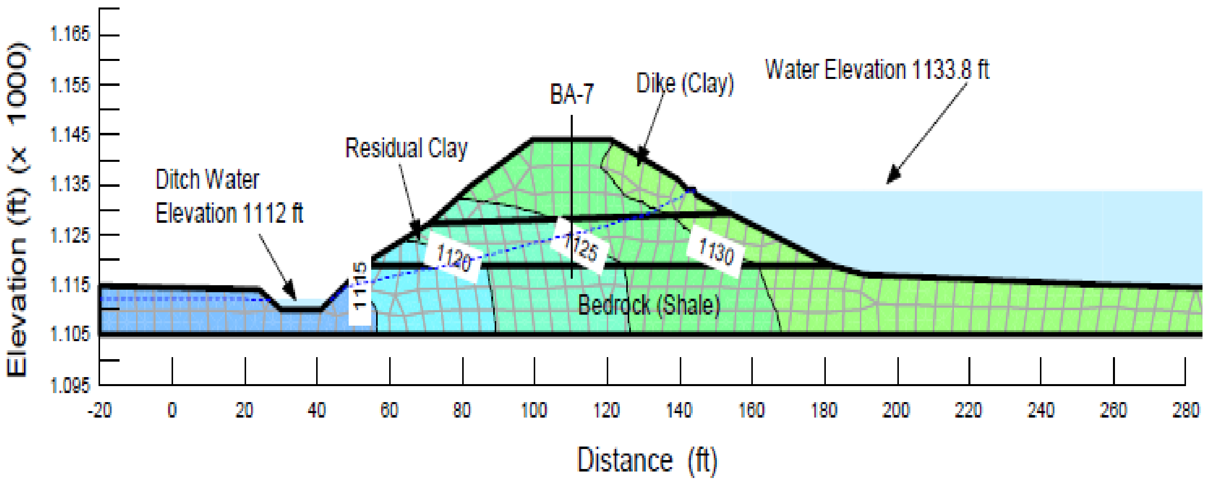


Figure 10. Cross Section I-I' (Total Head Contours in Feet)

The results from the seepage analysis were also examined to identify conditions where piping and erosion of soil might develop due to seepage forces. All earth embankments allow some amount of water to seep through the structure. However, if excessive hydraulic gradients develop through the embankment or foundation soils, then fine particles within the embankment may become transported (piped) out of the embankment. If left unattended, this slow internal erosion could then result in a failure of the earthen structure. Several factors, such as the type of foundation soils, embankment materials, embankment construction, compaction and pipe penetration, can lead to piping issues within earthen structure. Therefore, routine inspections are critical in identifying potential problem areas and arrest any piping issues prior a slope failure.

The model results indicated a shallow phreatic surface (ground water table) at the northern toe of the dike within the shale bedrock. The factor of safety with respect to soil piping (FS_{piping}) was computed for the surficial 3 to 5 feet of soil in this area (see Table 18). The factor of safety with respect to soil piping (FS_{piping}) is defined as:

$$FS_{piping} = \frac{i_{crit}}{i} \qquad \textbf{Eqn. 1}$$

Where:

i = the vertical gradient of a flow vector at a particular node
i_{crit} = is the critical gradient, a material property of the soils at the node

The critical gradient (i_{crit}) is related to the submerged unit weight of the soil and can be computed as:

$$i_{crit} = \frac{\gamma_{sub}}{\gamma_w} = \frac{G_s - 1}{1 + e} \qquad \textbf{Eqn. 2}$$

Where:

γ_{sub} = the submerged unit weight of the soil, γ_w is the unit weight of water,
G_s = the specific gravity of the soil particles

e = the void ratio.

For nearly all soils, the critical gradient is between about 0.6 and 1.4, with a typical value near 1.0.

Where FS_{piping} = 1, the effective stress is zero and the near-surface soils are subject to piping or heaving. Note that Eqn. 1 is valid only for vertical seepage that exits to the ground surface. If the phreatic surface is buried, then the FS_{piping} will be greater than 1.0 even when $i=i_{crit}$.

Table 18. Summary of Computed Exit Gradients and Factors of Safety against Piping

Vertical Gradient (i_v) at Critical Exit Point*	Critical Gradient (i_{crit})	Location of Maximum Vertical Gradient	FS_{piping}
0.2	1.28	Shale	6.4

The United States Army Corps of Engineers (USACE) design criteria (EM 1110-2-1901) indicates factors of safety against piping should be at least 3.0. The vertical gradient contours for cross section I-I' are presented below in Figure 11.

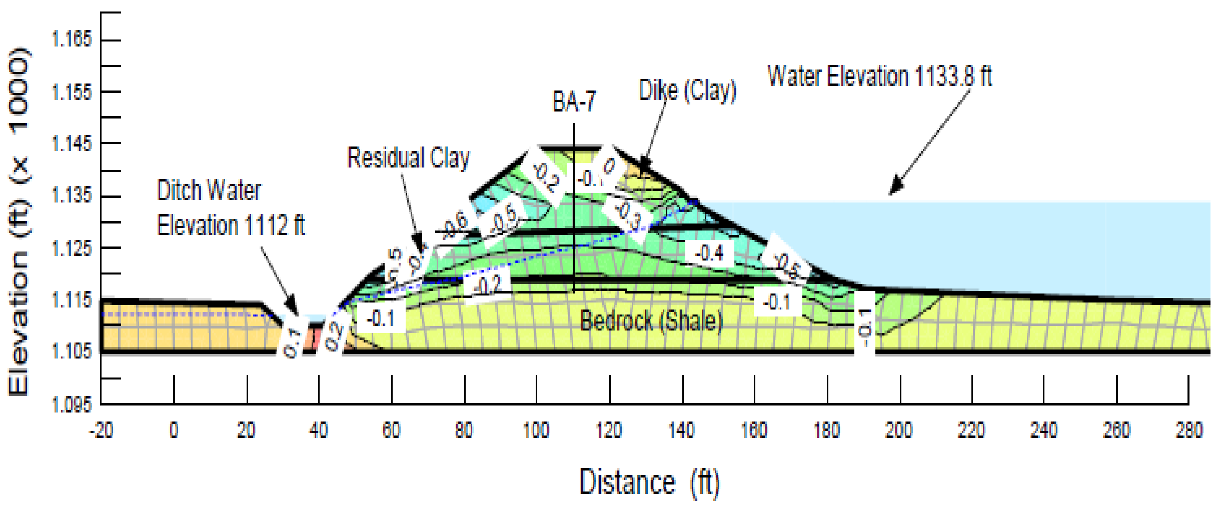


Figure 11. Cross Section I-I' (Vertical Gradient Contours)

11.3. Slope Stability Analysis

11.3.1. Background

The stability of the existing dike slopes was evaluated using two-dimensional limit equilibrium methods of analysis. For conventional, two-dimensional methods of analysis, the slide mass above an assumed failure surface is split into vertical slices and stresses are evaluated along the sides and base of each slice. The factor of safety against a slope failure (FS_{slope}) is defined as:

$$FS_{slope} = \frac{\textit{shear strength of soil}}{\textit{shear stress required for equilibrium}} \qquad \textbf{Eqn. 3}$$

where the strengths and stresses are computed along a defined failure surface, on the base of the vertical slices. The shearing resistance at locations along the potential slip surface are computed, with appropriate strength parameters (cohesion and friction angle), as a function of the total or effective normal stress.

Factors of safety against failure were calculated using Spencer's method of analysis. Spencer's method (1967) satisfies both moment equilibrium and force equilibrium, and uses the method of slices to examine inter-slice normal and shear forces. Circular and translational slip surfaces were used to identify critical surfaces. The resistance to sliding was calculated using effective stresses and shear strength parameters selected based on laboratory testing, standard penetration testing, and using phreatic line conditions obtained from piezometer readings. Slope stability analysis was performed using GeoStudio 2007 Slope/W, a software program developed for examining the stability of earth structures.

11.3.2. Cross Sections

Slope stability analysis was performed for the following cross sections. Profiles of selected cross sections are presented in Appendix B and stability output sections from Slope/W are presented in Appendix I. Typical cross sections of the different structures are presented in Figures 12 through 15

DRY FLY ASH STACK

1) A-A' (cross section through borings JS-53 to JS-57)
2) B-B' (cross section through borings JS-47 to JS-52)
3) C-C' (cross section through borings JS-43 to JS-46)
4) D-D' (cross section through borings JS-35 to JS-42)
5) E-E' (cross section through borings JS-28 to JS-34C)
6) F-F' (cross section through borings JS-23 to JS-27)
7) G-G' (cross section through borings JS-19 to JS-22)
8) H-H' (cross section through borings JS-15 to JS-18)

Bottom Ash Disposal Area No. 2

9) I-I' (cross section through boring BA-7)

Ash Disposal Area J

10) J-J' (cross section through borings JP-4)
11) K-K' (cross section through boring JP-3)
12) M-M' (cross section through boring JP-2)
13) O-O' (cross section through boring JP-1)

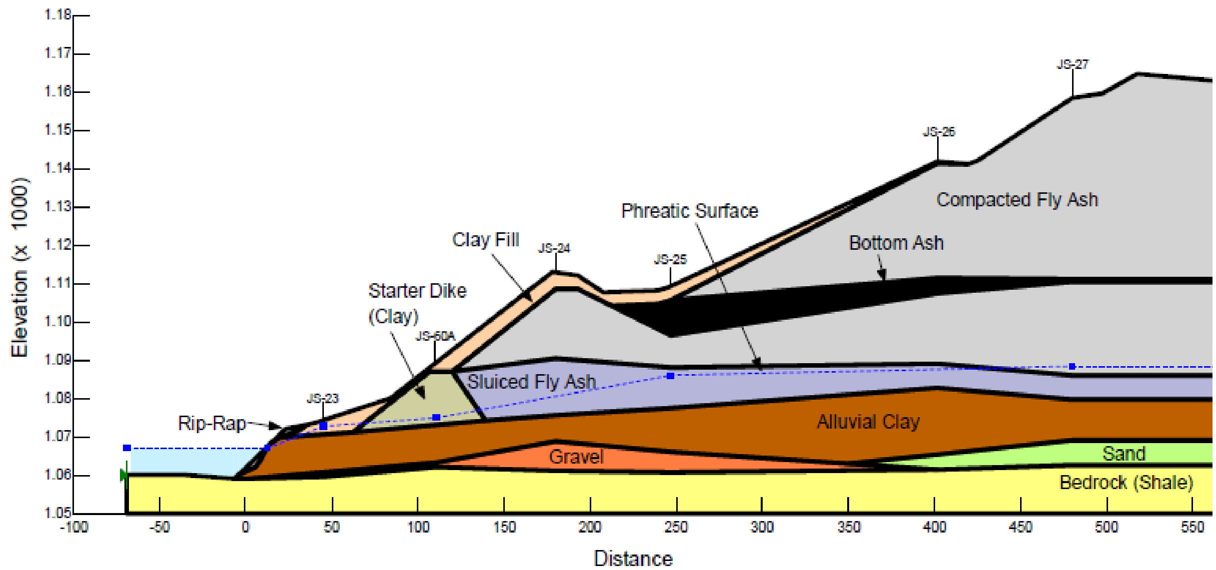


Figure 12. Typical Dry Fly Ash Stack Cross Section

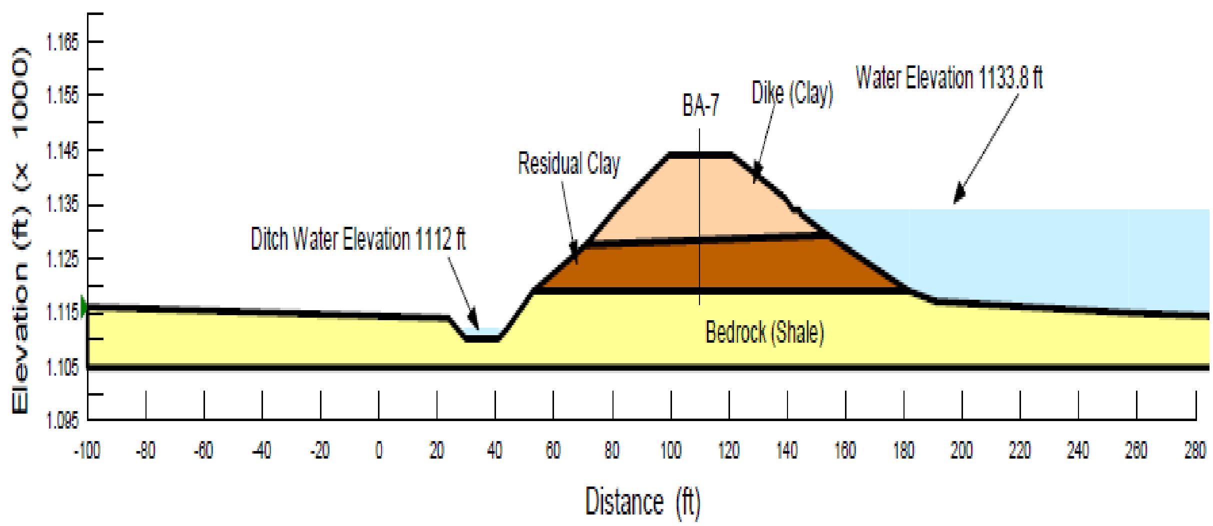


Figure 13. Typical Bottom Ash Disposal Area 2 Cross Section

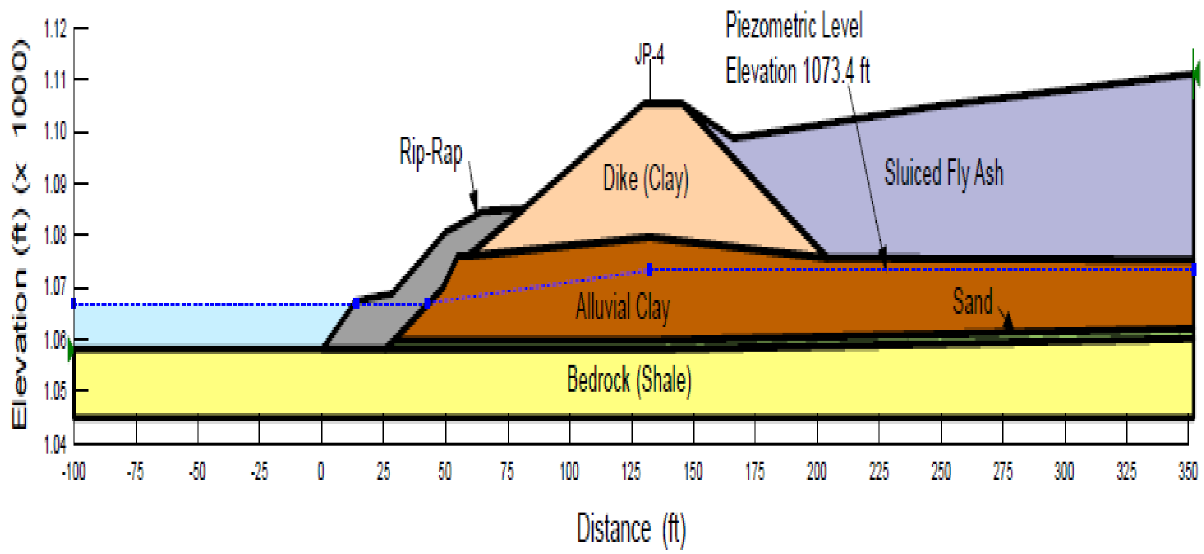


Figure 14. Typical Ash Disposal Area J Cross Section (West)

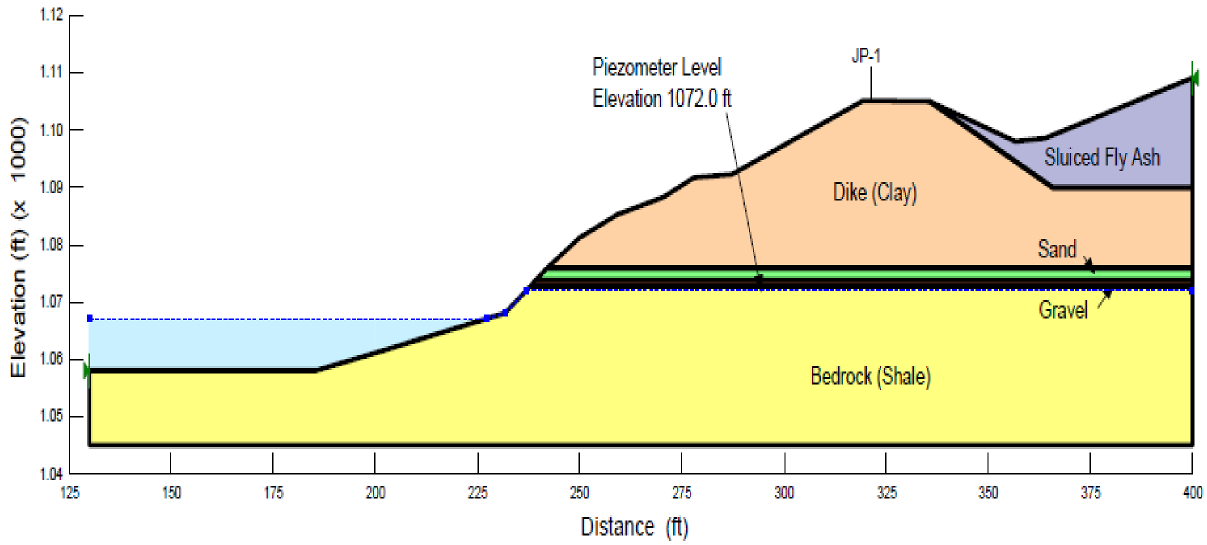


Figure 15. Typical Ash Disposal Area J Cross Section (East)

The above subsurface profiles were developed by combining the information collected from the borings advanced during this geotechnical exploration along with historical documents provided by TVA. Historical drawings provided information regarding original ground surface, original dike positioning and configuration, as well as some previous repairs. The historical drawings of the starter dikes were significantly useful in developing cross sections for the Bottom Ash Disposal Area 2 and Ash Disposal Area J dikes since these structures apparently were not expanded upward. However, this was not the case for the Dry Fly Ash Stack, where the starter dike was expanded but no related design or as-built information was available. Therefore, configuration of the Dry Fly Ash Stack Area upward dike expansion was developed based mainly on the boring information obtained as part of this exploration and assumed interpolations and extrapolations of soil horizon boundaries. Table 19 below lists historical drawings used to develop typical cross sections.

Table 19. Historical Drawings Used for Subsurface Profiles

Section	Reference Drawing	Date of Drawing	Description (Drawing used for developing or determining)
A - H	10N410 R3	4/1958	Original groundline
I	10W293-1 R2	8/1980	Original groundline
J - O	10W286-1 R3	12/1984	Original groundline
A - B & D - H	10N410 R3	4/1958	Starter dike configuration & location
C	10N290	7/1973	Starter dike configuration post 1973 failure
I	10W293-2 R1	4/1978	Starter dike configuration & location
J	10W286-4 R1	7/1985	Starter dike configuration & location
K - O	10W286-1 R3	12/1984	Starter dike configuration & location
A - B	10W206-1 R1	8/2002	Limits of placed riprap
D - F	10W206-2	3/2001	Limits of placed riprap
G & H	10W206-3	3/2001	Limits of placed riprap

11.3.3. Material Properties

Dry Fly Ash Stack

The starter dike was constructed in the late 1950’s and has exhibited its current cross-sectional geometry (slopes and crest elevation) for about 9 years since the last construction. Hence, excess pore pressures generated in the underlying soil during construction have had sufficient time to dissipate and steady state seepage conditions have developed within the dike. Additionally, 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, flattened slopes, or other geometric modifications are constructed, then undrained, total stress stability analyses will need to be performed.

Drained shear strength (S_d) of the sluiced fly ash soil was determined from effective stress strength parameters using the following equations:

$$S_d = c' + \sigma' \tan \phi' \qquad \textbf{Eqn. 4}$$

$$\sigma' = \sigma - u \qquad \textbf{Eqn. 5}$$

Where:

c’ = the effective cohesion
ϕ’ = the effective angle of internal friction
σ’ = the effective stress
σ = the total stress and
u = the pore water pressure

Uncemented or Granular Soil

Uncemented soils exhibit no strength at σ'=0, corresponding to c' = 0. In the case of unsaturated fine grained sands, suction results in apparent cohesion, but this component of strength is lost upon saturation. Over a large pressure range, most granular soils have a curved strength envelope. Fitting a straight line through segments of a curved failure envelope can result in c' > 0, but the values are applicable only over the specified range of effective stress.

Several uncemented (granular) soils were encountered during this exploration that were unable to be sampled using undisturbed methods and thus prevented triaxial testing to derive shear strength parameters. Compacted fly ash and bottom ash horizons were the predominant horizons encountered in the Dry Fly Ash Stack, while sand and gravel horizons were encountered at varying thicknesses within the foundation alluvium near the top of bedrock. These soils typically exhibited medium dense to very dense relative density (N-values ranging from 10 to 50+ blows per foot) with damp to moist moisture contents. The strength and unit weight parameters for these soil horizons were determined from published correlations between SP test blow counts (N_{60}), relative density, and effective friction angle Φ'. However, as discussed in Section 6.1 of this report, the SPT testing was performed utilizing an automatic hammer and were corrected prior to applying them in correlations with other soil index properties. The correction for hammer efficiency is a direct ratio of relative efficiencies as follows:

$$N_{60} = N_{80}\left(\frac{80}{60}\right) \qquad \textbf{Eqn. 6}$$

Stantec also corrected standardized N_{60} values resulting from SPT testing within these materials for the effect of overburden pressure prior to using the data in conjunction with correlations for non-cohesive soil parameters. The N_{60} values were normalized to vertical effective overburden stresses of 2,000 pounds per-square foot. This calculation requires an effective unit weight for each soil horizon multiplied by the depth of the soil horizon. The relationship between the correction factor, C_N, and the effective overburden stress, σ', was based on a relationship proposed by Liao and Whitman as referenced in Seed and Harder [1990]:

$$C_N = \frac{1}{\sqrt{\sigma'}} \qquad \textbf{Eqn. 7}$$

Where:

C_N	=	correction factor for overburden stress
σ'	=	vertical effective overburden stress (tsf)

Consequently, the standardized corrected N-value, $(N')_{60}$ is equal to:

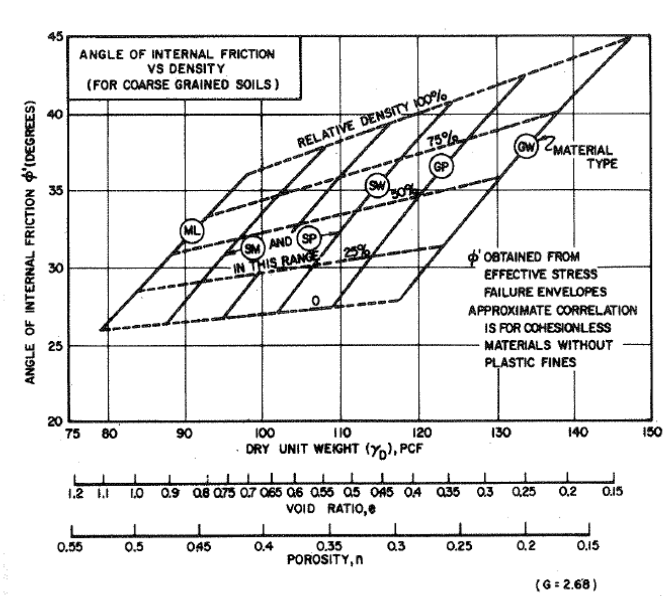
$$(N')_{60} = C_N N_{60} \qquad \textbf{Eqn. 8}$$

Where:

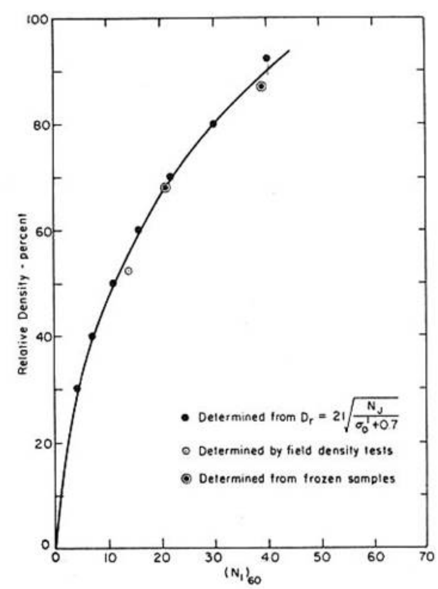
C_N = correction factor for overburden stress
$(N')_{60}$ = standardized N-value

The N-values noted on the graphical boring logs in Appendix B and typed boring logs in Appendix C are calculated based on the actual blowcount obtained in the field. They do not reflect corrections for hammer efficiency or overburden stress.

The N_{60} values were utilized to obtain relative densities based on relationships developed by Tokimatsu and Seed (1988) as shown in Figure 16 below. NAVFAC (1982) presents a relationship using relative density and specific soil types to correlate angle of internal friction, unit weight, and void ratio as shown in Figure 16 below. Soil classifications for the correlations are based on laboratory testing results and visual classifications performed by the on-site geotechnical engineer or geologist during the drilling process. Once the relationships for the angle of internal friction, unit weight, and void ratio were established, the in-situ unit weight was calculated based upon the natural moisture content.



From NAVFAC (1982)



From Tokimatsu and Seed (1988)

Figure 16. Charts used to Correlate N_{60} to ϕ'

Typical N_{60} values for the granular soils described above varied across each section. As such, the unit weight and drained friction angle of every soil horizon was estimated based upon blow counts (N-values) representative from each particular cross-section and using the 2/3rd rule. The rule implies that approximately two-thirds of the data points fall above and one-third fall below the chosen parameter. Table 20 below presents soil parameters for granular soils calculated and used for slope stability analyses.

Table 20. Material Properties for Granular Soils

Section	Compacted Fly Ash (Soil 4)		Alluvial Gravel (Soil 6)		Alluvial Sand (Soil 7)		Bottom Ash (Soil 3)	
	Φ'	UW	Φ'	UW	Φ'	UW	Φ'	UW
A	32.0	110.0	39.0	137.0	29.5	132.0	N/A	N/A
B	32.0	110.0	37.5	140.0	N/A	N/A	N/A	N/A
C	30.0	110.0	N/A	N/A	37.0	139.0	N/A	N/A
D	32.5	110.0	36.0	139.0	N/A	N/A	29.0	117.0
E	32.5	110.0	37.0	137.0	30.5	131.0	28.0	106.0
F	30.0	110.0	32.5	137.0	32.0	127.0	32.0	118.0
G	30.0	110.0	34.5	133.0	36.0	130.0	29.0	105.0
H	30.0	110.0	37.0	136.0	N/A	N/A	N/A	N/A

Clay Materials

For normally consolidated, saturated clays, the Mohr-Coulomb failure envelope exhibits c' = 0. At effective stresses below the pre-consolidation pressure, overconsolidated clays have a curved failure envelope that can be represented with a straight line having c' > 0. However, overconsolidated clays in the field are often fissured and the in situ c' is significantly smaller than values determined from testing of small samples in the laboratory. To avoid progressive failures in overconsolidated, stiff fissured clays, remolded soil samples are recommended for testing; this generally results in "fully softened" strengths with c' = 0. Thus, in the absence of particle cementation/bonding, long term (drained) shearing resistance related to c' > 0 is considered unreliable. In routine geotechnical design practice, values of c' = 0 are usually assumed for both normally and overconsolidated saturated clays, and for uncemented granular soils. Detailed testing and characterization of a particular soil, coupled with careful application of the fitted strength envelopes, are necessary where values of c' are used in a stability evaluation. For these analyses, c' = 0 was used for all soils.

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 soil parameters used for the dike, ash stack and existing foundation materials were derived using both current and historical laboratory test data (consolidated undrained triaxial tests, standard penetration testing data, and classification testing data) and Stantec's experience with these materials in similar applications.

An effective friction angle for the Clay Fill (Soil 1), Dike Clay (Soil 8) and Alluvial Clay (Soil 2) was selected based on (1) results of twenty four consolidated undrained triaxial (CU) tests, (2) results of the SPT data and (3) the plasticity index of each soil. A relationship between the plasticity index and peak friction angles for normally consolidated clays is shown in

Figure 17 (from Duncan and Wright, 2005). The unit weight for both soil horizons was selected based on density testing of consolidated undrained triaxial samples. The results of the testing can be found in Appendix F of this report.

Table 5.7 Typical Values of Peak Friction Angle (ϕ') for Normally Consolidated Clays[a]

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

Source: Data from Bjerrum and Simons (1960).
[a] $c' = 0$ for these materials.

Figure 17. Typical Values of Peak Friction Angle (Φ') for Normally Consolidated Clays

Soils 1, 2 and 8 parameters used for slope stability analysis on the Dry Fly Ash Stack are presented below in Table 21.

Table 21. Material Properties for Clay Materials found in Dry Fly Ash Stack

Material	Unit Weight (pcf)	Cohesion (c')	Friction Angle (Φ')
Clay Fill (Soil 1)	125	0	32º
Dike (Soil 8)	126	0	31º
Reconstructed Dike (Soil 8)	126	0	31º
Alluvial Clay (Soil 2)	120	0	31º

Sluiced Fly Ash

Stantec performed twelve (12) consolidated undrained triaxial tests on remolded and undisturbed samples of sluiced fly ash (Soil 5). The results are presented as Attachment 5 of Appendix F of this report. To select the representative strengths for Soil 5, the methodology outlined in the US Army Corps of Engineers Engineer Manual EM 1110-2-1902 was used as a guide. Failure stresses measured in the laboratory tests were expressed in terms of "p'-q" values, $[p' = 0.5(\sigma_1' + \sigma_3'), q = 0.5(\sigma_1' - \sigma_3')]$, then an envelope was conservatively fit through the data. The selected strength parameters represent a failure envelope where approximately two-thirds of the test data falls above the envelope. Strength parameter selection charts using "p'-q" plots are included in Appendix G.

In addition, information obtained at other TVA facilities was reviewed in selecting strength parameters for the sluiced fly ash deposits. For example, as a part of the root cause analyses of the Kingstone failure, AECOM performed 25 tri-axial compression tests with various consolidation techniques on hydraulically placed ash, and Law Engineering, Inc. completed six triaxial tests in 1995, as a part of a testing program on sluiced ash materials in

Dredge Cells I and III of the Kingstone ash disposal area. When plotting these test results on a scatter plot (see Appendix G), the resultant Φ' for the hydraulically placed ash is on the order of 25 degrees.

A friction angle (Φ') of 24 degrees was selected for the sluiced ash encountered under the Dry Fly Ash Stack. The unit weight selected for Soil 5 is 105 pounds per cubic foot.

Bottom Ash Disposal Area 2

As described in Section 10.2 of this report, two predominant soil horizons were encountered in borings drilled at this site, the dike material (Soil 1) and the foundation residual clay (Soil 10). According to historical information, it is believed that residual clay excavated from the interior of the disposal area 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 26 and 25 for the 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.

An effective friction angle for each soil was selected based on (1) results of six consolidated undrained triaxial (CU) tests performed on remolded samples, (2) results of the SPT data and (3) the plasticity index of each soil as discussed earlier in this section for the Dry Fly Ash Stack. The unit weight for both soil horizons was selected based on density testing of remolded samples. The results of the testing can be found in Appendix F of this report.

Parameters used for slope stability analysis on the Bottom Ash Disposal Area 2 are presented below in Table 22.

Table 22. Material Properties for Clays at the Bottom Ash Disposal Area 2

Material	Unit Weight (pcf)	Cohesion (c')	Friction Angle (Φ')
Dike (Soil 1)	123	0	33.0
Residual Clay (Soil 10)	121	0	33.0

Ash Disposal Area J

Two predominant clay horizons along with several granular soil horizons were encountered during drilling performed at the Ash Disposal Area J. Shear strength parameters used for slope stability analysis on the granular materials were estimated using standard penetration tests and relationships discussed earlier in this section for the Dry Fly Ash Stack. Shear strength parameters for the clay dike and alluvial clay were selected based (1) results of five consolidated undrained (CU) triaxial tests performed on remolded samples, (2) results of the SPT data and (3) the plasticity index of each soil as discussed earlier in this section for the Dry Fly Ash Stack.

The results of classification and CU testing on the Ash Disposal Area J soil samples can be found in Section 10.3 and Appendix F of this report. The plasticity index was determined to be 25 and 19, for the clay dike and alluvial clay, respectively. The unit weight for both cohesive soil horizons was selected based on density testing of undisturbed samples. No borings were advanced inside the dike limits and therefore parameters used for the sluiced fly ash were taken from testing and assumptions made for sluiced ash found at the Dry Fly Ash Stack. Parameters used for slope stability analysis on the Ash Disposal Area J are presented below in Table 23.

Table 23. Material Properties at the Ash Disposal Area J

Material	Unit Weight (pcf)	Cohesion (c')	Friction Angle (Φ')
Dike (Soil 1)	124	0	30.0
Alluvial Clay (Soil 2)	127	0	31.0
Sluiced Ash (Soil 5)	105	0	24.0
Alluvial Sand (Soil7)	118	0	30.0
Alluvial Gravel (Soil 6)	132	0	37.5

11.3.4. Failure Search Modes

The following failure modes were analyzed for all the cross sections.

X) Grid & Radius (circular failure forced through two points)
Y) Translational (non-circular failure forced through three points)
Z) Entry/Exit (circular failure forced through two points)

11.3.5. Phreatic Lines

Laboratory analyses provide effective strength parameters which are best utilized in conjunction with pore water pressure to determine the most accurate critical slip surfaces. Pore water pressure was simulated during slope stability analysis using data collected from piezometers positioned in line with their corresponding cross sections to develop each phreatic line. The phreatic line location in the analyses of the Dry Stack Area and Ash Disposal Area J, for all the cross sections and failure modes, was selected using the highest levels water levels recorded from piezometer readings. The lower end of the phreatic line was connected to the following river pool elevations.

a. Existing Pool (river pool elevation 1067 feet)

b. High Pool (river pool elevation 1073 feet, considered normal high pool elevation)

The existing river pool elevation of 1067 feet was obtained based on observation made throughout the exploratory fieldwork. The high river pool elevation of 1073 feet was assumed to be the normal pool elevation as indicated in the historical drawings. Table 24 lists piezometer data used to determine phreatic conditions for the different cross sections.

Table 24. Summary of Piezometer Information

Cross Section	Piezometer	Tip Elevation (ft)	Highest PZ Reading (ft)	Date of Highest PZ Reading (ft)
A-A’	JS-53	1068.0	1077.23	6/29/09
	JS-55	1080.4	1086.66	5/19/09
	JS-56	1074.0	1080.85	5/19/09
	JS-57	1081.8	1088.95	8/13/09
B-B’	JS-47	1063.8	1075.38	6/3/09
	JS-63B	1062.7	1075.85	10/13/09
	JS-49	1073.3	1081.96	6/3/09
	JS-50	1076.7	1087.89	6/29/09
	JS-52	1091.8	1105.03	6/29/09
C-C’	JS-43	1058.7	1076.20	6/29/09
	JS-45	1076.8	1091.73	6/29/09
D-D’	JS-35	1057.4	1076.69	6/3/09
	JS-37	1079.8	1090.35	6/3/09
	JS-39	1088.6	1098.12	6/17/09
	JS-42	1091.7	1105.74	6/3/09
E-E’	JS-28	1057.7	1077.25	5/19/09
	JS-61A	1060.3	1077.91	10/13/09
	JS-30	1075.6	1087.14	5/21/09
	JS-32	1084.6	1089.90	6/17/09
	JS-34C	1098.9	1110.15	10/13/09
F-F’	JS-23	1059.1	1072.78	6/30/09
	JS-60B	1062.0	1075.40	10/13/09
	JS-25	1068.1	1085.99	8/13/09
	JS-27	1078.3	1088.24	6/17/09
G-G’	JS-19	1057.8	1072.93	5/19/09
	JS-21	1066.0	1079.33	6/29/09
	JS-22	1060.4	1085.11	6/3/09
H-H’	JS-15	1059.4	1071.77	6/3/09
	JS-17	1061.5	1074.20	6/3/09
	JS-18	1070.2	1090.43	6/3/09
I-I’	BA-8	1110.7	1126.48	8/13/09
J-J’	JP-4	1059.6	1073.37	6/3/09
K-K’	JP-3	1070.9	1072.00	6/17/09
M-M’	JP-3	1070.9	1072.00	6/17/09
O-O’	JP-3	1070.9	1072.00	6/17/09

11.3.6. Results of Stability Analyses for Existing Conditions

All cross sections were first analyzed for existing conditions. The analyses for the Dry Stack and Ash Disposal Area J cross sections were performed assuming two river pool elevations as described before. Where the analyses did not result in acceptable factors of safety, the cross sections were analyzed further assuming certain corrective measures would be implemented, as discussed in the following section. Multiple search types were used to determine the lowest factor of safety at each failure location. Failure surfaces were constrained to a minimum depth of 10 feet.

Results of slope stability analyses for existing conditions assuming high pool and existing pool conditions are presented in Table 25. Drawings of the stability analysis are presented in Appendix I.

Slope Geometry	Search Type	High Pool Factor of Safety*	Existing Pool Factory of Safety*	Failure Location
Existing Conditions (as of 7-28-09) Section A-A’	Grid & Rad	1.9	1.9	Below Lower Road
	Entry & Exit	--	1.9	Below Upper Road
Existing Conditions (as of 7-28-09) Section B-B’	Grid & Rad	1.5	1.3	Below Lower Road
	Entry & Exit	--	2.0	Below Upper Road
Existing Conditions (as of 7-28-09) Section C-C’	Grid & Rad	1.5	1.3	Below Lower Road
	Entry & Exit	--	1.7	Below Upper Road
Existing Conditions (as of 7-28-09) Section D-D’	Grid & Rad	1.5	1.4	Below Lower Road
	Entry & Exit	--	1.6	Below Upper Road
Existing Conditions (as of 7-28-09) Section E-E’	Grid & Rad	1.7	1.4	Below Lower Road
	Entry & Exit	--	1.7	Below Upper Road
Existing Conditions (as of 7-28-09) Section F-F’	Grid & Rad	1.7	1.5	Below Lower Road
	Entry & Exit	--	1.7	Below Upper Road
Existing Conditions (as of 7-28-09) Section G-G’	Grid & Rad	2.0	1.6	Below Lower Road
	Entry & Exit	--	1.8	Below Upper Road
Existing Conditions (as of 3-19-09) Section H-H’	Grid & Rad	1.5	1.5	Below Lower Road
	Entry & Exit	--	2.0	Below Upper Road
Existing Conditions (as of 3-19-09) Section I-I’	Grid & Rad	--	1.5	Clay Dike Embankment
Existing Conditions (as of 10-16-09) Section J-J’	Grid & Rad	1.6	1.6	Riprap & Alluvial Clay
Existing Conditions (as of 10-16-09) Section K-K’	Grid & Rad	1.5	1.5	Clay Toe Dike Embankment
Existing Conditions (as of 10-16-09) Section M-M’	Grid & Rad	1.3	1.3	Clay Dike Embankment
Existing Conditions (as of 10-16-09) Section O-O’	Grid & Rad	1.7	1.7	Clay Dike Embankment

* The US Army Corps of Engineers Engineering Manual EM 1110-2-1902, “Slope Stability” recommends a target minimum factor of safety of 1.5 for long term embankment slope stability.

Stability analysis of existing conditions along sections B-B’, C-C’, D-D’, and E-E’ within the Dry Fly Ash Stack produced factors of safety less that the 1.5 target for slip planes located within the river bank, immediately below the toe of the starter dike. These sections all produced a factor of safety above 1.5 for failure surfaces between the lower (toe of starter dike) and upper perimeter roads. These slips were typically deep seated failures produced by the search type, Entry & Exit. Stability analysis for the Ash Disposal Area J produced factors of safety less than 1.5 for the existing and high pool conditions for section M-M’.

11.4. Results of Slope Stability Analyses for Conditions after Recommended Improvements are implemented

Where the analyses of existing conditions did not result in acceptable factors of safety, the cross sections were analyzed further assuming certain corrective measures would be implemented. In the case of the Dry Fly Ash Stack, the selected corrective measures were a toe sub-drain and placement of additional riprap on the river bank. The corrective measures selected for the Ash Disposal Area J was a buttress or rock berm to protect the toe of the dike.

Slope stability analyses of conditions after recommended improvements are implemented were performed for cross sections B-B’, C-C’, D-D’, E-E’, and M-M’. Typical profiles of each section are located in Appendix I.

Further discussion relative to implementation of corrective measures is presented in Section 13, Conclusions and Recommendations. Drawings of additional slope stability analysis are presented in Appendix I. Tables 26 and 27 present the results of stability runs which include the addition of the sub-drain system, riprap and rock buttress mentioned above.

Table 25. Results of Stability Analyses after Corrective Measures are Applied to Dry Fly Ash Stack

Slope Geometry	Search Type	Sub-Drain System	Additional Riprap	Factor of Safety High Pool	Factor of Safety Existing Pool	Failure Location
Existing Conditions (as of 7-28-09) Section B-B’	Grid & Rad	Yes	No	--	1.4	Below Lower Road
	Grid & Rad	Yes	Yes (2.5:1 w/ 5ft bench)	1.8	1.6	Below Lower Road
Existing Conditions (as of 7-28-09) Section C-C’	Grid & Rad	Yes	No	--	1.3	Below Lower Road
	Grid & Rad	Yes	Yes (2.5:1 w/ 5ft bench)	1.7	1.6	Below Lower Road
Existing Conditions (as of 7-28-09) Section D-D’	Grid & Rad	Yes	No	--	1.4	Below Lower Road
	Grid & Rad	Yes	Yes (2.5:1)	1.7	1.6	Below Lower Road
Existing Conditions (as of 7-28-09) Section E-E’	Grid & Rad	Yes	No	--	1.5	Below Lower Road

Table 26. Results of Stability Analyses after Corrective Measures are Applied to Ash Disposal Area J

Slope Geometry	Search Type	Rock Buttress Bench Width	Rock Buttress Grade	Factor of Safety High Pool	Factor of Safety Existing Pool	Failure Location
Existing Conditions (as of 10-16-09) Section M-M'	Grid & Rad	10 feet	2:1	1.5	1.6	Embankment
			2.5:1	1.5	1.6	Embankment
	Grid & Rad	12.5 feet	2:1	1.5	1.6	Embankment

12. Repair and Maintenance Work Completed in 2009

Stantec prepared three work plans to address certain conditions that needed the implementation of repair and maintenance measures. The first work plan, issued May 7, 2009, included the removal of woody vegetation from interior and exterior slopes of the Stilling Pond West, southwest exterior slope of the dry stack, west edge of Bottom Ash Pond Area No. 2 and north and west rim of the coal yard area. As an extension to this work plan, TVA also removed woody vegetation from exterior slopes of the Bottom Ash Pond Area 2, Ash Disposal Area J and Sediment Pond West. The work plan also addressed treatment of animal burrows found on the slopes of the dry stack and the Bottom Ash Pond Area No. 2, protection against wave action along the south side of the Bottom Ash Pond Area No. 2 stilling basin and general slope grading of the northwest side of the Chemical Pond and south side of the Coal Yard Runoff Pond.

The second work plan was issued May 27, 2009 to address recommended measures to protect an exposed pipe along the south side of the Coal Yard Runoff Ponds. The third work plan was issued June 5, 2007 to perform several repair and improvement measures to the interior of the Coal Yard Runoff Ponds. All the construction or maintenance measures included in the work plans mentioned above have been implemented.

13. Conclusions and Recommendations

13.1. Dry Fly Ash Stack Area

13.1.1. Historical Information

The Dry Fly Ash Stack area was originally developed as wet ash disposal area located on the floodplain of the Holston River. The principal feature of the disposal area was a 17-foot tall (approximate height), 4,375-foot long earthen dike constructed along the south flank of the river. A historical drawing (Drawing 10N410, labeled 'Record Drawing as Constructed' and dated 1-24-1956) shows the top of dike elevation as 1087 feet±. The disposal area was subdivided for operational purposes into several areas labeled Areas A through I, with the different areas presumably separated by divider dikes.

Drawing 10N410 also shows a future expansion of the dike as depicted in Figure 4 of this report, which would have raised the dike to elevation 1110 feet. However, it is unclear what plans, if any, were followed for this purpose. The next historical drawing available (Drawing

10N410, labeled Ash Disposal Area E Dike Repair and dated 7-26-1973) shows that at least in Area E, material was placed over the starter dike and well above elevation 1110 feet (see Figure 5) following no apparent well defined slope configuration. Based on Figure 5 it appears material placement extended onto the adjacent river bank and the sluiced ash level reached an elevation above 1100 feet.

As summarized in Table 2, there were several areas where the dike slope was disturbed by sloughing, sliding, cracking and erosion. Two of these events appear to have been of more significance in terms of the extent of the work required to repair the disturbance: (1) The 1973 dike failure in Area E and (2) the 1999-2001 instability of the dike face below elevation 1110 feet. In both cases, the repair work consisted in removing material placed over the starter dike slope and grading the dike slope close to the original design slope (3:1). In addition, there appears to have been several efforts to stabilize the river bank area immediately below the toe of the dike by placing riprap over it.

13.1.2. Subsurface Conditions and Slope Stability Analyses

Based on the historical information and the general layout of the dry stack, the main focus of the geotechnical exploration was directed to the lower portion (below elevation 1110 feet) of the dry stack north slope. The most unusual subsurface conditions were encountered along cross section D-D'. In Boring JS-36, advanced near the crest of the slope, the top 6 feet consist of clay deposits which are underlain by 7.5 feet of dense fly ash. A thick horizon of sluiced fly ash was encountered below the dense fly ash from a depth of 13.5 feet (elevation 1095 feet) down to 38.1 feet. The sluiced ash was found on top of soft alluvial deposits. Similar deposits of sluiced fly ash were encountered in Borings JS-37X and JS-38, which were drilled directly uphill of Boring JS-36. This information confirmed that wet fly ash was stored to an elevation well above the top of the starter dike (1087 feet), implying the dike had to be expanded upward to provide containment. Since no reliable historical information is available relative to the vertical expansion of the dike, additional subsurface exploration was conducted along the face of the slope. The additional exploration (Borings JS-60 through JS-65) revealed the presence of clay deposits in front of the sluiced ash, above and below elevation 1087 feet.

Potential less than acceptable stability conditions appear to exist along the toe of the slope where high phreatic levels and steep river bank slopes were encountered. Historical information tends to confirm this assessment. There is a sub-drain system along the east portion of the slope that collects drainage from specific pipe penetrations as well as some toe of slope seepage. Wet areas have been observed along the perimeter road bordering the toe of the slope, both within and outside the area covered by this sub-drain system. Likewise, the historical information documents attempts to stabilize the river bank below the toe of slope using riprap.

As discussed in earlier sections of this report, the degree of stability of the toe of the slope and adjacent river bank area is highly dependant on the river pool elevation, which is known to fluctuate significantly. When the river pool elevation is at 1073 feet, the pool provides toe support and the corresponding factors of safety remain at or above 1.5 in all critical sections. When the river level drops, as was the case this past summer, the toe support is reduced significantly and the factor of safety drops accordingly.

After reviewing different corrective measures, Stantec selected two construction measures to address high phreatic levels encountered at the toe of the slope and steep river bank conditions. One measure consists of installing an under-drain system along the toe of the slope, constructed under the lower perimeter road. Although the under-drain by itself would not raise the factors of safety to acceptable levels, it would control seepage emerging along the toe of the slope and the potential associated piping. In addition, and due to environmental reasons, the water collected by the under-drain will be pumped to the coal yard drainage pond where it will be treated as needed. The second measure consists of placing riprap over the river bank to add toe resistance and attain acceptable long term factors of safety. These measures are discussed in more detailed in a later section of this report.

As stated previously, the main focus of the geotechnical exploration was directed to the lower portion (below elevation 1110 feet) of the dry stack north slope. It is recommended that an appropriate geotechnical evaluation be preformed in conjunction with future built out or closure of the dry stack.

13.2. Bottom Ash Disposal Area 2

13.2.1. Historical Information

This 40-acre structure, in operation since 1979, receives sluiced bottom ash, fly ash (intermittently) and discharges from the Coal Yard Runoff Pond and Chemical Treatment Pond. A stilling pond is located in the west end of the area, separated from the rest of the structure by an internal dike. The structure was formed by constructing an 8,600-foot long earthen dike, measuring approximately 20 feet in height and with a 16-foot wide crest.

Historical information reports the presence of isolated areas where seeps, wetness and soft ground were observed along the exterior slope of the dike. No cases of sliding, sloughing or slumping have been reported.

13.2.2. Subsurface Conditions and Stability Analyses

It appears the dike was constructed using clayey soil excavated from the pool area and adjacent areas outside the dike. The dike and foundation material found in the different borings has a medium stiff to hard consistency based on the results of the standard penetration testing. Accordingly, the stability analyses performed along a cross section (Section I-I') where the slope of the dike is steeper than in most areas has an acceptable factor of safety for long term loading conditions.

13.3. Ash Disposal Area J

13.3.1. Historical Information

The construction of this 22-acre structure was completed in 1982 and thereafter it started receiving sluiced fly ash. In 1984, the west dike of the structure was modified by using a flatter slope and riprap was placed along 700 feet of shoreline next to the west end of the north dike. This last corrective measure was apparently implemented after a narrow tree area between the toe of the dike and steep river bank slumped into the river.

13.3.2. Subsurface Conditions and Stability Analyses

The dikes forming Ash Disposal Area J were apparently constructed with clayey soil excavated from within the pool area and a borrow site located southeast of the disposal area. The consistency of the dike and foundation materials is uniform, ranging from medium stiff to hard, with the exception of a depth interval encountered deep within Boring JP-04 where the foundation soil, probably alluvial material, was found to be very soft. This boring is located above the river bank area repaired as discussed in the previous paragraph.

A review of the events that preceded the 1984 repair of the shoreline suggests that similar conditions may potentially develop along other areas of the shoreline, as demonstrated by Sections K-K', M-M' and O-O'. Even though the stability analyses show that a less than acceptable long term factor of safety against deep failure only occurs at Section M-M', the factors of safety against shallow or maintenance type of failure is less than acceptable in Sections K-K' and O-O'. If the steep river bank is not stabilized, it is possible the tree area below the dike may slump into the river, which could potentially undermine the toe of the dike.

While the stability of dike slope areas represented by Section M-M' can be improved by flattening the slope, the toe of the dike slope still needs to be protected by stabilizing the river bank. A recommended method to stabilize the river bank is discussed in the next section of this report.

13.4. Slope Stability Improvement Measures

13.4.1. Dry Fly Ash Stack Area

At TVA's request, Stantec has started preparing work plans and recommendations to improve the stability of the north slope of the dry stack below elevation 1110 feet. The work plans include two main components: (1) an under-drain along the west two thirds of the stack and (2) re-grading the slope area located west of the ramp connecting the two lower perimeter roads. Additionally, the engineering analyses included the stability analysis of the river bank area below the toe of the slope after riprap is added to achieve an acceptable factor of safety for long term loading conditions.

The under-drain will be constructed along the lower perimeter road by excavating a 5-foot deep trench, lining the bottom and uphill side of the trench with a filter consisting of sand and crushed stone and placing a perforated pipe on crushed stone bedding. The rest of the trench will be backfilled with crushed stone and capped with a layer of clayey soil and a surfacing layer of crushed stone. Water collected by the under-drain will be directed to three manholes. Pumps installed within the manholes will pump the water through 3" diameter pipes to discharge the water into the chemical pond located next to the coal yard.

The re-grading of the slope area west of the ramp connecting the two perimeter roads will consist of flattening the slope slightly with the intent to remove humps and bulges and provide a uniform surface to facilitate its maintenance. The re-grading may require offsetting slightly the upper perimeter road toward the dry stack.

Although the work plans currently in preparation do not include placing riprap to improve the stability of the river bank, the stability analyses indicates that using relatively thin layers of riprap is the most practical way to achieve an acceptable factor of safety for long term loading conditions. A typical geometrical configuration of the rock berm, as derived from the stability analyses of individual cross sections to achieve this goal, is presented in Appendix B. These geometrical configurations can be used as a basis to design more uniform cross sections of the riprap layers in terms of access and constructability. Since new riprap would be placed on top of existing riprap, the only preparatory measures would consist of some clearing and grubbing. A permit from the regulatory agencies will more than likely be required as the proposed work would encroach the floodway of the Holston River.

13.4.2. Ash Disposal Area J

As described before, years of river flow scouring have exposed the top of the bedrock along the south bank of the river, immediately below all but about 700 feet of the Ash Disposal Area J north dike slope. The scouring has left a near vertical slope next to an area moderately vegetated with mature trees. In the past, a similar condition on the west side of the North Slope developed into a slump of the tree area toward the river, apparently compromising the stability of the dike.

It is recommended that a rock berm be constructed along the river bank to protect the tree area and thereby the toe of the dike. The use of a rock berm is needed in some areas to provide an acceptable factor of safety for long term loading conditions. The typical rock berm configuration needed, based on the stability analysis of section M-M' is presented in Appendix B. These geometrical configurations can be used as a basis to design more uniform cross sections of the rock berm in terms of access and constructability.

There are other options TVA can consider to attain long term stability of the north slope of this facility if constructing a rock berm on the river bank is to be avoided. The selection and design of other alternatives would probably require that geotechnical information be obtained along the toe of the North Slope.

13.5. Monitoring and Attaining Long Term Stability of Dike Slopes below Dry Fly Ash Stack Area

As explained earlier, there are historical drawings showing the starter dike configuration and its top elevation being at 1087 feet. Borings advanced during this geotechnical exploration from approximately this elevation (see logs of Borings JS-60 through JS-65) confirmed the presence of clay deposits where the starter dike would have been constructed. Borings advanced from above elevation 1087 feet (up to elevation 1110 feet) also encountered clay deposits, though much thinner, apparently placed above the starter dike; however, no historical information is available relative to the design configuration or construction of the starter dike upward expansion. Therefore, cross sections of the actual dike expansion could only be developed using the boring information, the outline of the starter dike as shown in historical drawings and assumed interpolation and/or extrapolation lines representing horizon boundaries. The configuration of the starter dike expansion is critical in evaluating the stability of the slopes, because both the starter dike and its expansion are barriers holding behind thick deposits of sluiced fly ash. The sluiced ash deposits are in turn the foundation layer supporting most of the tall dry ash stack present at the site.

An understanding of how the different cross section profiles were prepared is important in formulating measures to monitor and attain long term stability of the slopes located below the dry stack (below elevation 1110 feet). Because the engineering analyses reported herein are based on certain assumptions (as described above) and the limited information exploratory borings provide, it is recommended that the stability of these slopes be evaluated periodically through a rigorous instrumentation monitoring program. Depending on the results of the periodic evaluations and further analyses of corrective measures to attain long term stability of the Dry Fly Ash Stack, it is possible and it should be expected that additional geotechnical work, including installing more instrumentation, will need to be performed.

14. 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 project sites. 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 design plans or construction records indicating the methods used to construct the upward expansion of the starter dike forming the lower north and east slopes of the Dry Fly Ash Stack were not available for review. As a result, it should be understood that some generalizations and assumptions were made in preparing cross section profiles prior to performing the engineering analyses.

The scope of this evaluation was limited to consider only the potential risks to the facilities due to excessive seepage and slope instability under long-term, steady-state seepage loading conditions. This assessment did not consider potential failure modes related to spillway capacity and overtopping or seepage along penetrations through the embankment (including the buried spillway pipes).

15. References

The following is a list of documents referenced in this report and/or used to evaluate the stability of the structures at John Sevier Fossil Plant:

Soil Strength and Slope Stability, pp 49, Duncan, J. Michael, Wright, Stephen G., 2005.

Slope Stability, Department of the Army, US Army Corps of Engineers, Engineering Manual EM 1110-2-1902, October 31, 2003.

Geotechnical Investigations, Department of the Army, US Army Corps of Engineers, Engineering Manual EM 1110-1-1804, January 1, 2001.

Seepage Analysis and Control for Dams CH 1, Department of the Army, US Army Corps of Engineers, Engineering Manual EM 1110-1-1901, April 30, 1993.

Evaluation of settlements in sands due to earthquake shaking, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, August, pp. 861-878. Tokimatsu, K., and Seed, H. B. (1987).

Soil Mechanic Design Manual 7.1, Department of the Navy – Navy Facilities Engineering Command, May 1982.

A Method of Analysis of Embankments assuming Parallel Interslice Forces, Geotechnique, Vol. 17 (1), pp. 11-26, Spencer, E. (1967).

Appendix A

Historical Documents

Appendix B

Geotechnical Drawings

Appendix C

Boring Logs

Appendix D

Instrumentation Logs

Appendix E

Slope Monitoring Data

Appendix F

Laboratory Testing Results

Appendix G

SPT Correlation Tables and Mohr Plots

Appendix H

CPT and Vane Shear Testing

Appendix I

Engineering Analysis Results