Report of Phase 2 Geotechnical Exploration

Gypsum Stack
Widows Creek Fossil Plant
Stevenson, Alabama

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Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

February 5, 2010

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February 5, 2010 rpt_003_175569039

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

Re: Report of Phase 2 Geotechnical Exploration
(Original Date January 14, 2010, Revised February 5, 2010)
Gypsum Stack
Widows Creek Fossil Plant
Stevenson, Alabama

Dear Mr. Snider:

Stantec Consulting Services Inc. (Stantec) has completed a Geotechnical Exploration for the Gypsum Stack at the Widows Creek Fossil Plant. A report of the geotechnical exploration was issued January 14, 2010. The original report has been revised to remove recommendations or statements related to operational or procedural matters not directly related to the geotechnical engineering ananlyses of the gypsum stack. Specifically, Recommendations 12.2, 12.3, 12.4 and 12.10 and the last paragraph of Recommendation 12.5 were removed from the original report. These matters are being directly addressed by TVA.

This report documents the subsurface conditions encountered during the exploration, results of laboratory testing, findings from the historical document reviews, results of our analyses and evaluation, and recommendations for the facility. These services were performed under Engineering Service Request ESR/TAO 909 in accordance with the terms and provisions established in our System-Wide Services Agreement dated December 22, 2008.

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

Sincerely,

STANTEC CONSULTING SERVICES INC.

Robert D. Fuller

Robert D. Fuller, PE
Project Manager

/rdr

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Appendix

Executive Summary

The purpose of this study was to evaluate the current stability of the Gypsum Stack at Tennessee Valley Authority's Widows Creek Fossil Plant. This document is the first report in a two part series which will address the geotechnical issues of stability and seepage identified during the Phase 1 Facility Assessment. Using the information provided in the Phase 1 report, TVA identified the Gypsum Stack as a High Hazard Potential structure in accordance with federal guidelines for dam hazard classifications based on the consequences of failure. Given the hazard classification and unplanned Pond 2B discharge on January, 2009, TVA is in the process of converting the Widows Creek Plant to a dry stack process. At this time, it is anticipated the Gypsum Stack will be closed in association with the dry stack conversion. The following assessment of the Gypsum Stack and associated recommendations are described below.

The existing subsurface conditions of Gypsum Stack was investigated during a subsurface exploration and laboratory testing program, and the dike slopes were assessed for slope stability under static, long-term, steady-state conditions. Seepage and slope stability were evaluated using engineering analyses to quantify factors of safety. While drilling and laboratory testing was being completed, the results of a preliminary slope stability analysis of the existing stack and the observations made during our site inspections initiated recommendations for immediate risk reduction measures which TVA implemented over the past few months. This year's completed improvements include:

- Eliminating inactive pipe penetrations through the stack was completed on June, 2009. One of the grouted spillways caused the loss of pool event in January, 2009.
- Construction of a toe stability berm using a graded filter of sand and crushed stone, and slope flattening along a section on the west side of the stack to improve slope stability was completed in August, 2009. The toe berm featured a seepage collection toe drain with observation wells for monitoring the quantity of flow and turbidity of seepage.
- Two active spillway conduits from the stack were extended into the stilling pond. The new buried pipeline systems promote inspection of side slopes, maintenance mowing and expandability of the stack while safely conveying the plant's process water from the top of the stack to the stilling pond.
- On top of the stack, the operating freeboard was increased to more than three feet and the crest width of the perimeter dike was also increased to meet TVA's minimum operational criteria.
- With the installation of piezometers and slope inclinometers, a monthly monitoring program has been established.
- Assignment of supervisors who focus on maintenance and construction. This has resulted in increased frequency of woody vegetation clearing, maintenance of access roads and mowing to improve the ability to inspect and monitor.
- Dam safety training for TVA plant and police personnel and others who will be on the project.

Scope of Geotechnical Exploration

The investigation began with a review of available geologic and historical project information provided by TVA. A site inspection followed to determine the general field conditions of the stack for comparison to the available design drawings. Based on the inspection results, a geotechnical boring plan was developed to evaluate the subsurface conditions for potential problem areas around the facility. The geotechnical evaluation consisted of following: two (2) rock soundings, four (4) sample and core borings, eighteen (18) sample borings, two (2) vane shear borings, and four (4) cone penetration test borings. The instrumentation program consisted of installing fifteen (15) piezometer and three (3) slope inclinometers to monitor the existing slope conditions and subsurface water elevations at the stack.

Results of Exploration

The data collected from the geotechnical exploration was supplemented with subsurface information obtained by Ardaman and Associates, Inc. during 1981, 1990 and 2004 field explorations. Logs from these previous reports, plus laboratory test data provided additional information on the gypsum-fly ash and native soil materials. AECOM's conclusions regarding the mechanisms that contributed to the Kingston dredge cell failure were also considered in the investigation and analysis of the Gypsum Stack.

The engineering analyses focused on five cross sections (A,D,H,F,K) selected to represent the typical slope conditions of the stack considering variations in the perimeter dike and natural ground geometries. The cross sectional geometry, including the thickness and depth of various soil layers, was estimated using data from the site exploration program, the historical project drawings, and other project documentation.

The exploration encountered native foundation soils consisting of clays and silty clays and confirmed that these soils types were utilized to construct the initial starter dikes. It was also confirmed that material impounded behind the initial dike is co-mingled gypsum and fly ash. The relative density of the byproduct materials, as determined by standard penetration testing, varies significantly from boring to boring and within different intervals regardless of depth.

Stability Analysis Results for As Found Stack Height

The slope stability calculations produced factors of safety against sliding along various potential failure mechanisms. Current USACE criteria for the long-term stability require a factor of safety for slope stability of at least 1.5. The slope stability results show that the Gypsum Stack meets this criterion. Safety factors range from a low value of 1.5 (at Stability Section H) to a high value of 2.3 (at Stability Section D). In general, the lower safety factor was calculated along the northwest side of the Gypsum Stack.

Stability Analysis Results for Final Stack Height

The results from a five-year build out (time anticipated to switch to a dry stack landfill) were also reviewed for long term steady state seepage conditions, assuming that seepage control drains are provided at the toe of the stack and at each stability bench, as the stack height increases. The anticipated final stack height was estimated using reported slurry production rates and design drawings provided by TVA. The factors of safety for the five year build out ranged from a low of 1.3 to a high of 1.6. The lowest factors of safety at the planned final stack height are found along the western perimeter of the Gypsum Stack where factors of safety 1.30 were computed. As directed by TVA, mitigation plans are now underway to increase the long-term factors of safety against sliding to achieve the target minimum value as the stack height increases. The mitigation plan will incorporate specific interim risk reduction strategies and include both enhanced geotechnical instrumentation and construction of a dike embankment buttress, storm water and seepage control systems.

In conclusion, the gypsum stack at the planned final height does not exhibit acceptable factors of safety for long-term global stability. This does not imply that the dike is in immediate danger of failure, but TVA should undertake efforts to improve the safety of this facility in association with planned dry ash conversion process following the conclusions and recommendations presented herein.

The assessment and recommendations presented herein addresses the requirements identified in a draft Alabama Department of Environmental Management (ADEM) Consent Order dated June 26, 2009.

Report of Phase 2 Geotechnical Exploration

Gypsum Stack
Widows Creek Fossil Plant
Stevenson, Alabama

1. Introduction

1.1. Facilities Assessment Project

The Tennessee Valley Authority (TVA) requested that Stantec Consulting Services Inc. (Stantec) perform coal combustion by-product disposal facility assessments at 11 active fossil plants and one closed fossil plant near the Watts Bar Nuclear Power plant. These facilities are located in the states of Kentucky, Tennessee and Alabama. The purpose of the assessments was to observe the coal combustion product (CCP) disposal facilities at each site and report visible signs of distress that needed immediate attention or an engineering evaluation. Stantec's scope of work was developed at the direction of TVA and within the framework of current dam safety regulations. Stantec's scope of work for facility assessments is divided into four phases described briefly as follows:

- Phase 1A – Review most Recent TVA Inspection Reports, Observe Critical Disposal Features at Sites Accompanied by TVA Personnel, Develop a List of Primary Concerns and Recommend Immediate Action or Engineering Evaluation as Considered Necessary.

- Phase 1B – Review Available Historical Documentation for Sites and Facilities, 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 Investigation.

- Phase 2 – Compare TVA Facilities to Current Dam Safety Criteria in the Appropriate State where the Plant is Located, Conduct Geotechnical Investigations and Engineering Analyses at Sites Recommended in Phase 1B as well as Complete Conceptual Repair Designs and Budget Level Costs Estimates.

- Phase 3 – Design of Repairs of Sites Recommended in Phase 2, Plans and Specifications for Construction as well as Permit/Planning Documents.

- Phase 4 – Dam Safety Training for TVA Staff.

Following Phase 1A observations, Stantec compiled a list of priority concerns and recommendations for several facilities. Based on these observations, facilities were classified as Tier 1 requiring Phase 2 assessment. The active Wet FGD (Flue Gas De-sulfurization) Stacking Area at Widows Creek Fossil Plant (WCF) was classified as a Tier 1 facility.

In January, 2009, TVA requested that Stantec develop a geotechnical exploration plan to perform an interim conditional characterization and evaluation of the Wet FGD Stacking Area more commonly known as the Gypsum Stack. This report is the first report in a two part series which summarizes the results of the Phase 2 Geotechnical Exploration for the Widows Creek Fossil Plant.

2. General Site Description and Geology

2.1. Location

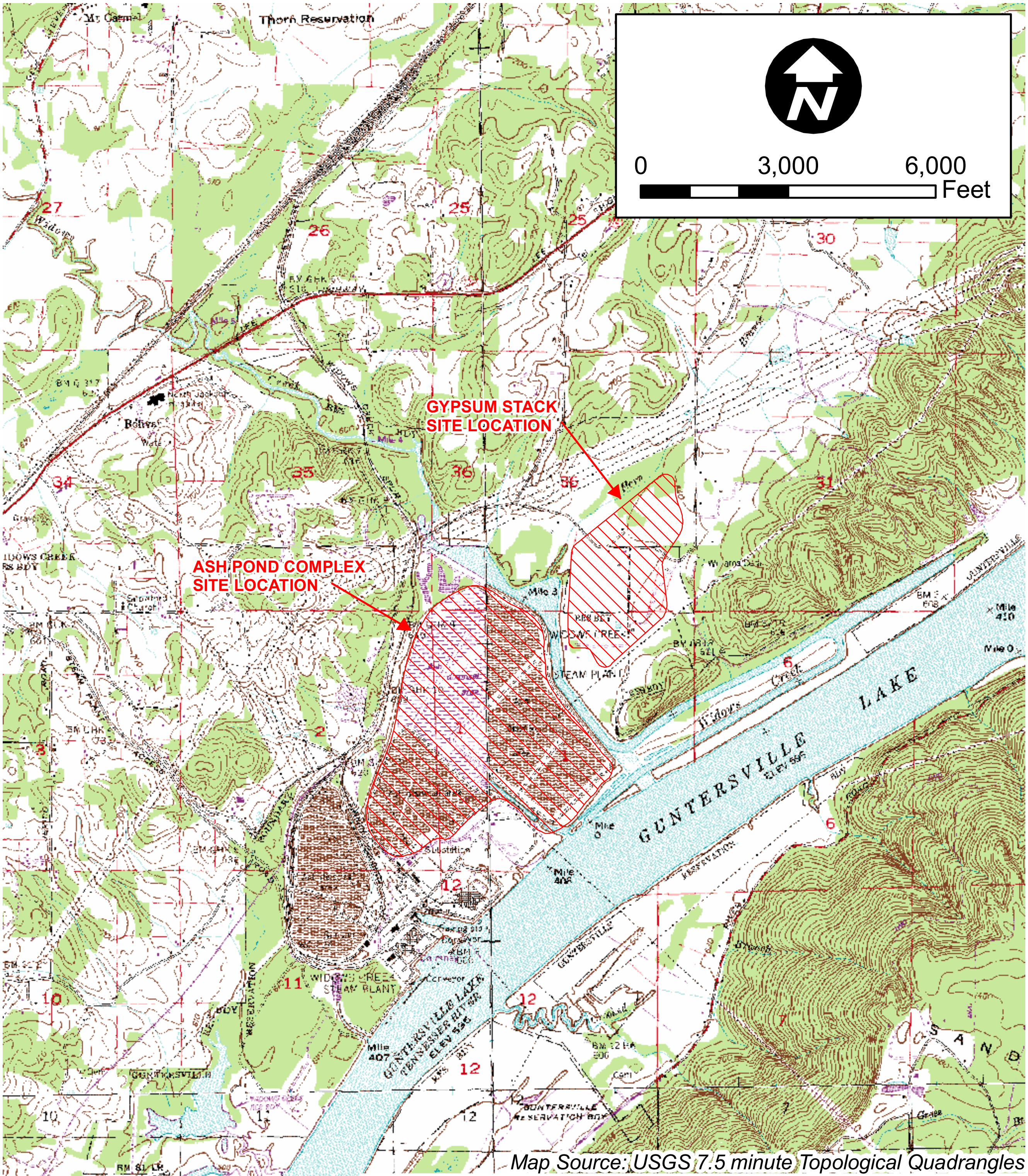
The Widows Creek Fossil (WCF) Plant is located in northeastern Alabama along the west shore of the Tennessee River and at the confluence of Widows Creek. More specifically, the plant is located at 2800 Steam Plant Road in Stevenson, Jackson County, Alabama approximately 40 miles southwest of Chattanooga, Tennessee. A site vicinity map showing the overall facility and the location of the Gypsum Stack is depicted on the geotechnical drawings included in Appendix C.

The active Gypsum Stack and associated Stilling Pond at WCF are situated on a 160-acre site centered approximately one-mile northeast of the plant's powerhouse. The site is accessed from the plant over a one-lane concrete bridge spanning Widows Creek.

2.2. Widows Creek Fossil Plant

Construction at the Widows Creek Fossil Plant began in 1950 and was finished in 1965 with the completion of eight coal-fired turbo-generator units. Units 1 through 6 are the oldest units and Units 7 and 8 became operational in 1964. Since then numerous modification and turn-arounds have occurred at the site; one such modification occurred to Unit 8 (a 550 MW coal-fired boiler) which was the first TVA Fossil Plant to be retrofitted with Flue Gas Desulfurization (FGD) unit or scrubber which allowed the forced oxidation of calcium sulfite to calcium sulfate (gypsum). The production of the effluent slurry from FGD required onsite disposal using the wet stacking method. Based on historical information, the winter net dependable generating capacity is 1,629 megawatts and the aggregate capacity of the eight units is 1,950 MW. The plant currently consumes 10,000 tons of coal a day resulting in approximately 750,000 dry tons of scrubber gypsum being produced each year.

The scrubber gypsum and fly ash mixture is wet sluiced to the on-site stacking area located on the east side of Widows Creek. The original construction of the complex took place in 1981-85 when an initial dike was built. Figure 2 shows the initial construction phases for the wet Gypsum Stack.



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

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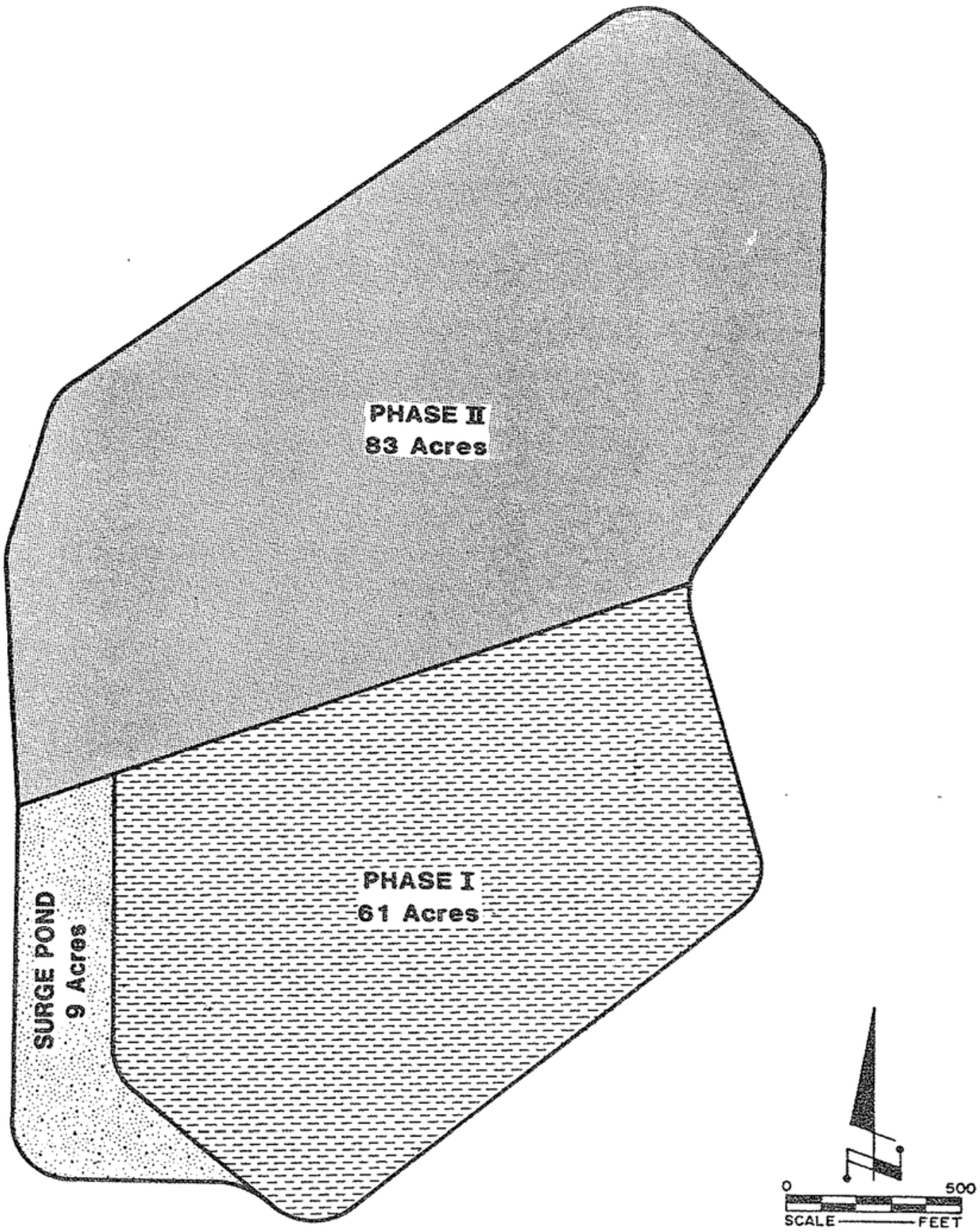


Widows Creek Fossil Plant
Gypsum Stack
Stevenson, Alabama

Stantec

Figure 2. Gypsum Stack Phase Layout (Ardaman, 1991)

Since then, the facility has been used for disposing gypsum and fly ash from Units 7 and 8. During the first phase (Phase 1), gypsum was sluiced into Pond 1 while Pond 2 was used as a clarification pond. Specifically, the gypsum cake as it comes off the filter is mixed with water and pumped as slurry through a pipeline system to the disposal area. The solids are allowed to settle out and clarified excess water is then decanted into a stilling pond. However, in 1994, faced with capacity issues, the complex was horizontally expanded with the development of a second phase (Phase 2) area and converted to wet stacking method which involves mechanically stacking CCP's on top of hydraulically placed gypsum to build perimeter dikes (see Figure 2). The Gypsum Stack is being operated using elevated rim ditching method and upstream method of construction. Long-reach excavators and bull dozers are being used to construct rim ditches and raise the perimeter gypsum dikes.

The wet gypsum-fly ash CCP is currently being sluiced to the Gypsum Stack via a series of six, 18-inch diameter solid HDPE pipes which extend from the east side of the powerhouse. From the plant, the sluice lines general follow the main access road through the Ash Pond complex, across the bridge at Widows Creek, and finally to a valve station located at the southeast end of the Stilling Pond (see Figure 3). From here the slurry can be directed north to Pond 3 or south to Pond 1 at which point it empties into a rim ditch. The rim ditches are positioned at the current crest elevation and traverse the entire perimeter of the pond until they reach a common point located near the near north end of Pond 3. The use of either rim ditch is periodically alternated based on material needs determined by the routine handling and maintenance operators.

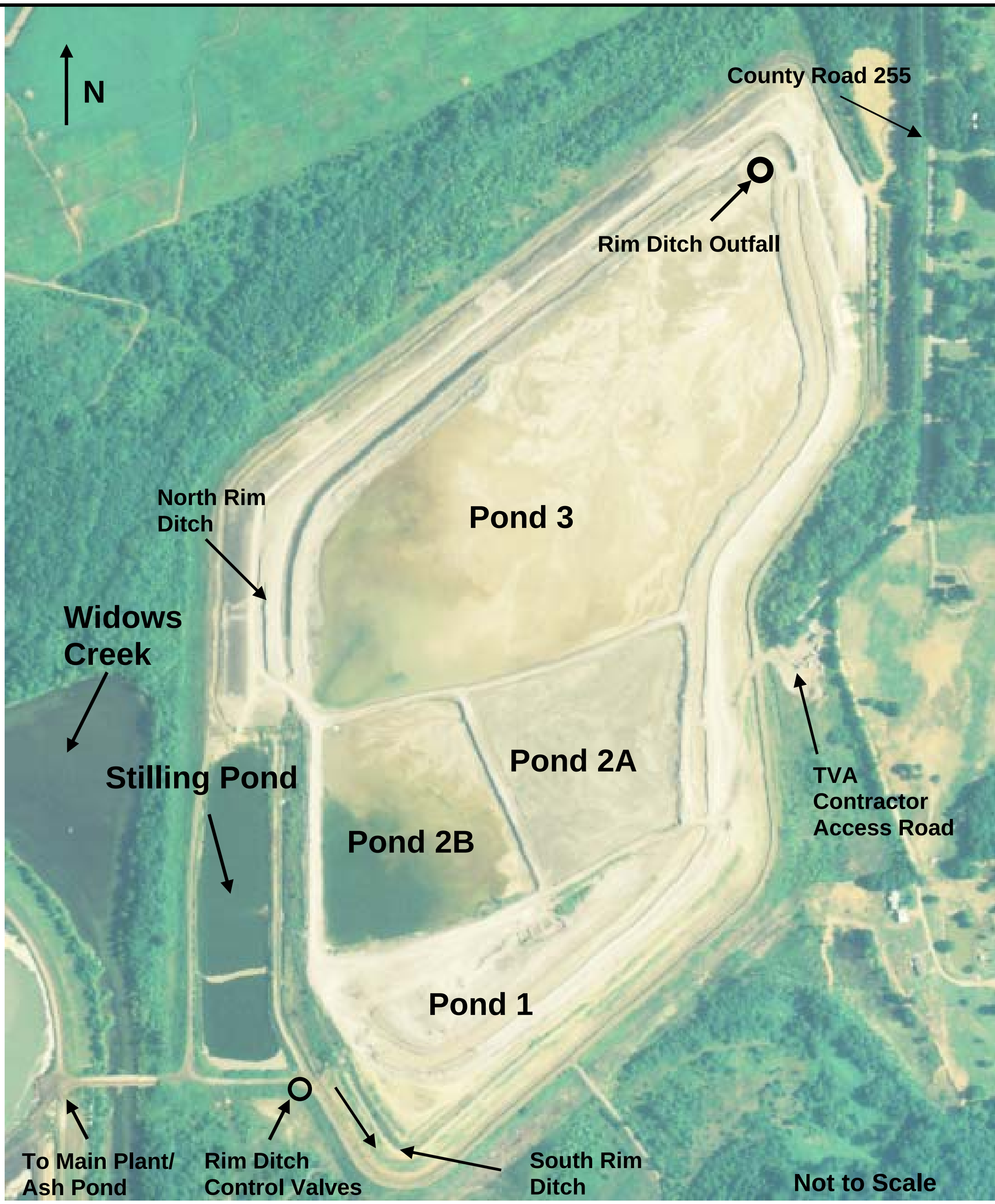
2.3. Gypsum Stack Ponds

The Gypsum Stack consists of four adjoining ponds – Pond 1, 2A, 2B and 3 – separated by internal divider dikes and accompanied by a stilling pond. The layout of these structures is presented in Figure 3.

As the oldest cell, Pond 1 converted from ponding wet sluiced material to stacking in 1998 with an as found stack elevation of about 677 feet. The remaining cells in the complex are active clarification ponds as listed in Table 1. All elevations are expressed in feet.

Table 1. Ponds of Gypsum Stack Complex

Pond	Free Water Area	Top Elevation	Toe Elevation
1	10 acres	677	619
2A	12 acres	678	625
2B	14 acres	670	617
3	59 acres	684	605
Stilling Pond	9 acres	625	610



Widows Creek Fossil Plant Gypsum Stack Stevenson, Alabama	Figure 3. Widows Creek Fossil Plant Gypsum Stack Layout
Stantec	

2.4. General Site Geology

The plant is situated in the Sequatchie Valley District of the Appalachian Plateaus Physiographic Province, just south of a northeast to southwest trending thrust fault. The geologic mapping indicates the plant is underlain by Ordovician age limestone and shale bedrock of the Sequatchie Formation, Nashville Group, and Stones River Group. The Sequatchie Formation consists of thin bedded calcareous shale and mudstone interbedded with fossiliferous or bioclastic limestone. The Nashville Group is described as argillaceous and fossiliferous limestone overlain by laminated silty limestone. The Stones River Group consists of locally argillaceous fossiliferous limestone with bentonite and bentonitic shale near the top of the formation. Although not depicted on the geologic mapping, alluvial deposits are likely present beneath the portions of the site adjacent to the river.

Three potential seismic zones are located in northern Alabama. These zones are identified as the New Madrid Seismic Zone (NMSZ), the Southern Appalachian Seismic Zone (SASZ), and the South Carolina Seismic Zone (SCSZ). Most earthquakes in Alabama occur within the SASZ. Historical records show that earthquakes with epicenters in Alabama have been recorded throughout most of the state but are often not strong enough to be felt on the ground surface. In contrast, if a large earthquake were to occur within the New Madrid zone to the northwest, potential structural damage could occur to dwellings in northern Alabama.

The 2008 version of the USGS National Seismic Hazard Maps estimates a peak horizontal acceleration of approximately 0.058g for a seismic event with a 10% probability of exceedance in 50 years. However, it should be noted that based on the available mapping, no faults or other geological hazards are located within the general vicinity of the project sit.

3. Review of Available Information

3.1. General

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

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

1. Proposed Borrow Site Field Exploration and Top of Rock Contour Map, December, 1980.

2. White Paper on “Evaluation of Engineering Properties and Wet Stacking Disposal of Widows Creek FGD Gypsum - Fly Ash Waste”, Garlanger, John E., Sal H. Magliente, Thomas S. Ingra, and James L. Crowe, December, 1983.

3. “Conceptual Design Recommendations for Construction and Management of the Widows Creek FGD Gypsum – Fly Ash Waste Disposal Facility”, Ardaman and Associates, Inc., August 12, 1983.

4. “Annual Inspection of Waste Disposal Areas”, TVA Engineering Design Services, 1983-2008.

5. “Interim Report on Evaluation of FGD Gypsum – Fly Ash Wet-Stacking Disposal Facility, Widows Creek Steam Plant, Stevenson, Alabama”, Ardaman and Associates, Inc., April 22, 1991.

6. “Proposed Management Plan, Widows Creek Gypsum – Fly Ash Storage Facility, Stevenson, Alabama”, Ardaman and Associates, Inc., June 18, 1991.

7. “Preliminary Evaluation, Proposed Underdrain System, Widows Creek Gypsum – Fly Ash Storage Facility, Stevenson, Alabama”, Ardaman and Associates, Inc., August 2, 1991.

8. “Report of Geotechnical Drilling, Gypsum/Fly Ash Storage Facility, Widows Creek Fossil Plant, Stevenson, Alabama”, MACTEC Engineering and Consulting, June 23, 2004.

9. “Engineering Evaluation and Design Recommendations for Renovation of Widows Creek Gypsum – Fly Ash Storage Facility”, Ardaman and Associates, Inc., October 14, 2004.

10. “WCF Gypsum Stack Remediation”, Memorandum from Mr. Bill Jackson of Ardaman and Associates, Inc. to Mr. Mike Hughes of TVA, August 24, 2005.

11. “Preliminary Assessment of Seepage Collection Drains at TVA Widows Creek Gypsum – Fly Ash Storage Facility”. Memorandum from Mr. Bill Jackson of Ardaman and Associates, Inc. to Mr. Mike Hughes of TVA, March 2, 2006.

12. “Widows Creek Gypsum – Fly Ash Storage Facility Summary of Report Findings (Draft)”, GeoSyntec Consultants, March 23, 2006.

13. “Drawing Series 10W235 Forced Oxidation Gypsum Stacking Phase 1 & 2”, December 3, 2007.

14. “Phase 1 Facility Assessment Report”, Stantec, June, 2009.

15. “Dike Stability Corrective Measures and Spillway Modification”, Stantec, June 2, 2009.

16. Root Cause Analysis of TVA Kingston Dredge Cell Pond Failure from December 22, 2008, AECOM, June 12, 2009

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

3.2. Site History

A list of key events related to the planning, construction and operation of the Gypsum Stack is provided in Table 2.

Table 2. Summary of Events

Date*	Event
December, 1980	Initial exploration performed (31 rock profile borings), topographic survey with top of rock contour map
1981	Pilot-scale stacking facility demonstration project
December, 1985	Earthen dike construction complete for Phase 1 ponding
November, 1990	Topographic survey and exploration performed (7 SPT borings including 3 piezometers)
June, 2004	Topographic survey and exploration performed (6 SPT borings including 6 piezometers)
2005	Components of toe drain system retrofitted in side slope of stack
2006	Vegetative soil layer and grassing on side slopes of stack
December 2008	O&M management transferred from TVA-HED to Trans Ash
January, 2009	Loss of Pool in Pond 2A/2B Compartment, Topographic survey
July, 2009	Exploration performed (6 SPT borings including 6 piezometers)
August, 2009	Completion of interim Dike Stability Corrective Measures (Work Plan 5)

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

Since 1967, TVA has performed yearly inspections of the stacks at the Widows Creek Fossil Plant and made subsequent repairs based on the observed conditions. As part of this year's facility assessment work, Stantec reviewed the information within these reports to determine if the recommendations/remediation measures had been implemented by TVA.

3.3. 1983 Conceptual Design Recommendations

The basis of operation is found in the referenced Ardaman reports dated 1983, 1991 and 2005. Development of engineering properties for the Widows Creek flue gas desulfurization (FGD) was first undertaken by Ardaman & Associates, Inc. (Ardaman) in the early 1980's. The results of that study were reported to TVA in 1983 and were subsequently used as the basis for the design and layout of the Phase 1 facility.

In 1983, Ardaman provided two design alternatives with one concept including an underdrain system and one with no underdrains.

Ardaman concluded that without underdrains the safe operation of the stack could be maintained using a design side slope geometry of 5H:1V. In this configuration, the final stack height would reach elevation 720 feet.

A significantly greater capacity could be achieved using underdrains. Ardaman presents a second alternative design with average side slopes at 3H:1V provided a system with three rings of internal drains be installed as the stack increased in height.

3.4. 1991 Design Modification

Ardaman was retained again in 1990 to provide engineering design recommendations. The 1991 Ardaman report documented the absence of underdrain components, instability along the toe of the stack caused by seepage and the side slopes were being constructed steeper than recommended. The two primary controlling factors in the overall stability of the stack were the relatively low strength of the clay foundation soils and the high phreatic surface and seepage gradients within the saturated gypsum – fly ash mass. Design modification reports provided in 1991 included a proposed underdrain system and proposed management plan. Ardaman suggested TVA make the interim field adjustments including underdrains and 3.0 horizontal to 1.0 vertical side slopes.

3.5. 2005 Engineering Evaluation

The 2003 annual inspection reported operational safety and stability issues. The 2005 Design modification recommendations include side slope regrading and improvement of the toe ditch.

The field adjustments included: (1) steepen the design side slopes to 2.75H:1V below elevation 655 and 2.5H:1V above elevation 655 feet to more closely represent field conditions (2) provide stability (a.k.a. setback) benches located every 35 feet vertically resulting in planned benches at elevation 655, 685, and 715 feet and (3) retrofit underdrain components at the toe of slope near the elevation 625 level.

A review of documents identifies a seepage control or underdrain system was recommended by Ardaman and Associates in 1991. As modifications to the original system, at least five types of buried drainage features are shown in the documents in past five years.

Figure 4. 2005 Site Conditions Photograph

Figure 5. 2006 Site Conditions Photograph

Figure 6. 2009 Site Conditions Photograph

3.6. 2007 Drawing Series 10W235

3.6.1. Overview of Seepage Control System

Drawing API08-2 consists of a plan titled “Forced Oxidation Gypsum Stacking Phase 1 & 2 – Top Drain, 650/655 Bench Drain And Outlet Pipes”. This drawing highlights the seepage system observed to be in place as of the FY2008 inspection. It also summaries the erosion, wet areas and sloughing observed during the inspection. According to the drawing a total of 15 outlet locations are planned around the Gypsum Stack but Outlets 1 and 2 (along the west slope) are not yet installed. Associated with the underdrain outlet structures is a series of interconnected toe drains and slope drains which in turn create an underdrain system for the stack. The components of the as-designed underdrain are discussed in more detail in the following sections.

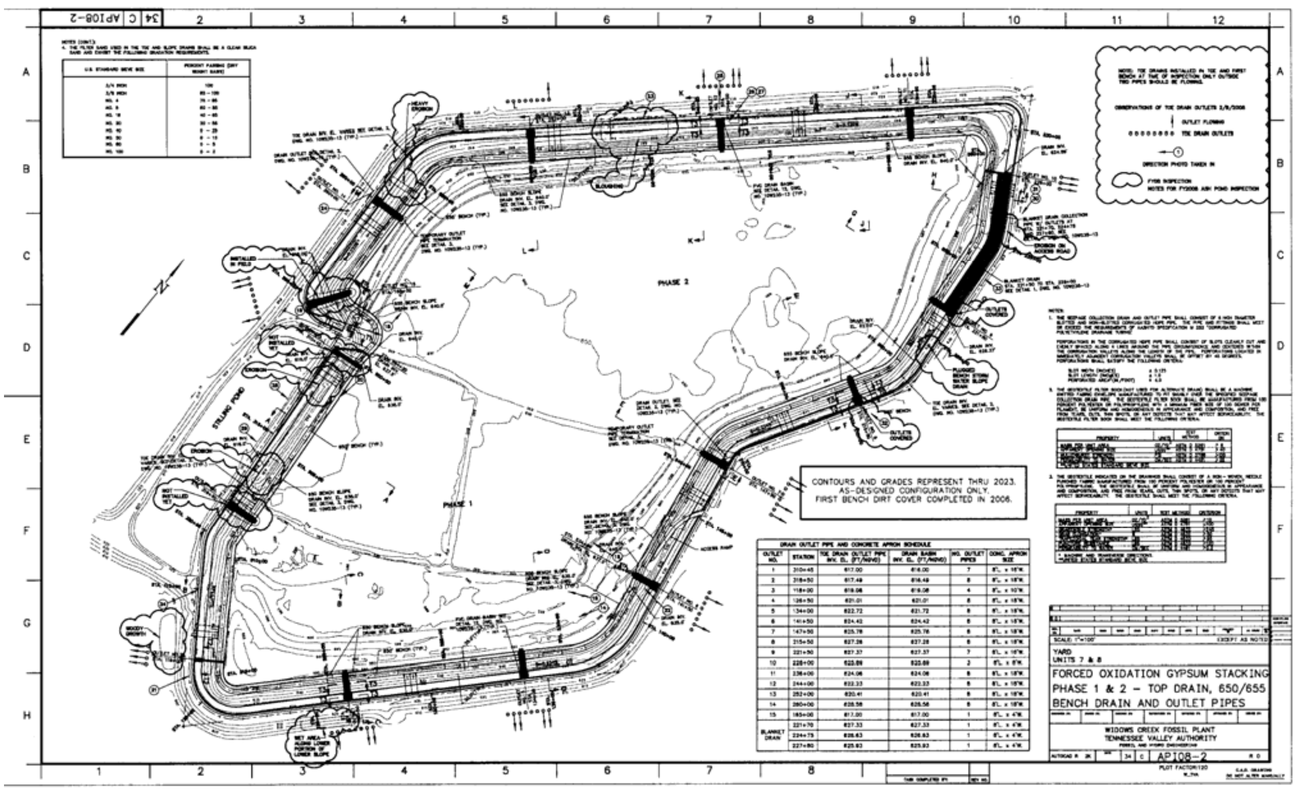


Figure 7. Toe Drain, 650/655 Bench Drain and Outlet Pipe

3.6.2. Toe Drains

Figure 8 presents a typical toe drain outlet taken from TVA Design Drawing 10W235-13 “Slope Drain Details”, revised December 3, 2007. According to the drawing detail, each outlet structure will consist of two 18” diameter PVC pipes which collects the seepage around the bottom of stack via a continuous toe drain and discharges the seepage water to the perimeter ditch through eight pipe outlets with a concrete apron/outlet structure. For a more detailed description please see the following figure.

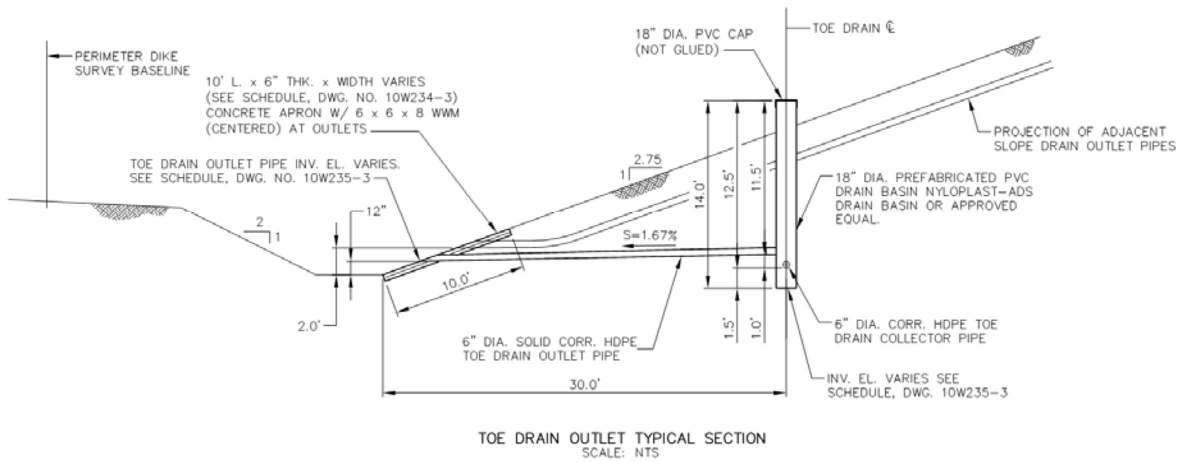


Figure 8. Toe Drain and Outlet Typical Section

3.6.3. Toe Drain Typical Section

Figure 9 presents a typical toe drain taken from TVA Design Drawing 10W235-13 "Slope Drain Details", dated December 3, 2007. According to the drawing detail, each toe drain will consist of a chimney drain measuring two feet wide by 8 feet tall and will traverse the bottom of the stack at the elevations shown. A 6" diameter corrugated HDPE pipe will be at the bottom of each toe drain to collect the seepage water and discharge into the toe drain outlets.

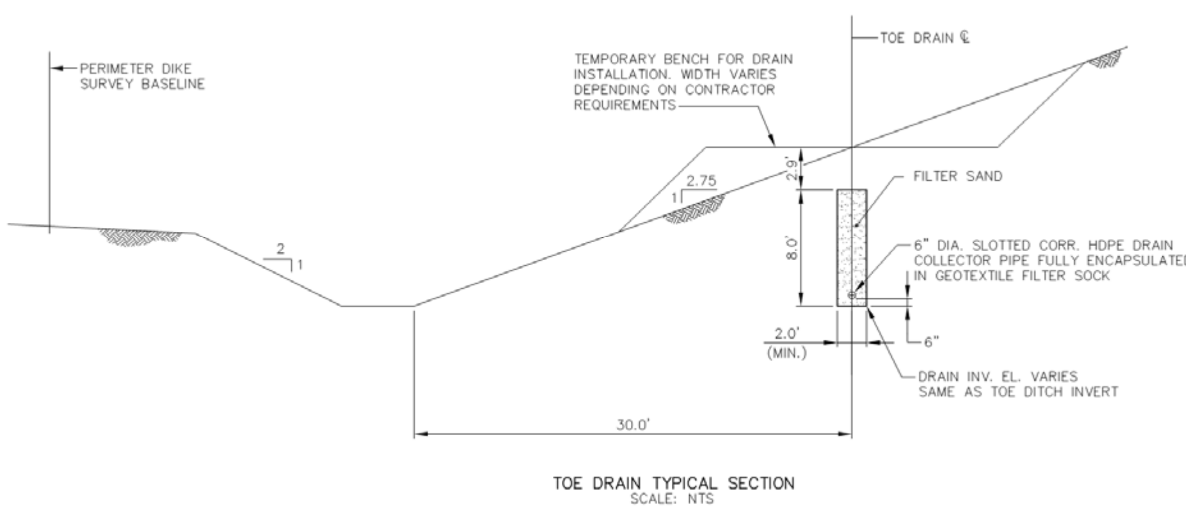


Figure 9. Toe Drain Typical Section

3.6.4. Slope Drain and Outlet Typical Section

Figure 10 presents a typical slope drain taken from TVA Design Drawing 10W235-13 “Slope Drain Details”, dated December 3, 2007. According to the drawing detail, each bench level (650/655, 685, 720) will have slope drains similar to the toe drains described above, to collect seepage as the Gypsum Stack is expanded. The slope drains consist of a chimney-type drain measuring two feet wide by 10 feet tall and will traverse around the stack at each bench level as shown in Figure 7. At each outlet well the slope drains will transition from a 6” corrugated HDPE collection pipe to a 6” diameter solid HDPE pipe which turn down the embankment slope (as shown in the detail) and outlet into the perimeter ditch at the concrete aprons.

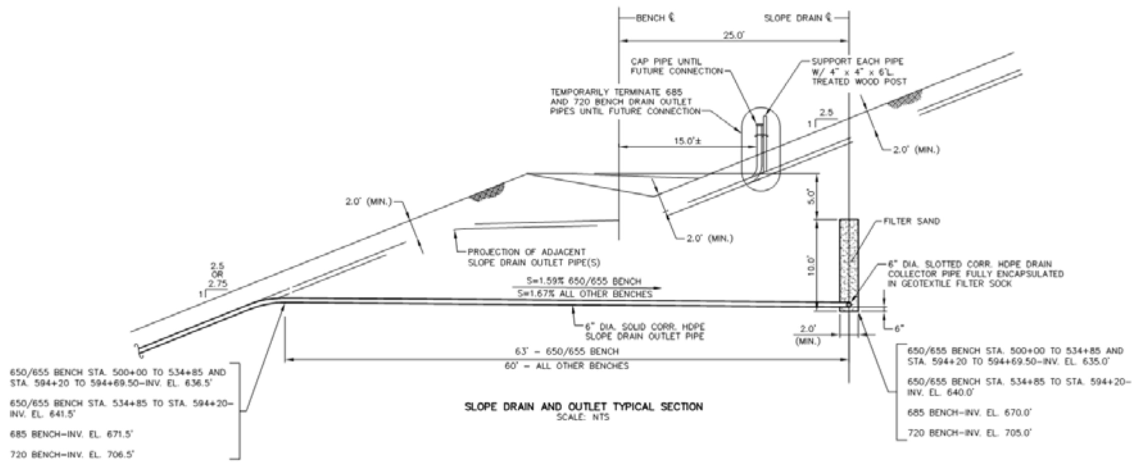


Figure 10. Slope Drain and Outlet Typical Section

3.6.5. Alternate Surface Water Collection

Figure 11 presents an alternate surface water collection detail to control surface water infiltration around the perimeter toe, planned near the south east end of the Stilling Pond. The detail was taken from TVA Design Drawing 10W235-16 - “Water Collection Detail Modified”, dated December 3, 2007. Based on the information obtained from the site inspections and the geotechnical exploration, this drain does not exist.

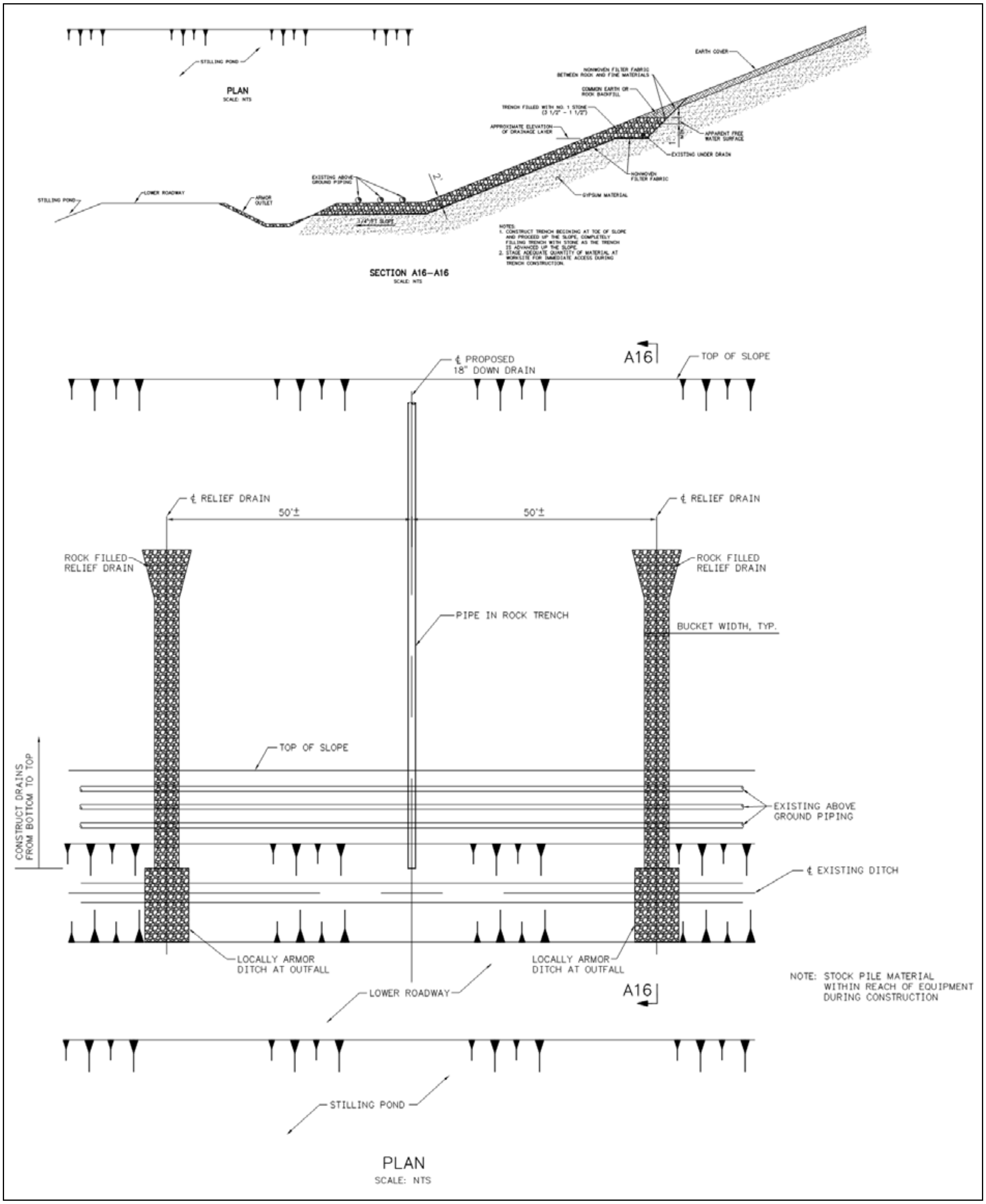


Figure 11. Modified Surface Water Collection

3.6.6. Blanket Drain and Outlet

Figure 12 presents a typical blanket drain and outlet taken from TVA Design Drawing 10W235-13 "Slope Drain Details", revised December 3, 2007. According to the drawing detail, the blanket drain will consist of a two foot deep section of ALDOT No. 357 coarse aggregate. The entire blanket drain will be wrapped in a geotextile fabric. Inside the coarse aggregate drain, a 6" diameter slotted corrugated HDPE collector pipe will be placed at the centerline. Three solid HDPE outlet pipes are required. The blanket drain will daylight into the perimeter ditch.

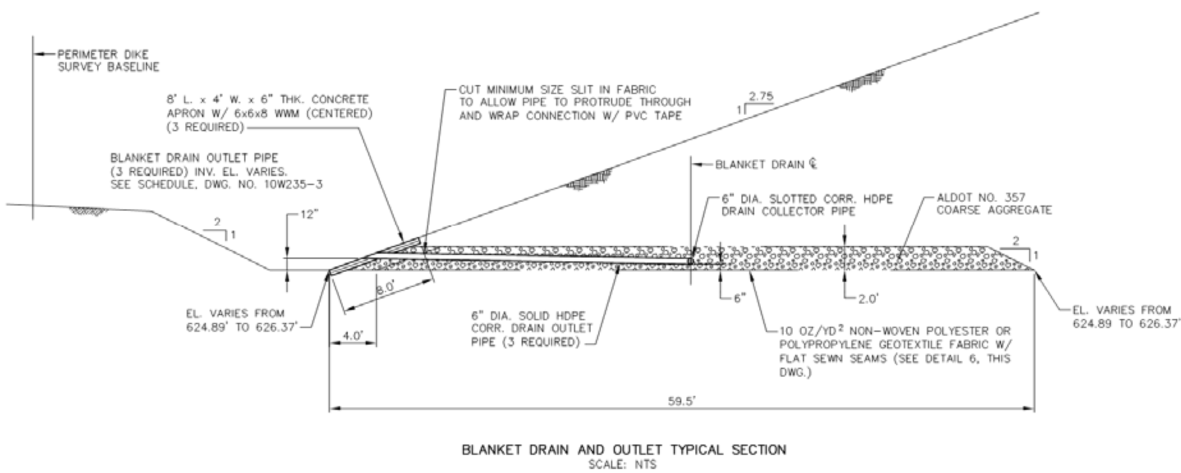


Figure 12. Blanket Drain and Outlet Typical Section

The blanket drain was proposed for approximately 650 feet on the northeast section of the Gypsum Stack between survey baseline Station 221+50 and Station 228+00. While Ardaman and Associates mentions the blanket drain in a memo from Bill Jackson to Mark Hughes dated March 2, 2006, Stantec did not encounter any gravel indicating the installation of the blanket drain during geotechnical exploration (STN-36) and seepage continues to be an issue at the toe of the stack. Further information is needed to verify whether this blanket drain was installed.

3.6.7. Pond 3 Internal Underdrains

Figure 13 presents a detail taken from Drawing 10E7416-3 "Conceptual Plan, Sections and Details", revised July 19, 2001. The referenced construction drawing calls for the installation of two perimeter underdrains in the central portion of Pond 3 but not for the installation of a corresponding toe drain. The drain design detail shown calls for an 8-inch perforated HDPE pipe, double wrapped in a woven geotextile fabric. The detail does not call for any gravel bedding or for a gravel collection zone.

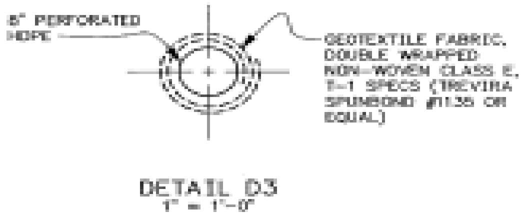


Figure 13. Pond 3 Internal Drain Detail

3.7. Review of Underdrain Performance

At the surface, a series of fifteen outlet structures are planned. There are eight outlet pipes for each outlet (see Figure 14). Based on the design of the slope drain and toe drain system, the two outside outlet pipes (1 and 8 as shown left to right) are connected to the toe drains, and the next two (2 and 7) are connected to the 655 bench slope drains. The middle four outlet pipes (3, 4, 5 and 6) are for the planned 685 bench and 720 bench slope drains which have not been installed.

Figure 14. Typical Outlet

Due to the fact that some outlet pipes were buried in silt and/or underwater, it was difficult to determine the effectiveness of the underdrain system. Through discussions with TVA, it is understood that Outlets 1 and 2 were not installed. Based on the site inspections, Outlets 3, 4, 5, 11, and 14 show signs that both the toe drain and 655 bench slope drain are working properly. Outlets 7, 8, 9, 10, and 15 had outlet pipes that were either covered up or submerged in the perimeter ditch, and it was not possible to determine functionality at those locations. Outlet 12 had some outlet pipes covered up, but the outer pipes were still visible and did not show signs of flowing water. Outlet 6 had all outlet pipes visible, while flow was only seen in one of the 655 bench outlet pipes. The performance of the underdrain system is discussed in detail in the memo sent from Bill Jackson of Ardaman and Associates to Mike Hughes of TVA on March 2, 2006. The referenced historical documents are included electronically on a DVD in Appendix A.

3.8. Surface Water Control

As mentioned above, the current design plans (TVA series 10W235) call for a series of eleven (601-611) bench drains around the 685 bench level and ten (Series 500-510) around the 650/655 bench level. During Stantec's field inspection only 6 of the 500 series bench drain pipes were found. However, corresponding inlet structures at the 655 bench level have not been installed. The 655 and 685 bench level are graded flat allowing storm water to sheet flow to the perimeter ditch. Erosion of the vegetative cover and underlying stack on these side slopes require significant maintenance after storms.

Likewise, the perimeter ditch which is located around the toe of the pond and is to receive the run-off from the bench drains, the seepage water from the chimney drains, and the water from the spillway structures was observed to be partially filled with gypsum-fly ash sediments and/or heavy vegetation which inhibits the flow of water. A Hydrology and Hydraulics study of the Gypsum Stack was conducted based on a 25 year, 24 hour storm event. Based on this analysis, remediation measures will be required to improve the surface water control and the perimeter ditch system.

3.9. Record Drawings

As-constructed drawings were not available for review for the Gypsum Stack at Widows Creek Fossil Plant. However, TVA periodically conducted topographic surveys of the Gypsum Stack. These surveys were generally performed in association with a geotechnical evaluation of the stack at intermediate heights. Surveys were conducted on August 16, 1990 and March 8, 2004. This topographic information gives a general idea of how the Gypsum Stack was constructed. Also, GeoSyntec, in their March 2006 report evaluating the as found conditions at the Gypsum Stack, discussed historical documents pertaining to the construction of the underdrain system proposed by Ardaman and Associates in 1991. GeoSyntec concluded that some portions of the underdrain system may not have been installed as designed due to construction difficulties.

3.10. O&M Manual

Stantec is unaware of an operations and maintenance manual for the Gypsum Stack. Geotechnical studies of the Gypsum Stack conducted by Ardaman and Associates and GeoSyntec include operational recommendations, but it is understood a manual has not been maintained. TVA conducts annual site inspections of all CCP facilities at the Widows Creek Fossil Plant, including the Gypsum Stack, and this information is used to determine any potential problems. The annual reports are discussed in the following section.

3.11. Annual Reports

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

3.12. 2009 Immediate Risk Reduction Measures

As a result of the January 9th loss of pool event, a project to improve stability was implemented along the west embankment slope (see Work Plan 5 in Appendix A) which consisted of flattening the side slopes, lowering the crest elevation of Pond 2B, constructing a crushed stone buttress along the toe/perimeter ditch, extending the active stack spillway pipes directly into the stilling pond, and installing a seepage collection system within the perimeter ditch. At the time of this report, the existing slope condition along the west embankment (Section K) consisted of a rock toe buttress with 2:1 side slopes extending to 630 feet in elevation. The rock buttress bench is approximately 10 feet in width. The slopes above the buttress were then flattened to 4H:1V and a two-foot layer of crushed stone, designed as a reverse graded filter, was extended up the slope to 640 feet in elevation. Above this elevation the side slopes covered with a vegetative cover soil and extended up to a 20 foot wide bench at 655 feet in elevation. The side slope then continues at 3:1 grade until reaching the current crest elevation at 660 feet. A more detailed description of the existing slope geometry along Section K can be found on the geotechnical drawings in Appendix C and G. It should be noted the slope conditions described above have most likely been modified, as of this writing. The Gypsum Stack is an on going operation and therefore existing conditions are continuously changing.

As part of the stability improvements described above, abandoned spillway pipes (Pipe 14 and 15) located between the Stilling Pond and Pond 2B were sealed and grouted in accordance with Work Plan 6 which is described in detail in Appendix A.

At the time of the Phase 1 site inspection at Widows Creek Facility, the west embankment (Section K) and the north east embankments (Sections D and F) appeared to exhibit signs of seepage. However, the west embankment was the only reach which presented excessive seepage. The bottom one third of the slope, located between Outlet 15 and the Future Outlet 2 (TVA Drawing 10W235-6 in Appendix A), appeared to be only marginally stable. As a result of the seepage and slope conditions observed along the west embankment at Section K, stability improvements were initiated on June 2, 2009.

4. Scope of Work

Immediately following the events on January 9, 2009, TVA requested Stantec to mobilize to Widows Creek Fossil Plant and provide 24-hour emergency mitigation and engineering services. On January 11th, Stantec started drilling the initial six soil borings labeled STN-45, 48, 49, 50, 51, and 52. This initial drilling effort was completed on January 20, 2009. Following the emergency drilling response and a general site inspection of the stack, Stantec provided TVA with a written scope of work on February 3, 2009 to address TVA's Engineering Service Request (ESR) 909. It should be noted however; the ESR 909 includes both the Gypsum Stack and Main Ash Pond Complex. This report addresses only the Gypsum Stack. The Ash Pond report will be submitted under a separate cover. As the Phase 2 drilling was completed and Stantec began to work through some preliminary analyses and the scope of work was better defined to meet TVA's immediate requests, Stantec submitted Addendum 3 on June 30, 2009. The Addendum included additional scope items such as additional construction field services for various work plans, engineering services for hydrologic and hydraulic assessments, surveying work, and CPT/Vane Shear test borings.

The fieldwork for the Geotechnical Exploration was completed in late July, 2009. The scope of work included advancing a total of twenty-two (22) auger sample borings, two (2) vane shear borings, and four (4) CPT borings across the site at the approximate locations shown on the boring layout in Appendix C. The borings were drilled using a track-mounted and truck-mounted drill rig equipped with 3¼ and 4¼-inch (ID) hollow stem augers equipped with a carbide-tipped tooth bit and NQ size rock coring equipment. All of the boring locations were initially staked in the field by Stantec personnel based on the observed site conditions. Continuous standard penetration testing (SPT) were performed in most of the borings advanced at the site, while undisturbed sampling, rock core sampling, cone penetration testing, and vane shear testing were performed in selected locations. Of the 24 borings a total of eighteen (18) were instrumented with the following equipment; three (3) were instrumented with slope inclinometer casing and fifteen (15) were installed with standpipe piezometers (casagrande style and/or slotted PVC). The location of instrumented holes is shown on the instrumentation plan in Appendix C. The slope inclinometers were installed to monitor any possible slope movement along the north and west exterior slopes of Pond 3 and the interior divider berm between Pond 3 and Pond 2B. The standpipe piezometers (casagrande style) were installed to determine the pore water pressures/groundwater levels at selected locations within the embankment to aid in seepage design and in determining trigger levels/alters for possible slope instability. The final surface elevations and as-drilled locations for each of the borings/instruments were obtained by TVA's survey crew upon completion of the drilling. Detailed boring logs and piezometer installation records can be found in Appendix B.

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

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

The results of the field and laboratory testing services were then used to develop critical stability sections around the Gypsum Stack at locations were possible signs of instability were observed. Stantec initially reviewed 15 cross sections (A through O) located at various distances along the perimeter of the Gypsum Stack. Based on the results of the field exploration, cross-section geometry, piezometer conditions, and observed slope conditions; Stantec reduced the total number of stability sections to five critical sections (Sections A, D, F, H, and K). Stantec then performed seepage and slope stability analyses based on the observed existing conditions for the five critical sections. The results are presented in Section 9.

An inventory of the pipe penetrations through the perimeter of the stack was conducted. The assessment, remedial design, and construction work is being documented under a separate cover referred to as Work Plan 6.

5. Results of Geotechnical Exploration

5.1. Summary of Borings

A summary of the boring information is presented in Table 3, where all measurements are expressed in feet. Typed drilling logs and piezometer installation records are presented in Appendix B and the CPT results are included in Appendix G.

Table 3. Summary of Borings

Boring Number	Surface Elevation (feet)	Top of Rock Elevation (feet)	*Refusal/Begin Core Elevation (feet)	Boring Termination Depth (feet)	Length of Rock Core (feet)	Bottom of Hole Elevation (feet)
STN-28	651.3	592.5	592.5	58.8	-	592.5
STN-29	623.9	-	NR (605.4)	18.5	-	605.4
STN-31	672.6	591.5	591.5	81.1	10.7	580.8
STN-32	656.2	597.0	597.0	59.2	-	597.0
STN-33	640.2	613.9	613.9	26.3	-	613.9
STN-34	674.1	-	NR (621.1)	53.0	-	621.1
STN-35	656.1	618.9	618.9	37.2	-	618.9
STN-36	631.9	617.7	617.7	14.2	-	617.7
STN-37	627.0	-	NR (598)	29.0	10.3	587.7
STN-38	675.5	585.5	585.3	90.2	-	585.3
STN-39	655.2	-	NR (600.7)	54.5	-	600.7
STN-40	621.4	585.3	585.3	36.1	10	575.3
STN-41	657.8	-	NR (603.8)	54.0	-	603.8
STN-42	659.0	-	NR (599.5)	59.5	-	599.5
STN-43	672.7	-	NR (621.7)	51.0	-	621.7
STN-44	655.8	-	NR (601.8)	54.0	-	601.8
STN-45	655.2	-	NR (604.2)	51.0	-	604.2
STN-46	654.7	594.9	594.9	59.8	-	594.9
STN-47	655.5	595.3	595.3	60.2	9.8	585.5
STN-48	654.9	-	NR (597.5)	57.4	-	597.5
STN-49	655.0	591.5	591.5	63.5	-	591.5
STN-50	654.9	592.7	592.7	62.2	-	592.7
STN-51	621.1	586.6	586.6	34.5	-	586.6
STN-52	621.2	587.4	587.4	33.8	-	587.4
**CPT-10	630.0	-	599.3	30.7	-	599.3
**CPT-11	655.0	-	614.1	40.9	-	614.1
**CPT-12	630.0	-	591.5	38.5	-	591.5
**CPT-13	655.0	-	625.4	29.6	-	625.4
***V-9	655.0	-	614.0	41.0	-	614.0
***V-10	655.0	-	617.0	38.0	-	617.0

Key

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

5.2. Subsurface Soil Conditions

Based on the results of the drilling program, the Gypsum Stack is underlain by four predominant soil types: Soil 1 - Cast Gypsum-Fly Ash, Soil 2 - Sedimented Gypsum-Fly Ash, Soil 3 - Residual Fat Clay, and Soil 4 Weak Sedimented Gypsum – Fly Ash. Soil 1 was visually classified as Silt (Cast Gypsum-Fly Ash), light gray to dark gray in color, damp to wet in natural moisture content, and soft to very stiff in consistency. Soil 1 ranged in thickness across the site up to over 54 feet at the crest (~Elev. 655 feet). The average thickness of the cast gypsum-fly ash encountered during this exploration was 26 feet.

The next predominant soil type, Soil 2, was visually classified as Silt (Sedimented Gypsum-Fly Ash), light gray to black in color, moist to wet in natural moisture content, stiff to very stiff in consistency. Soil 2 ranged in thickness up to 60 feet and had an overall average thickness of 18 feet.

The third soil horizon, Soil 3, was visually classified as residual Fat Clay, tan to red in color with gray mottling, moist to wet in natural moisture content, soft to very stiff in consistency, with occasional chert fragments. Soil 3 was encountered directly above the top of rock and measured approximately 10 to 35 feet in thickness.

The fourth soil horizon, Soil 4, was encountered in only six of the borings (STN-38, 39, 42, 45, 48, and 49). Soil 4 was encountered primarily along the west embankment and ranged in elevation from 600 feet to 625 feet. Soil 4 was visually described as silt (Weak Sedimented Gypsum-Fly Ash), dark gray to black in color, moist to wet in natural moisture content and soft in consistency.

Four additional soil types were encountered within the borings drilled during this exploration; however these soil types were encountered in only a few borings and occurred at sporadic locations throughout the Gypsum Stack. The remaining soil types encountered were visually described as follows: Soil 5 –Lean Clay, tan to red, moist, very stiff to stiff, with occasional chert fragments; Soil 6-Clay with Silt, brown to gray, moist, stiff; Soil 7-Crushed Limestone (KY No. 57); Soil 8-Alluvial Silt, gray medium stiff to stiff, with weathered rock fragments.

5.3. Standard Penetration Tests

A total of twenty-two (22) borings were sampled with standard penetration tests (SPT) at the approximate locations shown on the attached layout in Appendix C and at the depths indicated on the boring logs in Appendix B. The SPT sampling was performed in accordance with the procedures outlined in ASTM D 1586, "Penetration Test and Split Barrel Sampling of Soils". This method is typically used to obtain soil samples, estimate the consistency or relative density of the soil, and also to estimate the vertical limits of the subsurface soil horizons. A summary of the average blow counts for the four predominant soil horizons encountered in each of the borings is presented in Table 4 where all measurements are expressed in feet. The N values have also been corrected due to overburden and hammer efficiency and estimates of unit wet weights along with friction angle estimates are included in the SPT correlation tables in Appendix D.

Table 4. Summary of Standard Penetration Tests

Stack Location	Boring Number	Soil Horizon	*Average Blow Count Value $N_{(80)}$
North	STN-37	Cast Gypsum – Fly Ash	11
		Sedimented Gypsum – Fly Ash	NA
		Fat Clay	16
		Weak Sedimented Gypsum – Fly Ash	NA
East	STN-35	Cast Gypsum – Fly Ash	30
		Sedimented Gypsum – Fly Ash	19
		Fat Clay	23
		Weak Sedimented Gypsum – Fly Ash	NA
	STN-36	Cast Gypsum – Fly Ash	NA
		Sedimented Gypsum – Fly Ash	NA
		Fat Clay	9
		Weak Sedimented Gypsum – Fly Ash	NA
South	STN-32	Cast Gypsum – Fly Ash	18
		Sedimented Gypsum – Fly Ash	27
		Fat Clay	13
		Weak Sedimented Gypsum – Fly Ash	NA
	STN-33	Cast Gypsum – Fly Ash	19
		Sedimented Gypsum – Fly Ash	17
		Fat Clay	15
		Weak Sedimented Gypsum – Fly Ash	NA
	STN-28	Cast Gypsum – Fly Ash	14
		Sedimented Gypsum – Fly Ash	20
		Fat Clay	8
		Weak Sedimented Gypsum – Fly Ash	NA
	STN-29	Cast Gypsum – Fly Ash	NA
		Sedimented Gypsum – Fly Ash	38
		Fat Clay	17
		Weak Sedimented Gypsum – Fly Ash	NA
West	STN-38	Cast Gypsum – Fly Ash	19
		Sedimented Gypsum – Fly Ash	23
		Fat Clay	NA
		Weak Sedimented Gypsum – Fly Ash	3
	STN-39	Cast Gypsum – Fly Ash	23
		Sedimented Gypsum – Fly Ash	26
		Fat Clay	6
		Weak Sedimented Gypsum – Fly Ash	4
	STN-40	Cast Gypsum – Fly Ash	9
		Sedimented Gypsum – Fly Ash	NA
		Fat Clay	15
		Weak Sedimented Gypsum – Fly Ash	NA
	STN-42	Cast Gypsum – Fly Ash	45
		Sedimented Gypsum – Fly Ash	27
		Fat Clay	20
		Weak Sedimented Gypsum – Fly Ash	11
	STN-45	Cast Gypsum – Fly Ash	27
		Sedimented Gypsum – Fly Ash	NA
		Fat Clay	9
		Weak Sedimented Gypsum – Fly Ash	8

Table 4. Summary of Standard Penetration Tests

Stack Location	Boring Number	Soil Horizon	*Average Blow Count Value $N_{(80)}$
West	STN-46	Cast Gypsum – Fly Ash	NA
		Sedimented Gypsum – Fly Ash	34
		Fat Clay	15
		Weak Sedimented Gypsum – Fly Ash	NA
	STN-48	Cast Gypsum – Fly Ash	27
		Sedimented Gypsum – Fly Ash	NA
		Fat Clay	15
		Weak Sedimented Gypsum – Fly Ash	5
	STN-49	Cast Gypsum – Fly Ash	25
		Sedimented Gypsum – Fly Ash	NA
		Fat Clay	17
		Weak Sedimented Gypsum – Fly Ash	1
	STN-50	Cast Gypsum – Fly Ash	30
		Sedimented Gypsum – Fly Ash	14
		Fat Clay	18
		Weak Sedimented Gypsum – Fly Ash	NA

Note: $N_{(80)}$ denotes the number of blows required to drive a two-inch diameter split-spoon sampler the final one-foot of the 1.5-foot test interval utilizing a 140-pound hammer free-falling 30 inches.

Based on the average blow counts presented above, the Cast Gypsum – Fly Ash appears to have an average N value of 25, which indicates it is relatively dense. However, some of the increased blow counts recorded on the boring logs is most likely due to mechanical compaction associated with the heavy equipment on site and the placement of borings within the roadway. The second prevalent soil type, Sedimented Gypsum-Fly Ash exhibited an average N value of 22, the Weak Sedimented Gypsum – Fly Ash had an average N value of 3, and the in-situ Fat Clay had an average N value of 14. Based on the drilling information, the Weak Sedimented Gypsum – Fly Ash zone is most likely a layer of historical pond residual which has subsequently been covered up as the stack height increases. As noted in the Ardaman 2004 report, the darker materials typically contained more fly ash than gypsum.

5.4. Undisturbed Sampling

A total of forty-nine (49) undisturbed Shelby tube samples were obtained from select borings drilled during the exploration. The undisturbed samples were retrieved from the borings via a 30-inch long thin walled tube fixed-head piston sampler, which measured 2 7/8-inches inside diameter. The undisturbed soil samples were performed in general accordance with the procedures outlined in ASTM D-1587, "Standard Practice for Thin-walled Tube Sampling of Soils for Geotechnical Purposes." All of the Shelby tube samples were sealed in the field by a Stantec representative and transported to Stantec's Lexington, Kentucky office. A summary of the undisturbed samples retrieved from the site is presented in Table 5, where all measurements are expressed in feet. For a more detailed description please see the boring logs in Appendix B and the geotechnical cross sections in Appendix C.

Table 5. Summary of undisturbed Shelby Tube Samples

Stack Location	Boring Number	Sample Depth (feet)	Recovery Length (feet)
North	STN-37	5.0 – 7.0	1.9
		10.0 – 11.6	1.6
East	STN-32	19.5 – 21.5	2.0
		35.0 – 37.0	2.0
	STN-33	7.0 – 9.0	0.0
		15.0 – 17.0	2.0
		20.5 – 22.5	2.0
	STN-34	15.0 – 17.0	2.0
		32.5 – 34.5	2.0
	STN-35	25.5 – 27.5	1.2
South	STN-28	10.5 – 12.5	1.4
		39.5 – 41.5	2.0
	STN-29	4.5 – 6.5	1.5
West	STN-31	25.0 – 27.0	2.0
		44.5 – 46.5	1.2
		64.5 – 66	1.5
	STN-38	12.0 – 13.7	1.7
		32.0 – 34.0	2.0
		71.0 – 73.0	2.0
		74.5 – 76.5	2.0
		82.8 – 84.8	2.0
	STN-39	27.5 – 29.5	1.8
		37.0 – 39.0	2.0
		49.5 – 51.5	2.0
	STN-40	7.5 – 9.5	2.0
	STN-41	22.0 – 24.0	2.0
		37.0 – 39.0	1.9
		52.0 – 54.0	1.5
	STN-42	28.5 – 29.9	1.4
		30.5 – 32.5	1.2
		32.5 – 34.5	1.9
		34.5 – 36.5	1.8
		36.5 – 38.5	0.0
		38.5 – 40.5	1.8
		42.0 – 44.0	1.9
		44.0 – 46.0	1.9
		46.0 – 48.0	1.9
		48.0 – 49.0	1.0
	STN-44	22.0 – 24.0	1.0
		37.0 – 39.0	2.0
		52.0 – 54.0	1.9
	STN-47	22.0 – 24.0	2.0
		35.0 – 37.0	2.0
		50.0 – 52.0	1.7

Table 5. Summary of undisturbed Shelby Tube Samples

Stack Location	Boring Number	Sample Depth (feet)	Recovery Length (feet)
West	STN-V9	33.0 – 35.0	2.0
		37.0 – 39.0	2.0
		39.0 – 41.0	2.0
	STN-V10	31.0 – 33.0	2.0
		36.0 – 38.0	2.0

5.5. Vane Shear Testing

In June 2009, two vane shear borings were advanced on the western dike between Pond 2B and the Stilling Pond (see attached layout in Appendix C). Three vane shear tests were conducted within each boring. Undisturbed Shelby tube samples were performed at selected depths. The field testing was conducted in an attempt to better define the in situ undrained shear strengths of the sedimented gypsum-fly ash materials. A summary of the vane shear testing results are provided in Table 6 below.

Table 6. Summary of Vane Shear Testing

Boring Number	Soil Horizon	Test Elevation (feet)	Maximum Measured Torque (lbs)	Vane Diameter Size (inch)	Undrained Shear Strength (psf)	Residual Shear Strength (psf)	Sensitivity
V-9	Sedimented Gypsum – Fly Ash	622.0	>600	2.031	Unknown	Unknown	NA
	Sedimented Gypsum – Fly Ash	618.0	295	2.031	1,464	496	2.95
	Weak Sedimented Gypsum – Fly Ash	616.0	220	2.031	1,092	372	2.93
V-10	Weak Sedimented Gypsum – Fly Ash	624.0	155	2.031	769	323	2.38
	Weak Sedimented Gypsum – Fly Ash	621.0	145	2.031	695	99	7.00
	Weak Sedimented Gypsum – Fly Ash	619.0	115	2.492	297	78	3.83

5.6. CPT Testing

Four Cone Penetration Test (CPT) borings were performed along the west embankment slope of the Gypsum Stack between Pond 2B and the Stilling Pond, at the locations identified on the boring layout in Appendix C. Two of the borings (CPT-10 and CPT-12) were located along the top of the recently constructed toe buttress (630' in elevation) and two (CPT-11 and CPT-13) were located along the stability bench (655' in elevation). The CPT testing was performed in general accordance with ASTM Standard D 5778 "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils." The cone test involved advancing an integrated electronic seismic piezocone within the overburden materials to measure tip resistance, sleeve friction and dynamic pore pressure at roughly one-inch intervals. In addition, pore pressure dissipation testing and seismic testing was performed at selected intervals.

As noted on the boring layout, CPT borings were performed near previously drilled geotechnical sample borings. The previous sample logs were used to predict the depths for each soil horizon as the cone penetration testing was being performed. It should be noted that one of the CPT borings, CPT-13, refused within the Cast Gypsum - Fly Ash soil; however the other three borings were continued down into the underlying Weak Sedimented Gypsum -Fly Ash. Due to crushed stone on the toe buttress, CPT-10 and CPT-12 required pre-augering prior to beginning any tests. The pre-auger depths were between 2 to 4 feet. The two tests performed on the 655 bench elevation experienced spikes in pore water pressure around 641 feet in elevation and the lower bench experienced similar spikes in pore water pressure between the 605 and 610 feet in elevations. This most likely indicates the in-situ phreatic water level at the time of the testing.

The CPT borings also provided a continuous readout of the undrained shear strength parameters and effective phi angles for each soil horizon. The strength values were plotted versus depth along with the Vane Shear test results to determine an empirical N_{kt} factor for the given soil profile. A more in depth discussion explaining the methods/procedures used for the CPT testing have been attached to this report along with the CPT logs in Appendix G. The shear strength values for each soil horizon were then determined based on the graphical data from the CPT results using a two-thirds rule. The two-thirds rule implies that approximately two-thirds of the data points fall above the chosen shear strength envelope and one-third fall below the chosen value. The results of the undrained shear strength values and effective phi angles calculated from the CPT borings are presented below in Table 7 and the graphical results are located in Appendix G.

Table 7. Average Shear Strengths per Soil Horizon

Soil Horizon	Undrained Shear Strength (psf)	Effective Phi Angles (degrees)
Cast Gypsum – Fly Ash	3,384	42
Weak Sedimented Gypsum – Fly Ash	2,544	34
Fat Clay	3,120	29

The penetration and vane shear data were used as a means to interpret the consistency of the gypsum-fly ash materials. Significant engineering judgment is required in interpreting this shear data as the material exhibits a dilatency during the application of shearing strain. The dilatency results in the development of negative pore pressures at the tip of the vane thus affecting the undrained shear strength by some unknown amount.

5.7. Rock Core Samples

A total of fourteen borings were extended to auger refusal and four of the borings (STN-31, STN-37, STN-40, and STN-47) were extended approximately ten feet into the underlying bedrock. The apparent top of rock elevation ranged from 585.3 feet in boring STN-40 to 618.9 feet in boring STN-35.

The rock core samples collected from the geotechnical borings show the underlying bedrock to consist of limestone with dolomitic zones. The limestone was described as gray to dark gray in color, fine to micro-crystalline grained, thin to medium bedded, hard, with shale stringers and occasional argillaceous zones. The bedrock encountered at the site correlates well with the Sequatchie Formation, Nashville Group, and Stones River Group described in Section 1.2. A detailed description of the rock core samples, including the base of weathered rock is presented on the geotechnical logs in Appendix B and on the geotechnical cross sections in Appendix C.

6. Field Instrumentation

A total of eighteen borings were instrumented with slope inclinometers and/or piezometers to monitor possible slope movement and determine the pore water pressures and groundwater levels at selected locations within the Gypsum Stack. The current results of the instrumentation are summarized in the following sections.

6.1. Piezometers

A total of fifteen borings were instrumented with vented piezometers to monitor pore pressures within the Gypsum Stack at the specific depths and locations shown on the piezometer installation records in Appendix B and on the instrumentation plan in Appendix C.

In general, the casagrande style piezometer tip (2 feet in length) was surrounded by a five-foot thick sand filter and the five-foot slotted PVC screen was surrounded by a sand filter pack which extended approximately two feet above the upper most opening and two feet below the bottom of the screen. After, placing the sand filter pack, a two-foot thick layer of bentonite was placed on top of the sand to seal the filter zone. Next, the annulus of the borehole was grouted up to the surface with a bentonite and Portland cement mix. Finally, at the ground surface the piezometer was protected with a steel flush mount or riser type protective cover and a concrete pad measuring 3 feet x 3 feet x 1.5 feet was installed around the piezometer well. Three of the borings (STN-45, 48, 50) were instrumented with nested piezometers to monitor two separate material types within the Gypsum Stack. The piezometers zones monitored during this exploration are listed below in Table 8 and the piezometer installation records have been included in Appendix B. The graphical results of the piezometer readings have been included in Appendix H.

Table 8. Summary of Piezometer Sensing Zones

Stack Location	Soil Horizon	Boring Number	Sensing Depth Interval (feet)	Sensing Elevation Interval (feet)
South	Sedimented Gypsum – Fly Ash	STN-28	34.6 - 36.6	616.7 - 614.7
	Cast Gypsum – Fly Ash	STN-29	4.5 - 6.5	619.4 - 617.4
East	Sedimented Gypsum – Fly Ash	STN-32	16.5 - 18.5	639.7 - 637.7
	Sedimented Gypsum – Fly Ash	STN-33	19.0 - 21.0	621.2 - 619.2
	Cast Gypsum – Fly Ash	STN-35	20.3 - 22.3	635.8 - 633.8
	Fat Clay	STN-36	12.2 - 14.2	619.7 - 617.7
West	Sedimented Gypsum – Fly Ash	STN-39	45.7 - 47.7	609.5 - 607.5
Stilling Pond	Sedimented Gypsum – Fly Ash	STN-42	35.5 - 37.5	623.0 - 621.0
	Sedimented Gypsum – Fly Ash	STN-43	48.0 - 50.0	624.7 - 622.7
	Cast Gypsum – Fly Ash	STN-45 U	13.0 - 15.0	640.2 - 642.2
	Weak Sedimented Gypsum – Fly Ash	STN-45 L	38.0 - 40.0	617.2 - 615.2
	Cast Gypsum – Fly Ash	STN-48 U	14.5 - 16.5	640.4 - 638.4
	Cast Gypsum – Fly Ash	STN-48 L	45.0 - 47.0	609.9 - 607.9
	Weak Sedimented Gypsum – Fly Ash	STN-49	34.0 - 36.0	621.0 - 619.0
	Sedimented Gypsum – Fly Ash	STN-50 U	18.0 - 20.0	636.9 - 634.9
	Cast Gypsum – Fly Ash	STN-50 L	31.0 - 33.0	623.9 - 621.9
	Fat Clay	CPT-10	21.0 - 26.0	608.0 - 603.0
	Fat Clay	CPT-12	16.0 - 21.0	613.0 - 608.0

During the construction of Work Plan 5, piezometer wells STN-42, STN-48U and STN-49 were destroyed.

At the time of this report, a total of nine monitoring trips have been performed over the period from January 22, 2009 through October 1, 2009. A summary table of the piezometer readings is presented in Appendix H.

The piezometers were also tested in the field to determine the horizontal in-situ hydraulic conductivity of the underlying Cast Gypsum-Fly Ash, Sedimented Gypsum-Fly Ash, and Weak Sedimented Gypsum-Fly Ash soil horizons. The tests were performed in general accordance with ASTM D 4044 "Test Method for (Field Procedure) for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers" and the results are shown in Table 9.

Table 9. Summary of Instantaneous Change in Head Test Results

Soil Horizon	Location	Ratio (K_h/K_v)	1	3	15	25	50	100
		Boring Number	K_h (cm/sec)	K_h (cm/sec)	K_h (cm/sec)	K_h (cm/sec)	K_h (cm/sec)	K_h (cm/sec)
Cast Gypsum – Fly Ash	South	STN-29	NA	$1.70x10^{-4}$	$1.48x10^{-4}$	NA	NA	NA
	East	STN-35	NA	$5.70x10^{-4}$	$4.96x10^{-4}$	NA	NA	NA
	Stilling Pond	STN-45U	NA	$2.27x10^{-4}$	$1.98x10^{-4}$	NA	NA	NA
		STN-48U	NA	$1.51x10^{-4}$	$1.31x10^{-4}$	NA	NA	NA
		STN-48L	NA	$3.36x10^{-4}$	$3.06x10^{-4}$	NA	NA	NA
		STN-50L	NA	$1.32x10^{-4}$	NA	NA	NA	NA
Sedimented Gypsum – Fly Ash	South	STN-28	$2.02x10^{-4}$	$2.26x10^{-4}$	NA	$2.97x10^{-4}$	$3.21x10^{-4}$	$3.47x10^{-4}$
	East	STN-32	$5.11x10^{-4}$	$5.14x10^{-4}$	NA	$7.53x10^{-4}$	$8.15x10^{-4}$	$8.83x10^{-4}$
		STN-33	$2.19x10^{-4}$	$2.51x10^{-4}$	NA	$3.12x10^{-4}$	$3.35x10^{-4}$	$3.59x10^{-4}$
	West	STN-39	$3.03x10^{-5}$	$3.43x10^{-5}$	NA	$4.30x10^{-5}$	$4.65x10^{-5}$	$5.04x10^{-5}$
	Stilling Pond	STN-43	$1.02x10^{-4}$	$1.26x10^{-4}$	NA	$1.41x10^{-4}$	$1.51x10^{-4}$	$1.61x10^{-4}$
		STN-50U	NA	$1.09x10^{-4}$	NA	$1.40x10^{-4}$	$1.52x10^{-4}$	$1.65x10^{-4}$
Weak Sedimented Gypsum – Fly Ash	Stilling Pond	STN-45L	$3.08x10^{-4}$	$1.84x10^{-4}$	NA	$4.27x10^{-4}$	$4.58x10^{-4}$	$4.92x10^{-4}$
		STN-49	$1.53x10^{-4}$	$1.47x10^{-4}$	NA	$2.14x10^{-4}$	$2.32x10^{-4}$	$2.51x10^{-4}$
Weak Sedimented Gypsum – Fly Ash	Stilling Pond	STN-42	$1.84x10^{-4}$	$2.34x10^{-4}$	NA	$2.75x10^{-4}$	$2.98x10^{-4}$	$3.22x10^{-4}$

* The k_h/k_v ratios presented above were based on reported values and a comparison between in-situ permeability testing and laboratory Falling Head Permeability testing. The results of the comparison are included in Appendix G.

6.2. Slope Inclinometers

Three of the borings (STN-31, STN-37, and STN-40) were instrumented with 2.75 inch OD slope inclinometers at locations where evidence of slope instability were observed. At this time, a total of eight readings have currently been performed at the site and were conducted from February 27, 2009 to September 29, 2009.

As of September 29, 2009, SI-31 shows a cumulative displacement in the upslope direction at approximately 667 feet in elevation (about 6 feet below the ground surface). At this time, the total recorded displacement is approximately 0.9 inches. This slope inclinometer was installed along the north side of Pond 2B on the divider dike. The SI casing was set at this location due to the development of tension cracks along the dike after the contractor placed material to reinforce the divider dike. The movement observed within this inclinometer is most likely a result of surface drying and desiccations. SI-37 also shows some cumulative down slope displacement between 626.9 feet (ground surface) and 615.5 feet in elevation.

However, at this time the total recorded displacement is approximately 0.1 inches and is centered at approximate elevation 623.0 feet. At this time, it is recommended to continue to monitor the slope within this area. SI-40 shows only initial settling of the backfill on the order of 0.75 inches. After settling, little to no movement has been observed since the first comparison reading on April 22, 2009.

The displacement curves for the slope inclinometers are presented in Appendix E and the maximum displacement observed for each of the slope inclinometers is plotted on the respective cross section in Appendix C.

6.3. Measured Water Levels

A total of fifteen piezometers were installed during the geotechnical exploration between January 12, 2009 and February 22, 2009. Since installation, each piezometer has been read approximately once a month and the result input into a spread sheet to monitor the pore pressures readings and establish a baseline reading for the phreatic water level. A summary of the average water elevation in each piezometer is presented in Table 10.

Table 10. Average Piezometer Water Elevations

PZ Number	Number of Readings	Ground Elevation (feet)	Average Water Elevation (feet)	Average Depth to Water (feet)
STN-35	11	656.1	642.5	13.6
STN-32	10	656.2	642.5	13.7
STN-33	10	640.2	629.0	11.2
STN-28	10	651.3	634.3	17.0
STN-29	10	623.9	622.7	1.2
**STN-36	9	631.9	633.1	-1.2
STN-39	11	655.2	631.3	23.9
STN-42	8	659.0	640.1	18.9
STN-43	9	672.7	655.8	16.9
STN-45 Lower	13	655.2	635.6	19.6
STN-45 Upper	11	655.2	642.1	13.1
STN-48 Lower	11	654.9	634.9	20.0
STN-48 Upper	10	654.9	640.7	14.2
STN-49 Lower	10	654.9	641.3	13.6
*STN-49 Upper	10	654.9	-	-
STN-50 Lower	12	654.9	640.2	14.7
STN-50 Upper	12	654.9	641.3	13.6
CPT-10	2	630.0	613.3	16.8
CPT-12	2	630.0	619.3	10.7

Notes:

* Upper well was dry
** Artesian flow

The phreatic levels measured in the piezometers were also compared to the normal pool elevation for Pond 3, 2B, and 2A in an attempt to develop a hydraulic pattern between the pool elevation and the piezometer levels. However, due to the on going construction along the west embankment the water levels for Ponds 2A and 2B were drawn down and maintained at a minimum level near elevation 651.5 feet.

7. Laboratory Testing

7.1. Introduction

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

7.2. Testing of Standard Penetration Test (SPT) Samples

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

7.2.1. Natural Moisture Content

The gypsum by-product material at the Widows Creek facility consists of dehydrate calcium sulfate ($CaSO_4$-$2H_20$) which contains two attached molecules of water. Because of the naturally chemically bound water, the apparent moisture content of the gypsum can vary with drying temperature. In order to ensure that this chemically-bound water is not expelled from the sample during natural moisture content testing, an oven temperature of 40° C was selected. Using the typical temperature of 110° C (ASTM Standard D-2216) would have yielded inaccurate moisture contents due to some loss of the naturally bound water. Following the 40° C test, the samples were then dried at an oven temperature of 200° C to completely expelling all bound water from the sample. The 40° C and 200° C values were then compared to determine the change in apparent moisture content. Natural moisture content determinations were performed on fifty-three (53) undisturbed samples and one hundred and forty-seven (147) split-spoon samples.

The quantity of the gypsum within the gypsum-fly ash materials (Soils 1, 2, and 4) can then be predicted using the apparent difference in moisture content between the 40° C and 200° C moisture contents. Assuming all gypsum is in the dehydrate form, a sample comprised of 100% gypsum would have a theoretical change in moisture content of 20.92%. This is assuming all non-chemically bounded water is completely expelled from the sample at 40° C and all chemically-bound water is expelled from the sample at 200° C. This information is then used to estimate the percent of gypsum in the samples.

A detailed summary showing each split spoon sample has been included Appendix I. Based on the moisture content test results, the percent gypsum varies from 66% to 82%, with an average of 73%, equating to a non-gypsum component of 27%.

7.2.2. Chemical Composition

Four disturbed samples of Cast Gypsum – Fly Ash, two disturbed samples of Sedimented Gypsum – Fly Ash and two disturbed samples of Weak Sedimented Gypsum – Fly Ash were sent to TVA's Central Laboratory Services in Chattanooga for laboratory determination of chemical composition. This information was used to determine the percent dry weight of fly ash (acid insoluble), gypsum, un-reacted limestone, and calcium sulfite. The results of the TVA chemical analyses are depicted in Table 11 below.

Table 11. Chemical Composition of Gypsum Fly Ash

Boring	Test Interval (feet)	Soil Horizon	Chemical Composition (% of dry weight basis)				Specific Gravity
			Insoluble (Fly Ash)	Gypsum	Unreacted Limestone *	Calcium Sulfite	
STN-28	1.5-6.0	Cast Gypsum-Fly Ash	46.0	36.0	18.0	0.0	2.48
	14.0-20.0	Sedimented Gypsum-Fly Ash	25.6	62.5	11.9	0.0	2.42
STN-32	6.0-10.5	Cast Gypsum-Fly Ash	18.0	62.0	20.0	0.0	2.44
	13.5-19.5	Sedimented Gypsum-Fly Ash	18.5	46.6	34.8	0.1	2.49
STN-38	6.0-12.0	Cast Gypsum-Fly Ash	28.6	55.8	15.4	0.2	2.44
	21.5-26.0	Weak Sedimented Gypsum-Fly Ash	31.6	47.5	14.1	6.9	2.45
STN-45	10.5-16.5	Cast Gypsum-Fly Ash	19.7	58.4	21.5	0.5	2.45
	34.5-40.5	Weak Sedimented Gypsum-Fly Ash	21.0	51.6	23.4	4.0	2.47
Average			26.1	52.5	19.9	1.5	2.46
* Includes Dolomite CaCO3, MgCO3, and Calcium Carbonate CaCO3							

The percent gypsum for the Cast Gypsum – Fly Ash varies from 36% to 62% with an average of 53%, equating to a non-gypsum component of 47%. Likewise, the percent gypsum for the Sedimented Gypsum Fly - Ash varies from 47% to 62% with an average of 55% and the percent gypsum for the Weak Sedimented Gypsum – Fly Ash varies from 21% to 32% with an average of 27%. Based on the results of the chemical composition testing, it appears the percent gypsum drops significantly within the weak soil horizon as compared to Cast and Sedimented Gypsum – Fly Ash. This may also explain why the strength parameters for the weak horizon are so much different when compared to the sedimented horizon (See Section 8.4).

Furthermore, the percent gypsum calculated based on the 40°C and 200°C moisture contents appears to be higher than those calculated by TVA. This may be because the percent gypsum calculated from the natural moisture contents is based on a maximum theoretical moisture change of 20.92%, in which case all chemically-bound water would be expelled. These values could differ due to the possibility that all of the free-standing water was not expelled at the 40° C temperature, yielding a higher percent moisture change between the two temperatures.

7.2.3. Particle Size Distribution and Fines Content

Particle size distribution tests were conducted on ten samples recovered from the standard penetration test borings. In general, three to four SPT samples of similar soil type were combined and the particle size distribution test was performed on the composite sample. The test was conducted in accordance with ASTM D 422, "Particle Size Analysis of Soils." The gradation tests were performed on the predominant soil types to supplement the visual classifications made by the engineer/geologist in the field. The gradation curves from the particle size distribution tests are presented in Appendix F.

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

Fifteen of the twenty-four sample borings drilled for the Gypsum Stack included undisturbed (Shelby) tube soil sampling with a 3-inch diameter fixed-head piston sampler. The undisturbed samples were obtained within all four of the primary soil horizons and transported back to Stantec's laboratory in Lexington, Kentucky. In the lab the samples were extruded and trimmed into six-inch long specimens. Lab personnel performed visual inspections of the soil samples, unit weights (wet and dry), and natural moisture determinations for each six-inch specimen prior to submitting a summary report of the extruded specimens to the geotechnical engineer.

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

7.3.1. Dry and Wet Unit Weights

A total of fifty-two (52) unit weights were determined from the Shelby tube samples. Both the wet unit weight and dry unit weight were determined for each sample, as well as the moisture content of the sample. As previously mentioned, the characteristics of the gypsum-fly ash material varied significantly around the stack depending on the plan location and depth of the sample. Therefore, average unit weights for the north, south, east, and west embankment slopes were calculated and are presented in Table 12.

Table 12. Average Unit Weight per Location

Boring Location	Horizon Center Elevation (feet)	Natural Moisture Content (%) @ 40° C	Average Total Unit Weight (pcf)	Average Dry Density (pcf)	Average Void Ratio, e	Average Porosity, n
North Embankment	646.8	24.7	112.3	90.7	0.9	0.5
South Embankment	622.0	24.9	112.4	90.3	0.9	0.5
East Embankment	614.3	29.5	117.2	90.8	0.9	0.5
West Embankment	618.9	35.1	110.7	84.2	1.0	0.5

Note: North includes STN-34 and STN-35. South includes STN-28 and STN-29. East includes STN-32 and STN-33. West includes STN-38, STN-39, STN-41, STN-42, STN-44, and STN-47.

Stantec also reviewed the average unit weights per soil type, and the results are presented in Table 13. With the exception of one sample in STN-41 (65.8 pcf), the dry densities obtained from Stantec's exploration correlate well with those reported in the 2004 Ardaman report. The results of the unit weight tests are presented in Appendix F.

Table 13. Average Unit Weight per Soil Type

Soil Horizon	Horizon Center Elevation (feet)	Natural Moisture Content (%) @ 40° C	Average Total Unit Weight (pcf)	Average Dry Density (pcf)	Average Void Ratio, e	Average Porosity, n
Sedimented Gypsum-Fly Ash	613.4	28.2	111.5	89.9	0.9	0.5
Cast Gypsum-Fly Ash	637.2	31.6	109.9	84.7	1.0	0.5
Fat Clay	624.2	28.7	121.1	94.0	0.8	0.4
Weak Sedimented Gypsum-Fly Ash	614.9	40.1	111.3	80.8	1.1	0.5

7.3.2. Consolidated-Undrained (CU) Triaxial Testing

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

Table 14 summarizes the results of the triaxial compression tests. The average effective angle of internal friction (ϕ') and average effective cohesion (c') was also determined for each soil type. The average angle of internal friction for the Sedimented Gypsum-Fly Ash, Cast Gypsum-Fly Ash, Fat Clay, and Weak Sedimented Gypsum-Fly Ash was 41.2°, 40.4°, 24.7 and 42.5,° respectively. The gypsum-fly ash materials had an apparent cohesion of 0 and the Fat Clay had an apparent cohesion of 380 psf. The effective phi angle for the Cast Gypsum-Fly Ash ranged from a low of 38.7° to a high of 42.0°. Likewise, the Sedimented Gypsum-Fly Ash phi angle ranged from a low of 35.8° to a high of 43.9° and the Weak Sedimented Gypsum-Fly Ash phi angle ranged from 41.7° to 43.2°. A summary table listing the results of each sample tested has been included in Appendix I and the lab test results are presented in Appendix F.

Table 14. Average CU Triaxial Test Results per Soil Type

Soil Horizon	Total Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	c'	ϕ' (deg.)
Sedimented Gypsum-Fly Ash	111.7	82.7	0	41.2
Cast Gypsum-Fly Ash	112.9	78.2	0	40.4
Fat Clay	123.3	96.1	380	24.7
Weak Sedimented Gypsum-Fly Ash	107.9	78.1	0	42.5

7.3.3. Unconsolidated-Undrained (UU) Triaxial Testing

Unconsolidated-Undrained triaxial compression tests were performed on selected undisturbed Shelby tube samples obtained during the vane shear testing. Two tests were performed on the Sedimented Gypsum-Fly Ash and three were performed on Weak Sedimented Gypsum-Fly Ash, the results are presented in Table 15.

Table 15. Summary of Unconsolidated – Undrained Triaxial Testing

Boring Number	Soil Horizon	Sample Interval (feet)	Confining Stress (psi)	UU Triaxial Strength	
				(psf)	(tsf)
V-9	Sedimented Gypsum – Fly Ash	33.0-33.5	30	20,884	10.44
V-9	Sedimented Gypsum – Fly Ash	37.1-37.6	35	29,033	14.52
V-9	Weak Sedimented Gypsum – Fly Ash	39.0-39.5	35	1,129	0.56
V-10	Weak Sedimented Gypsum – Fly Ash	31.1-31.6	25	10,205	5.10
V-10	Weak Sedimented Gypsum – Fly Ash	36.7-37.2	30	1,656	0.83

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

7.3.4. Falling Head Permeability Testing

Falling head permeability tests were performed on four undisturbed Shelby tube samples. After extrusion, each sample was mounted in a triaxial-type permeameter and tested in accordance with ASTM Standard D 5084, using de-aired gypsum saturated water. The results of the permeability tests can be found in Appendix F and are summarized below in Table 16.

Table 16. Summary of Falling Head Permeability Test Results

Boring	Soil Horizon	Test Interval (feet)	Sample Center Elevation (MSL)	Initial Conditions				Coefficient of Permeability Kv (cm/sec)
				Dry Density (pcf)	Moisture Content (%) @ 20° C	Void Ratio, e	Degree of Saturation (%)	
STN-28	Sedimented Gypsum-Fly Ash	39.5-41.5	610.8	82.4	31.2	0.787	93.5	4.47×10^{-5}
STN-44	Cast Gypsum-Fly Ash	22.0-24.0	632.8	93.0	21.5	0.658	80.6	3.22×10^{-5}
	Sedimented Gypsum-Fly Ash	52.0-54.0	602.8	57.0	70.0	1.726	100.9	2.02×10^{-6}
STN-47	Cast Gypsum-Fly Ash	35.0-37.0	619.5	92.8	33.3	0.668	123.6	2.68×10^{-6}

The average coefficient of permeability determined from the falling head permeability test for the Cast Gypsum - Fly Ash is 1.74×10^{-5} cm/sec and for the Sedimented Gypsum - Fly Ash is 2.34×10^{-5} cm/sec. The values presented above correspond well with those found in the 2004 Ardaman report.

8. Engineering Analysis

8.1. General

Geotechnical engineering analyses included evaluations of strength and permeability parameters, seepage analyses, and slope stability analyses. Prior to beginning the analyses, the geotechnical data and fifteen initial cross sections were selected. The geometry of the existing embankments slopes and soil horizons were approximated using current and historical information. Once the geometry of the sections was determined, each section was reviewed and evaluated to determine the most critical reaches around the Gypsum Stack. The criteria for selecting the critical sections was based on the steepness of slopes, the geometry of the sections, height/location of phreatic surface, and soil conditions. Based on this evaluation, five critical cross sections (Section A, D, F, H, and K) were selected for seepage and slope stability analyses. Results of the analyses and evaluations are summarized in the following paragraphs and output files/cross sections for each reach is included in Appendix G. The plan location for each cross section is identified on the geotechnical drawings included in Appendix C.

It should be noted that construction records indicating the methods used to construct the Gypsum Stacks; as-built configurations, etc. were not available for review. In addition, the variable nature of the historical and current strength data shows some signs of inconsistencies in the construction of the stack. As a result, generalizations in soil parameters and slope geometry were needed to construct the seepage and stability models.

8.2. Soil Horizons

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

- *Fat Clay*: This material represents the clay soils which were encountered within the clay starter dikes and the residual foundation soils. Based on historical information the interior slopes for the clay starter dike were to be constructed at a 2:1 and the exterior slopes were to be constructed at a 3:1.

- *Cast Gypsum-Fly Ash*: This represents material encountered during the field exploration above the original starter dike which appears to have been used for raising of the perimeter dike. The material consists of a mixture of gypsum and fly ash which has been mechanically excavated from the rim ditch and allowed to dry and consolidate under its own weight. Based on historical information the interior slopes were to be constructed at a 2:1 and the exterior slopes were constructed at a 2.5:1. It should be noted the stability sections constructed in Appendix G have been updated based on the boring information. Therefore, the interior/exterior outslopes and soil horizons depicted on the sections may differ from TVA's original design template.

- *Sedimented Gypsum-Fly Ash*: This represents the hydraulically placed gypsum-fly ash that is contained by the original starter dike and subsequent cast gypsum-fly ash perimeter dikes. It was primarily encountered upstream of the starter dike and below and upstream of the perimeter dike.

- *Weak Sedimented Gypsum-Fly Ash:* This horizon represents an original (Phase 1 Pond) sluiced material which appears to be concentrated around the west interior slopes and between 600 feet and 625 feet in elevation. The results of the exploration indicate this material is generally very soft and exhibits relatively low shear strength.

8.3. Seepage Analyses

8.3.1. SEEP/W Model

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

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

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

8.3.2. Boundary Conditions

Steady-state seepage was assumed for the analysis, with static water levels on both the upstream and downstream side of the embankments, where applicable. The boundary conditions modeled for each section are presented below in Table 17.

Table 17. Boundary Conditions

Stability Section	Upper Boundary Condition	Upper Boundary Condition Elevation (feet)	Lower Boundary Condition	Lower Boundary Condition Elevation (feet)
Section A	Rim Ditch	670.0	None	NA
Section D	Pond 2A	659.4	None	NA
Section F	Pond 3	668.6	None	NA
Section H	Pond 3	668.6	None	NA
Section K	Pond 2B	654.5	Stilling Pond	614.0

The boundary conditions used in the SEEP/W analysis for the ponds described above were modeled as Total Head equal to the given pool elevation. For sections were the pool limits were just beyond the cross section or the ground line was above the recorded pond elevation, a Total Head vertical boundary line equal to the pond elevation was input into the model. For this scenario, the hydraulic head at each node was constant with depth and equal to the pool elevation on that side of the embankment. At other locations along the ground surface were potential seepage might occur a Total Flux condition was modeled and potential seepage reviewed. At locations where a chimney drain was located (toe drain or slope drain) a Total Head equal to the pipe invert elevation was modeled. The horizontal boundary at the base of the model (located within the limestone bedrock) was modeled as a seepage barrier, with no vertical flow across the boundary nodes.

8.3.3. Seepage Properties

For each modeled cross section of the dike, a representative subsurface profile was compiled based on boring logs, available drawings, and project history. Material properties were estimated based on available laboratory data and typical values for similar soils. Material properties used in the seepage analysis are summarized in Table 18.

Table 18. Seepage Parameters

Soil Horizon	Saturated K_v (cm/sec)	Ratio k_h/k_v	Volumetric Water Content	
			Saturated (%)	Residual (%)
Cast Gypsum - Fly Ash	1.74×10^{-5}	15	34.6	2.0
Sedimented Gypsum - Fly Ash	2.02×10^{-6}	100	38.0	2.0
Weak Sedimented Gypsum - Fly Ash	2.02×10^{-6}	100	47.0	2.0
Fat Clay	6.23×10^{-7}	9	24.8	3.0
Crushed Stone	2.54×10^{-2}	1	NA	NA
Sand Drain	2.54×10^{-3}	1	NA	NA

Significant engineering judgment is needed to select appropriate hydraulic properties for earth material. Unlike other key properties, hydraulic conductivity can vary over several orders of magnitude for a range of soils, often with substantial anisotropy for seepage in horizontal versus vertical directions. Laboratory test samples often do not represent important variations within a larger soil deposit. For the Gypsum Stack, an iterative parametric calibration was used to arrive at final seepage design parameters. The results from trial SEEP/W simulations were compared to field data (measured piezometric levels). The material parameters were then varied until the solutions reasonably matched the field data for the representative cross sections. The final set of parameters identified in Table 17 were a result of the piezometer calibration process which is discussed below in Section 8.3.4.

The ratio of horizontal hydraulic conductivity (k_h) to vertical hydraulic conductivity (k_v) was estimated based on the known depositional environment of the given material and slug test results within the various soil horizons. An isotropic material (sands and gravels) would have $k_h/k_v = 1$, while deposits of horizontally layered soils (silt, gypsum-fly ash) might have values as high as $k_h/k_v = 100$. For the Gypsum Stack, a ratio of 100 was assumed for the sedimented gypsum-fly ash materials and a ratio of 15 was assumed for the cast gypsum-fly ash material. A more modest value of $k_h/k_v = 9$ was assumed for the Fat Clay.

The governing equations in SEEP/W are formulated to consider seepage through unsaturated soils. In order to accomplish this, the SEEP/W model implements a model based on two curves, a hydraulic conductivity function and a volumetric water content function. Three parameters are needed to define this behavior: the saturated hydraulic conductivity, saturated water content, and residual water content (water content of air dried soil). Of these, only the residual water was assumed based on typical values for similar soils, the remaining values were obtained from laboratory tests.

8.3.4. Comparison to Field Observations

After the initial seepage parameters were estimated, results from the SEEP/W model were compared to the pore water pressures measured in piezometers installed along the Gypsum Stack. Data from nine piezometers along all five critical cross sections were used in this evaluation. Nodes were placed in the model at the same location as the piezometer tip and then the total predicted head at the node was compared to the corresponding piezometer reading.

After reviewing/comparing the results for all nine piezometers, the material properties in each modeled cross section were varied (if necessary) until a reasonable match was obtained between the predicted SEEP/W elevation and the actual field piezometer readings. The comparison between the field piezometer readings and the predicted SEEP/W values at each piezometer are plotted in Figure 16. The maximum difference between the predicted and field measured is 5.5 feet (STN-39), while most differ by less than 2 feet. Given the typical differences between the modeled cross section and the unknown the subsurface soil conditions, Stantec believes this difference is acceptable.

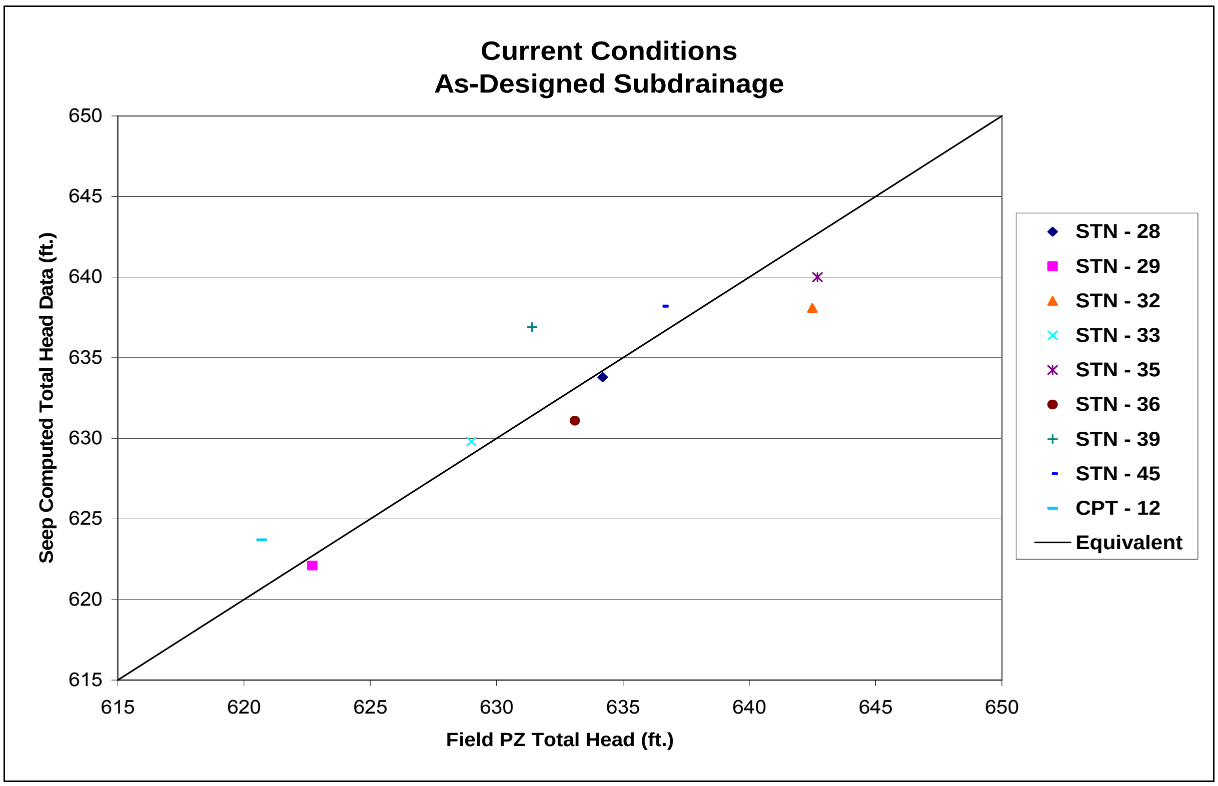


Figure 15. Comparison between Field Piezometer Readings and Pore Water Pressures predicted in SEEP/W Model

The predicted SEEP/W values presented above, assume that the planned subdrains have been installed and are functioning properly. Every section evaluated, excluding Section K, had toe drains and slope drains installed according to the current inspection report. The seepage was also modeled without subdrains; however the piezometer data indicates that the model including subdrains is a better reflection of the actual field conditions.

8.3.5. Results from Seepage Analyses

SEEP/W plots for the five critical cross sections are presented in Appendix G. The plots show the finite element mesh, material zones, and boundary conditions used in each analysis. The results are depicted in contour plots of total head, pore water pressure, and seepage gradients. For the slope stability analyses discussed in Section 8.5.2, the seepage line generated from the SEEP/W model was input as the phreatic surface. The seepage gradients were also assessed for maximum exit gradients and the potential for soil piping as discussed in Section 8.3.6 below. These results are presented in Section 9.1.

The phreatic surface (groundwater table or line of zero pore water pressure) is shown on the plots in Appendix G. In SEEP/W, the location of the phreatic surface is found by interpolation between positive pore water pressures in the upper areas of saturated soil and negative or suction pore pressures in the unsaturated soil zone above. In the SEEP/W formulation, seepage flows are tracked in both the saturated and unsaturated zones. Hence, the top flow line in the SEEP/W results will be above the phreatic line. In more traditional seepage analyses, where unsaturated flows are ignored, the top flow line and the phreatic surface coincide. Hence, while the more complete unsaturated flow formulation in SEEP/W gives a reasonable prediction of the phreatic surface location and shape, the results are often different than would be obtained with a solution that considers only saturated flow.

8.3.6. Critical Exit Gradients

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 things such as: the type of foundation soils, embankment materials, embankment construction, compaction, pipe penetration, etc. 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.

In an effort to determine if any of the critical sections have potential piping issues, Stantec reviewed each seepage analysis based on the following Factors of Safety for piping. 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 it is defined as the vertical gradient in the soil at the exit point. 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} is the submerged unit weight of the soil, γ_w is the unit weight of water, G_s is the specific gravity of the soil particles, and e is the void ratio. For nearly all soils, the critical gradient is between about 0.6 and 1.4, with a typical value near 1.0.

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 $i=i_{crit}$.

8.4. Strength Parameters

The static stability of the Gypsum Stack at the Widows Creek Fossil Plant was evaluated using the limit equilibrium slope stability methods. The soil parameters used in these stability analyses were established as follows.

The Gypsum Stack was first constructed with an initial perimeter dike in the early 1980's. In 1994, the stack was horizontally expanded as part of second phase to increase capacity. It is assumed that excess pore pressures have dissipated and steady state seepage conditions have developed within the dike. In addition, the current analyses will focus only on static conditions (no earthquake or other dynamic loads). For these conditions, only soil unit weights and drained strength parameters (c' and ϕ') are needed. If stabilizing berms or other modifications to the dike cross section are built, then undrained, total stress stability analyses will be needed to assess stability during construction.

The soil parameters used for the dikes and existing foundation materials were derived using both current and historical data from laboratory consolidated undrained triaxial tests, cone penetration data, standard penetration test data and classification test data. In addition, the strength parameters selected were further refined by comparisons with the strength parameters used in the historical design reports reviewed.

To select the representative shear strengths for each soil horizon, 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 envelopes were conservatively fit through the data. In general, the selected strength parameters represent a failure envelope where about two-thirds of the test data falls above the envelope.

8.4.1. Drained Soil Parameters

Excess (or deficient) pore water pressures, generated by changes in mean stress or shearing stress, will dissipate under static, long term conditions. Pore pressures within a soil can then be computed assuming hydrostatic conditions or from a solution for steady state seepage. As long as the distribution of pore pressure within the cross section can be quantified, effective stresses can be computed and the drained shear strength (S_d) of the soil can be determined from effective stress strength parameters (c' and ϕ'):

$$S_d = c' + \sigma' \tan\phi' \qquad \textbf{Eqn. 3}$$

Uncemented soils exhibit no strength at σ' = 0, corresponding to c' = 0. In the case of unsaturated fine grained soils, 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.

The Mohr-Coulomb failure envelope for normally consolidated, saturated clays exhibits c' = 0. At effective stresses below the preconsolidation 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.

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 regrading and maintenance, is considered to be less critical in a stability analysis.

8.4.2. Gypsum Stack Soil Parameter Selection

Discussions regarding selection of the shear strength parameters are provided in the following paragraphs. Refer to Table 19 for a summary of derived soil parameters.

The clay dike and residual clay primarily consist of fat clay materials with occasional occurrences of lean clays. The cohesive soils sampled during the field exploration were subjected to CU triaxial tests. The results of triaxial testing were evaluated and effective stress p' versus q scatter plots were prepared using all of the data points. Failure was assumed to occur at the point of the maximum effective principal stress ratio (σ_1'/σ_3') Once the p' versus q plots were prepared; a failure envelope was then selected such that about two-thirds of the plotted values were above the envelope. The p' versus q plots and selection of the failure envelope are shown for each soil horizon on the graphs presented in Appendix I. The resulting strength parameters were rounded down to the nearest degree for ϕ' and to the nearest 50 pounds per square foot for the cohesion intercept. Consistent with the discussions in Section 8.4.1, the measured cohesion intercept was neglected (assumed c'=0) in evaluating the dike stability. The unit wet weight was determined by taking the average unit weight of the samples that are included within the soil horizon. These results were compared to the values used in the Ardaman reports for consistency.

The Sedimented Gypsum – Fly Ash material was primarily encountered above the residual clay and clay dikes, with the Cast Gypsum – Fly Ash material above. Stantec performed CU triaxial tests on samples from each soil horizon and the results were plotted on a scatter plot as described above. Again the unit wet weight was determined by taking the average unit weight of the samples included within the soil horizon. These values were compared to the historical values and SPT correlation tables.

The unit weights and shear strength values used for the crushed stone and bedrock were assumed based on typical values presented in the Naval Facilities Engineering Command, NAVFAC Design Manuals 7.1 and 7.2.

The Weak Sedimented Gypsum – Fly Ash was located on the western side of the Gypsum Stack. It was primarily located between 600 and 625 feet in elevation, above the clay soils and below the Cast Gypsum – Fly Ash. CU triaxial testing was again performed. The results indicate that the ϕ' for the constructed ash is on the order of 39º. Since low blow counts were observed for this soil during SPT testing, the shear strength parameters were determined using the SPT correlation tables. Based on these results, an internal friction angle of approximately 27.5° with a cohesion value of 0 psf was used. Stantec also performed, at the request of TVA, a back analysis which specifically targeted the weak layer. The result of the back analysis was provided to TVA under a separate cover.

Table 19. Selected Strength Parameters for Stability Analysis

Soil Horizon	Unit Weight (pcf)	Effective Stress Strength Parameters	
		c' (psf)	ϕ' (degrees)
Cast Gypsum – Fly Ash	113	0	40.0
Sedimented Gypsum – Fly Ash	112	0	41.0
Weak Sedimented Gypsum – Fly Ash	108	0	27.5
Fat Clay	123	0	25.0
Crushed Stone	125	0	35.0
Bedrock	125	0	N/A

8.5. Slope Stability Analyses

The five critical stability sections identified for the Gypsum Stack were evaluated using the limit equilibrium methods as implemented in the UTEXAS4 software, which was developed by Stephen G. Wright. All of the stability sections presented in this report were analyzed for static, long-term conditions with steady-state flow parameters. In this study, steady-state pore pressures were obtained from the SEEP/W model. The long-term analyses looked at the effective-stress, internal angle of friction, and zero cohesion parameters to simulate the condition which will exist long after the excess pore pressures have dissipated from within the embankment material. The unit weight and shear strength properties used in the stability analyses were determined from the geotechnical exploration and laboratory testing as described above in Section 8.

8.5.1. Limit Equilibrium Methods in UTEXAS4

Limit equilibrium methods for evaluating slope stability consider the static equilibrium of a soil mass above a potential failure surface. For conventional, two-dimensional methods of analysis, the slide mass above an assumed failure surface is 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:

$$FS_{slope} = \frac{\text{shear strength of soil}}{\text{shear stress required for equilibrium}} \qquad \textbf{Eqn. 4}$$

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 Mohr-Coulomb strength parameters, as a function of the total or effective normal stress.

Spencer's solution procedure (Spencer 1967; USACE 2003; Duncan and Wright 2005), which satisfies all of the conditions of equilibrium for each slice, was used in this study. An automatic search was selected within the program to find the critical slip surface corresponding to the lowest FS_{slope}. Both rotational and translational failure surfaces were evaluated for this study.

8.5.2. Slope Stability of the Gypsum Stack

The slope stability analyses were carried out using UTEXAS4, which incorporates various search routines to locate the critical slip surface; for the analyses presented here, a circular search (Spencer's Method) and noncircular search (Duncan and Celestino (1981)) was evaluated. For the circular search, an initial center point and tangent depth were chosen for the starting circle. Center points for the trial circles were then confined to a specified range above the slope surface, while the trial radii were varied based on tangent horizontal lines within the soil. The minimum and maximum range for the center points and tangent lines were parametrically varied over a wide range to determine the most critical circle with the lowest factor of safety. The noncircular search was initiated by choosing four initial starting points (which often mirrored the rotational failure) along a possible failure plane, then the program parametrically varied the points based on a set of parameters chosen by the user and the most critical failure plane (lowest factor of safety) was identified.

It should be noted, however that where the surface slope is composed of non cohesive ($c' = 0$) materials, an infinite slope failure (shallow sliding parallel to the surface) will most likely be the critical failure generated by an automatic search. While solutions were obtained for this case, there is less concern for this potential failure mechanism at the Gypsum Stack. Suction pressures in unsaturated surface soils will often create enough apparent cohesion (100 to 200 psf) to prevent this type of failure. If shallow sliding does occur, the resulting deformations are unlikely to threaten the integrity of the dike (global stability) and can be repaired. When these cases were reported as the critical surface, the minimum weight of the failure mass was increased resulting in a deeper failure which could potentially cause a breach in the pond/dike.

9. Results

9.1. Seepage Exit Gradients

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

Considering the SEEP/W results in Appendix G, the predicted phreatic surface is observed to intersect the sloping ground surface just above the perimeter ditch. With the exception of Section K, this condition is predicted for all cross sections analyzed. Ground water seeping through the saturated dike materials may be flowing out to the ground surface, even though direct observations might be obscured by vegetation, evaporation, or the submerged ground surface. In these locations, the seepage forces associated with the hydraulic exit gradients are acting in the same direction as gravity. Because of the high potential for initiating the movement of soil particles and piping, a condition of groundwater seeping to the sloping surface of the downstream face is usually considered unacceptable in the evaluation of earth dams.

The potential for piping due to vertical seepage to the ground surface was also evaluated using the factor of safety defined in Section 8.3.6. First, contour plots of vertical gradient (Appendix G) were examined to determine the general location of the maximum vertical exit gradient. For the factor of safety calculations, average vertical gradients were determined over a depth of 3 to 5 feet just below the ground surface. This way when the model geometry converged to a sharp point (as it normally does), the high exist gradients within this small zone (which is not reflective of the actual conditions in the field) was ignored. In all of the cross sections, except Section K which recently had slope repairs, the maximum upward gradient occurs near the toe of the dike just above the perimeter ditch. For Section K, the maximum upward gradient occurs within the clay dike at the Stilling Pond water line.

The factors of safety against piping presented in Section 8.3.6 were computed based on the exit gradients results from SEEP/W and the critical gradients determined from the soil properties. The results of the computed exit gradients and factors of safety against piping are summarized in Table 20. The lowest computed factor of safety is 1.9 at Section F. TVA requires factors of safety against piping to be greater than three for existing facilities and four for new facilities. Therefore, Sections A, D, F, and K do not meet the design criteria for piping at the seepage exits.

Table 20. Summary of Computed Exit Gradients

Section	Vertical Gradient (i_y) at Critical Exit Point	Location of Critical Exit Point	Soil Horizon	Critical Gradient (i_{crit})	FS_{piping}
Section A	0.36 – 0.35	Perimeter Ditch	Sedimented Gypsum-Fly Ash	0.847	2.3
Section D	0.27 – 0.33	Perimeter Ditch	Sedimented Gypsum-Fly Ash	0.847	2.6
Section F	0.20 – 1.31	Perimeter Ditch	Sedimented Gypsum-Fly Ash	0.847	1.9
Section H	0.13 – 0.38	Perimeter Ditch	Cast Gypsum-Fly Ash	0.697	3.9
Section K	0.34 – 0.45	Stilling Pond Water Elev.	Fat Clay	0.944	2.1

9.2. Slope Stability Results for As Found Stack Height

Using the strength parameters selected (c' and ϕ'), in conjunction with the results of the seepage analyses, the existing Gypsum Stack configuration was analyzed at Section A, D, F, H, and K for rotational failure surfaces. The existing conditions for Section A, D, F, and H were based on the January 2009 topographic survey. The underdrain system was modeled based on the proposed design from TVA drawing numbers 10W235. Section K was modeled with the regraded slope and rock toe buttress constructed as part of Work Plan 5. This section does not have the proposed toe drain components installed. UTEXAS4 was used for the analyses with pore pressures imported from the SEEP/W analyses. Minor details of the geometry, such as the graded stone within the crushed stone buttress along Section K, were not represented in the stability model. The results of the stability analysis are presented in Table 21 below and the critical failure planes are presented in Appendix G.

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

Table 21. Existing Conditions Stability Analysis Results

Stability Section	Station*	Target Factor of Safety	Computed Factor of Safety	Upper Pond Modeled	Upper Pond Elevation (feet)
Section A	125+75	1.5	1.8	Rim Ditch	670.0
Section D	144+27	1.5	2.3	2A	654.5
Section F	225+44	1.5	1.5	3	668.6
Section H	260+75	1.5	1.7	3	668.6
Section K	312+48	1.5	1.7	2B	654.5

Note* Refer to Appendix C for plan view of site with project baseline.

Based on the results of the analysis, each section achieved the minimum factor safety and therefore should not require long term slope mitigation.

It should be noted that past shallow sliding (infinite slope failures) along the downstream face of the Gypsum Stack was reported in the project records, in an area just above the perimeter ditch. These surficial slides were observed and repaired by TVA as part of their routine maintenance program. If additional shallow sliding were to develop again, the failure would be initially confined to the sloping face of the dike. If not repaired and given enough time, these shallow slides could progress up the slope and endanger the stack. Because this progressive failure mechanism would be expected to take months or longer to impact the crest, a robust monitoring program can reduce the risk of failure due to shallow sliding. Hence, in the interim while a permanent mitigation plan is being developed, analyzed, and constructed, TVA should continue to monitor the Gypsum Stack with routine inspections and instrument readings.

9.3. Slope Stability Results for Final Stack Height

Based on discussions with TVA, the Gypsum Stack was evaluated to determine the slope stability based on a five year build out. A production rate of 2,866 dry tons of ash per day from the 1991 and 2005 Ardaman and Associates reports was used to estimate the height of the stack in five years. The 2009 survey was used as the existing surface and the final height was projected using the design slope geometry shown in TVA Drawing Series 10W235. This design mandates 2.5:1 slopes above the 655 bench with 20 foot wide stability benches at 685 feet and 720 feet with the crest at 755 feet. Based on a 2005 production rate, current Gypsum Stack configuration, and design slopes, the elevation of the stack in five years was estimated to be 700 feet. Again, the phreatic surface was determined using SEEP/W with the proposed underdrain system from TVA drawing number 10W235 modeled. The underdrain system consists of a toe drain as well as slope drains within the Gypsum Stack at the 655 and 685 benches. The results of the five year build out analysis are presented in Table 22 below.

Table 22. Five Year Build out Stability Analysis Results

Stability Section	Station	Target Factor of Safety	Computed Factor of Safety	Upper Pond Modeled	Upper Pond Elevation (feet)
Section A	125+75	1.5	1.5	Rim Ditch	690.0
Section D	144+27	1.5	1.6	2A	695.0
Section F	225+44	1.5	1.5	3	695.0
Section H	260+75	1.5	1.3	3	695.0
Section K	312+48	1.5	1.4	2B	695.0

The Gypsum Stack meets the minimum required factor of safety for existing conditions; however, based on the results from the stability analysis for the five year build out, the calculated minimum factor of safety is 1.3, with factors of safety ranging from 1.3 to 1.6. This does not meet the required factor of safety of 1.5 and some measures will be required to improve the stability of the Gypsum Stack as the height continues to increase. This is discussed further in the recommendation section of this report.

10. Limitations of Study

The scope of this evaluation was limited to considering only the potential risks at the Gypsum Stack due to excessive seepage and slope instability. This assessment did not consider potential failure modes related to spillway capacity and overtopping, seepage along penetrations through the embankment (including the buried spillway pipes), erosion due to wave action or flood stage flows, vegetation on the dike face, performance of the internal divider dikes, or other possible mechanisms.

The stability of the Gypsum Stack during a potential earthquake was not specifically analyzed. Data from the site explorations indicate low penetration resistance (low density) in the saturated weak gypsum-fly ash material. In a strong earthquake, these soils will be prone to liquefaction, which would undermine the stability of the stack. However, the seismic risk at this site (likelihood of experiencing a large magnitude earthquake) is quite low for the remaining life.

Stability analyses were not performed for rapid drawdown conditions. On the upstream side (Gypsum Stack or Stilling Pond), a rapid drawdown condition would correspond to a failure of the stack, perhaps due to a breach in the dike or failure of the spillway. While the upstream dike slope may be vulnerable to sliding due to rapid drawdown, this mechanism would not likely result in a breach or global failure of the stack. Therefore, this failure scenario was not evaluated.

11. Conclusions

The conclusions and recommendations that follow are based upon Stantec's understanding of the facility as outlined herein. This understanding of the facility developed from reviews of historical information provided by TVA and discussions with TVA personnel throughout the course of this work and results of the geotechnical exploration and stability analysis.

11.1. A general topographic review of the available 2007 mapping (TVA 10W235 series drawings) and the updated 2009 mapping indicate the stack slopes below the 650/655 bench level are on the order of 2.75:1 and above the bench level they are approximately 2.5:1. Therefore, based on this comparison, the slope conditions appear to follow the intended design slopes. However, a few of the cross sections did indicate a shift away from the stack from the original design plans.

11.2. The results from the seepage analyses were examined to identify conditions where piping and erosion of soil might develop due to seepage forces. The model results indicated a shallow phreatic surface (ground water table) just above the perimeter ditch and stilling pond; these results are generally confirmed by the observation of shallow seeps in the areas. Out of the five cross sections modeled, four indicate high exit gradients at the toe of the Gypsum Stack. This condition creates the potential for the initiation of soil piping, as seepage water will tend to erode material from within the dike. Upward, vertical exit gradients in the area of the dike toe were also found to be excessive. Factors of safety against piping, computed for the surficial 3 to 5 feet of soil in these areas, ranged from 1.9 to 3.9. Based on TVA Design Criteria, factors of safety against piping should be greater than three for existing facilities. The results from the seepage model thus demonstrate that the majority of the Gypsum Stack does not meet current criteria for preventing soil piping due to seepage.

11.3. Current criteria for the long-term stability of the Gypsum Stack require a factor of safety for slope stability of at least 1.5. The slope stability results at the as found stack height show that the Gypsum Stack meets this criterion. However, this does not imply that the dike will remain stable for the life of the structure or during the expansion of intermediate phases.

Based on the stability analysis results for the final stack height, the calculated minimum factor of safety is 1.3, with factors of safety ranging from 1.3 to 1.6. This does not meet the required factor of safety of 1.5 and some measures will be required to improve the stability of the Gypsum Stack as the height continues to increase. Therefore, TVA should undertake specific efforts to improve the safety of this facility and reduce the interim risks (estimated five years) associated with stacking the wet gypsum-fly ash until a new dry stacking operation is operational. The following actions are recommended.

12. Recommendations

12.1. Active Spillways

12.1.1. Decant weir structures should be fitted with skimmers for the management of cenospheres.

12.1.2. It is recommended that all active spillway risers be instrumented with staff gages, such that the water elevations in the surrounding ponds can be compared with the piezometer data. This additional information will aid in monitoring the normal phreatic levels within the stack which could be affected by future expansion/construction.

12.2. Annual Evaluation of Drainage Systems

12.2.1. Beginning in the current year, annual inspections of the Gypsum Stack should include a thorough evaluation of the underdrain and surface drain systems. For documentation purposes of the evaluation, the summary included on pages 9 through 13 of the March 2, 2006 Memorandum titled "Preliminary Assessment of Seepage Collection Drains" should be developed into a form to be updated in the field.

12.3. Seepage Improvements

12.3.1. The seepage analyses results identified a potential mode of failure for the as found stack height. Based on our field observations, evidence of excessive seepage is present around the perimeter toe of the Gypsum Stack and the underdrain system is only partially effective in depressing the phreatic surface in the stack and clay dikes. As part of the Phase 3 engineering design work, design modifications to the stacking plan should be developed to make field adjustments to the planned underdrain (Slope Drains at the 655 and 685 levels) and to address/add protection against piping failures. Specific improvements to address potential soil piping should include the use of reverse graded filters located at the toe of the stack and clay dikes.

12.4. Stability Improvements

12.4.1. The stability of the Gypsum Stack will decrease as the height continues to increase as indicated in the final stack height stability analysis. To improve the long-term stability, TVA should initiate a mitigation design and construction program. The mitigation project should involve the placement of stabilizing berms at the toe of stack and on the downstream face of the clay dike.

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

12.4.3. The berm should also be designed to provide protection against seepage and piping failures, and increase the factor of safety against piping to meet the design guideline value of 3 for existing facilities. Where the berm is built over areas subject high to exit seepage gradients (toe of the stack and downstream face of the clay dike), the gradation of the berm should be selected to filter the potential seepage water.

12.5. Storm Water Improvements

12.5.1. In parallel to the Phase 2 geotechnical engineering assessment, Stantec has also conducted H&H analyses of the gypsum stack and stilling pond based on the proposed stack side slopes, closure cap at a final stack height of elevation 700 feet, and perimeter drain piping. The analysis was performed using the following criteria from the draft URS Governance Document, dated July 3, 2009:

- Stilling pond should contain the volume of runoff from a 25-year 24-hour rain event;

- Stilling pond should pass the 100-year 24-hour rain event without overtopping; and

- Gypsum stack should pass the Probable Maximum Precipitation (PMP) with at least 5 feet of freeboard.

Based on the URS volume criteria, the pond is sized properly at about 50 acre-feet of storage. As for the discharge, it should still have about two feet of freeboard during the 100-year storm, assuming the principal spillway is open and functioning properly. If TVA maintains at least 5 feet of freeboard above the operating pool of the gypsum stack it should adequately manage the storm surge from the PMP.

12.5.2. Based on our H&H analysis, Stantec recommends the following:

- Re-establish the Stilling Pond Emergency Spillway at elevation 619.0 feet to provide an emergency overflow path should the principal spillway not function properly. The outfall pipe from the Stilling Pond is at a flat slope, so clogging is possible. The emergency spillway should be 100 feet wide with transition slopes on each side to the embankment elevation of 620.0'. Alternately, if the embankment has been raised a similar configuration will work at the higher elevation provided the overflow path is at least 100 feet in width and one foot in depth. The slope and toe area downstream of the emergency spillway should be protected with rip-rap.

- For the gypsum stack, TVA should maintain at least 5 feet of freeboard between the operational pool and crest of the dam from now until the facility is closed. This conforms to the design guidelines above and assuming the operational footprint does not significantly change, it should allow an adequate storm retarding storage above the normal pool.

- Drainage of the intermediate stability benches (655 and 685 levels) should be designed using a pipe system with surface inlets at each level to minimize slope erosion on the stack.

12.5.3. Surface drainage from the gypsum stack has also been evaluated based on field observations and the available project documents. The toe ditch around the gypsum stack has an average slope of 0.29% and was found to be undersized given the criteria listed in Section 12.5.1. Stantec recommended one of three alternative solutions be pursued to address the capacity issues with the toe ditch. The most viable option selected by TVA involves the replacement of the ditch with a pipe system with surface inlets.

12.6. Comprehensive Design Modifications

12.6.1. It is recommended that the mitigation plan address each geotechnical and hydraulic deficiency identified herein under one project since all involve the area at the toe of stack and are somewhat interrelated. The design of seepage, stability, and storm water improvements should achieve the following goals:

- Provide a gravity drain for existing underdrain elements around the stack (see Figure 14.);
- Install planned underdrain elements consisting of Slope Drains at the 655 and 685 levels.
- Collect seepage at the lowest possible level at the toe of the stack;
- Add surcharge load to improve stability against global slope failure (see Appendix J-1 of 5);
- Provide a gravity drain for post-closure conditions with the capacity to manage the storm surge from the PMP;
- Provide protection against soil piping at the toe of the stack and downstream slope of the clay dikes.

12.6.2. It should be understood the conceptual design of the improvements is currently ongoing. Sketches of the conceptual design modifications are presented in Appendix J. The improvements shown in the typical cross section on Figure J-1 have a computed minimum factor of safety against sliding $F_{sliding}$ = 1.5. The conceptual cross section will require modifications as it is applied around the stack perimeter.

12.6.3. For cost estimating purposes, it was assumed the improvements shown will be applied to the perimeter of the stack as shown in Figure J-2 and J-3 in Appendix J. Our opinion of probable cost for the recommended design modifications is $13M. An estimate to closure the stack was also developed based on the underdrain and surface drains included on current TVA design plans (10W235 series drawings). Our opinion of probable cost for the closure (excluding the recommended design modifications) is $13M for a total cost including both of $26M.

These estimates do not include routine maintenance and handling costs associated with operating the stack. A derivation of the cost estimate is presented in Appendix J. The estimator who is currently, Stantec, should be provided with annual operational cost and cost data of this year's improvements to improve his/her ability to estimate costs in the detailed design phase of work.

13. Closure

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

These conclusions and recommendations are based on data and subsurface conditions from the borings advanced during this investigation using that degree of care and skill ordinarily exercised under similar circumstances by competent members of the engineering profession. The boring logs and related information presented in this report depict approximate subsurface conditions only at the specific boring locations noted and at the time of drilling. Conditions at other locations may differ from those occurring at the boring locations. Also, the passage of time may result in a change in the subsurface conditions at the boring locations.

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

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

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

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Appendix A

Historic Documents

Appendix B

Drilling Logs and Piezometer Installation Records

Appendix C

Geotechnical Drawings

Appendix D

SPT Correlation Tables

Appendix E

Slope Inclinometer and TVA Ground Survey Monuments

Appendix F

Results of Laboratory Testing

Appendix G

Results of Engineering Analysis

Appendix H

Piezometer Readings

Appendix I

Laboratory Summary Tables

Appendix J

Recommendations

Appendix K

Record Drawings

Appendix L

Results of 5 Year Build Out Analysis