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Coal Combustion Residue Impoundment

Round 11 - Dam Assessment Report

Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Alabama

Prepared for:

United States Environmental Protection Agency
Office of Resource Conservation and Recovery

Prepared by:

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INTRODUCTION, SUMMARY CONCLUSIONS AND RECOMMENDATIONS

The release of over five million cubic yards of coal combustion residue (CCR) from the Tennessee Valley Authority's Kingston, Tennessee facility in December 2008, which flooded more than 300 acres of land, damaging homes and property, is a wake-up call for diligence on coal combustion residue disposal units. A first step toward this goal is to assess the stability and functionality of the ash impoundments and other units, then quickly take any needed corrective measures.

This assessment of the stability and functionality of the Main Ash Complex and Gypsum Stack impoundments is based on a review of available documents and on the site assessment conducted by Dewberry personnel on Monday, September 13, 2011. We found the supporting technical documentation inadequate (Section 1.1.3). As detailed in Section 1.2, there is one recommendation that would help to ensure a safe and trouble-free operation.

In summary, the Widows Creek Fossil Plant's Gypsum Stack is rated SATISFACTORY and the Main Ash Complex is rated **FAIR** for continued safe and reliable operation. The FAIR and not SATISFACTORY rating is due to a lack of liquefaction assessment information.

PURPOSE AND SCOPE

The U.S. Environmental Protection Agency (EPA) is embarking on an initiative to investigate the potential for catastrophic failure of Coal Combustion Surface Impoundments (i.e., management unit) from occurring at electric utilities in an effort to protect lives and property from the consequences of a dam failure or the improper release of impounded slurry. The EPA initiative is intended to identify conditions that may adversely affect the structural stability and functionality of a management unit and its appurtenant structures (if present); to note the extent of deterioration (if present), status of maintenance and/or a need for immediate repair; to evaluate conformity with current design and construction practices; and to determine the hazard potential classification for units not currently classified by the management unit owner or by a state or federal agency. The initiative will address management units that are classified as having a Less-than-Low, Low, Significant or High Hazard Potential ranking. (For Classification, see pp. 3-8 of the 2004 Federal Guidelines for Dam Safety)

In February 2009, the EPA sent letters to coal-fired electric utilities seeking information on the safety of surface impoundments and similar facilities that receive liquid-borne material that store or dispose of coal combustion residue. This letter was issued under the authority of the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) Section 104(e), to assist the Agency in assessing the structural stability and functionality of such management units, including which facilities should be visited to perform a safety assessment of the berms, dikes, and dams used in the construction of these impoundments.

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EPA requested that utility companies identify all management units including surface impoundments or similar diked or bermed management units or management units designated as landfills that receive liquid-borne material used for the storage or disposal of residuals or by-products from the combustion of coal, including, but not limited to, fly ash, bottom ash, boiler slag, or flue gas emission control residuals. Utility companies provided information on the size, design, age and the amount of material placed in the units. The EPA used the information received from the utilities to determine preliminarily which management units had or potentially could have High Hazard Potential ranking.

The purpose of this report is **to evaluate the condition and potential of residue release from the Main Ash Complex and Gypsum Stack management units and to determine hazard potential classification**. This evaluation included a site visit. Prior to conducting the site visit, a two-person team reviewed the information submitted to EPA, reviewed any relevant publicly available information from state or federal agencies regarding the unit hazard potential classification (if any) and accepted information provided via telephone communication with the management unit owner. Also, after the field visit, additional information was received by Dewberry & Davis LLC about the {management unit} that were reviewed and used in preparation of this report.

Factors considered in determining the hazard potential classification of the management units(s) included the age and size of the impoundment, the quantity of coal combustion residuals or by-products that were stored or disposed of in these impoundments, its past operating history, and its geographic location relative to down gradient population centers and/or sensitive environmental systems.

This report presents the opinion of the assessment team as to the potential of catastrophic failure and reports on the condition of the management unit(s).

LIMITATIONS

The assessment of dam safety reported herein is based on field observations and review of readily available information provided by the owner/operator of the subject coal combustion residue management unit(s). Qualified Dewberry engineering personnel performed the field observations and review and made the assessment in conformance with the required scope of work and in accordance with reasonable and acceptable engineering practices. No other warranty, either written or implied, is made with regard to our assessment of dam safety.

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APPENDIX A

Doc 01: *Phase 2 Geotechnical Exploration, Ash Pond Complex, Widows Creek Fossil Plant, Stevenson Alabama*, Stantec Consulting Services, February 4, 2010

Doc 02: *Phase 2 Geotechnical Exploration, Gypsum Stack, Widows Creek Fossil Plant, Stevenson Alabama*, Stantec Consulting Services, February 5, 2010

Doc 03: *Breach Inundation Analysis, Widows Creek Fossil Plant, Jackson County, Alabama,* Stantec Consulting Services, June, 2010

Doc 04: TVA Spreadsheet "INCR_VOL_DATA_RHOM_WCF", dated September 28, 2011

Doc 05: Area Topographic Map

Doc 06: *2010 Annual Inspection of CCP Facilities and Ponds, Widows Creek Fossil Plant, Stevenson, Alabama,* Stantec Consulting Services, May 14, 2010

Doc 07: *2011 Annual Inspection of CCP Facilities and Ponds, Widows Creek Fossil Plant, Stevenson, Alabama,* Stantec Consulting Services, July 27, 2011

Doc 08: TVA Widows Creek Fossil Plan NPDES Permit Number AL0003875

Doc 09: Stantec Consulting Services Memorandum September 28, 2010, Reference: "Hydrologic and Hydraulic Calculations summary, Spillway Improvement Project, Widows Creek Fossil Plant (WCF), Main Ash Pond"

Doc 10: Stantec Consulting Services Memorandum September 30, 2010, Reference: "Hydrologic and Hydraulic Calculations summary, Spillway Improvement Project, Widows Creek Fossil Plant (WCF), Gypsum Stack Stilling Pond"

Doc 11: Stantec Consulting Services correspondence September 29, 2011, Reference:"Results of Seismic Slope Stability Analysis, Active CCP Disposal Facilities, Widows Creek Fossil Plant:

Doc 12: *Seepage Action Plan (SAP), Widows Creek Fossil Plant, Stevenson, Alabama,* Stantec Consulting Services, June 25, 2010

Doc 13: *Tennessee Valley Authority (TVA) routine handling operations and maintenance (RHO&M) operating support document, Widows Creek Fossil Plant*, July 2011

Doc 14: *Calculation Package and Basis of Design Report, Slope Stability Improvements, Ash Pond Complex, Jackson County, AL,* March 2012, URS Corp.

Doc 15: *Basis of Design Report, IRRP #1, Clay Dike Buttress, Widows Creek, Jackson County, AL,* December, 2010, URS Corp.

Doc 16: Stantec Consulting Services Inc. correspondence February 15, 2012, Reference "Results of Pseudostatic Slope Stability Analysis, Active Disposal Facilities, BRF, COF, GAF, JSF, JOF, KIF, PAF, and WCF

APPENDIX B

Doc 17: Main Ash Pond - Dam Inspection Check List Form

Doc 18: Gypsum Stack – Dam Inspection Check List Form

Doc 19: Dewberry Memorandum dated May 25, 2012, Regarding Qualitative Assessment

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1.0 CONCLUSIONS AND RECOMMENDATIONS

1.1 CONCLUSIONS

Conclusions are based on visual observations from a one-day site visit, September 13, 2011, and review of technical documentation provided by Tennessee Valley Authority (TVA).

1.1.1 Conclusions Regarding the Structural Soundness of the Management Unit(s)

The dike embankments and spillway appear to be structurally sound based on a review of the engineering data provided by the owner's technical staff and consultants, and Dewberry engineers' observations during the site visit. The dikes meet minimum factors of safety under static and seismic conditions. However, the Main Ash Complex dikes have been built and raised such that parts of the embankments sit atop ash material, and no liquefaction potential information has been provided.

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

Hydrologic and hydraulic analyses provided to Dewberry indicate adequate impoundment capacity to contain the 1 percent probability design storm without overtopping the dikes.

1.1.3 Conclusions Regarding the Adequacy of Supporting Technical Documentation

The supporting technical documentation pertaining to the seismic stability analyses is inadequate, due to the lack of a liquefaction potential analysis. The engineering documentation reviewed is referenced in Appendix A.

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

The description of the management unit provided by the owner was an accurate representation of what Dewberry observed in the field.

1.1.5 Conclusions Regarding the Field Observations

Dewberry staff was provided access to all areas in the vicinity of the management unit required to conduct a thorough filed observation. The visible parts of the embankment dikes and outlet structure were observed

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to have no signs of overstress, significant settlement, shear failure, or other signs of instability. Embankments appear structurally sound. There are no apparent indications of unsafe conditions or conditions needing remedial action.

1.1.6 Conclusions Regarding the Adequacy of Maintenance and Methods of Operation

The current maintenance and methods of operation appear to be adequate for the fly ash management unit. Except for recently constructed inverted filter drains for seepage control, there was no evidence of significant embankment repairs or prior releases observed during the field inspection.

1.1.7 Conclusions Regarding the Adequacy of the Surveillance and Monitoring Program

The surveillance program appears to be adequate. The management unit dikes are instrumented. Embankments impounding the Main Ash Complex and Gypsum Stack are both instrumented with groundwater piezometers and slope inclinometers.

1.1.8 Classification Regarding Suitability for Continued Safe and Reliable Operation

The Gypsum Stack management unit is rated SATISFACTORY. The CCR ash management unit is rated FAIR for continued safe and reliable operation due to lack of sufficient engineering data. Implementation of the following recommendation would help improve the rating. It is anticipated that the Main Ash Complex management units would be considered satisfactory for continued safe and reliable operation upon implementation of the recommendation in 1.2.1 below.

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1.2 RECOMMENDATIONS

1.2.1 Recommendations Regarding the Supporting Technical Documentation

Additional documentation is recommended to:

- Conduct liquefaction susceptibility analysis of Main Ash Complex Raised Dike, and CCR materials supporting the raised dike.

1.3 PARTICIPANTS AND ACKNOWLEDGEMENT

1.3.1 List of Participants

Bill Roddy, Tennessee Valley Authority
Adam Thomas, Tennessee Valley Authority
Scott Turnbow, Tennessee Valley Authority
Alan D. Dincher, Tennessee Valley Authority
Jimmy Mullins, Tennessee Valley Authority
Marty Helton, Tennessee Valley Authority
Kris Johnson, Tennessee Valley Authority
Jake Booth, Tennessee Valley Authority
Stanley Nixon Tennessee Valley Authority
Robert Fuller, Stantec
Joseph P. Klein, III, Dewberry
Frank B. Lockridge, Dewberry

1.3.2 Acknowledgement and Signature

We acknowledge that the management unit referenced herein has been assessed on September 13, 2011

Joseph P. Klein, III, P.E.
Registered, AL #25976

Frank B. Lockridge, P.E.

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

2.1 LOCATION AND GENERAL DESCRIPTION

The Widows Creek Fossil Plant is located on the northwest bank of the Tennessee River, north of the city of Stevenson, Jackson County, Alabama. The site location and area topography is shown on Figure 2.1-1.

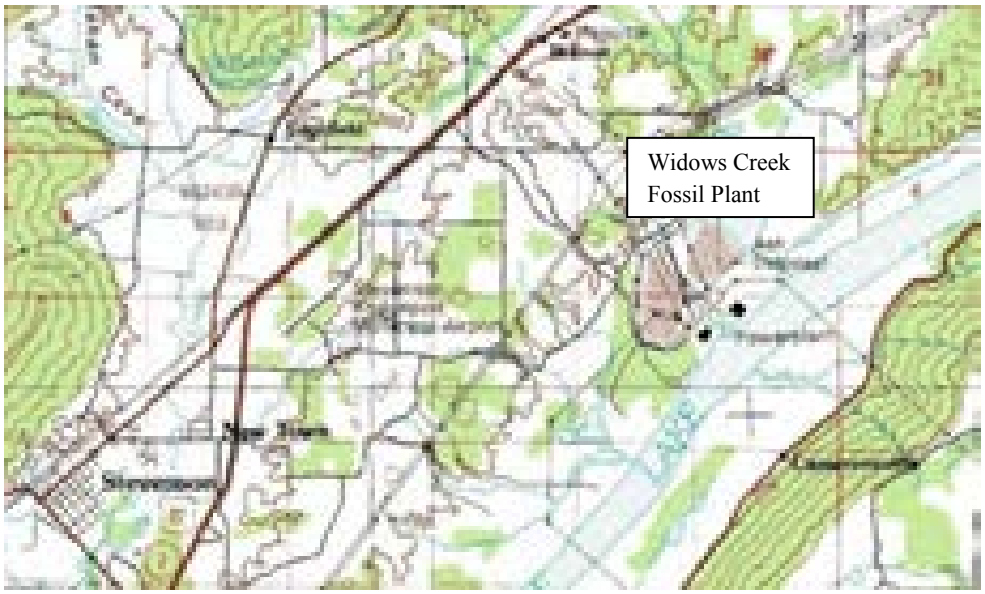


Figure 2.1-1 Widows Creek Fossil Plant Location

Main Ash Complex

The Main Ash Complex impoundment is located about 0.4 miles north-northeast of the main plant. The Main Ash Complex includes three units being used to impound coal combustion residuals (CCR) including fly ash and bottom ash. The three units are impounded by a common exterior dike. The three units include: Dredge Cell, Main Ash Pond A and Bottom Ash Stack. Figure 2.1-2 shows the location of the Main Ash Complex and Gypsum Stack impoundment on the plant site.

The Main Ash Complex includes five additional units not used for CCR storage: the Red Water Pond; Copper and Iron Ponds (also referred to as Chemical ponds); the Upper and Lower Stilling Ponds, and the Pump Pond.

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Gypsum Stack

The Gypsum Stack impoundment is located about 850 feet northeast of the Main Ash Complex. Widows Creek separates the Main Ash Complex and the Gypsum Stack impoundment. The Gypsum Stack receives sluiced gypsum, mechanically collects the wet gypsum, spreads it out within the impoundment for drying, and when dried, places the material onto the stack.



Figure 2.1-2 Main Ash Complex and Gypsum Stack Locations

Table 2.1: Summary of Dam Dimensions and Size		
	Main Ash Complex[1]	**Gypsum Stack[2]**
Dam Height (ft)	30	35
Crest Width (ft)	22	22
Length (ft)	18,200	10,300
Side Slopes (upstream) H:V	2(H):1(V)	2.5(H): 1(V)
Side Slopes (downstream) H:V	2.8(H):1(V) to 3(H):1(V)	2(H):1(V)

[1] Based on information and drawings in Stantec Consulting Services, Inc, *Report of Phase 2 Geotechnical Exploration, Ash Pond Complex, Widows Creek Fossil Plant, Stevenson, Alabama*, February 4, 2010 (See Appendix A – Doc 01)

[2] Based on information and drawings in Stantec Consulting Services, Inc, *Report of Phase 2 Geotechnical Exploration, Gypsum Stack, Widows Creek Fossil Plant, Stevenson, Alabama*, February 5, 2010 (See Appendix A – Doc 02)

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2.2 COAL COMBUSTION RESIDUE HANDLING

2.2.1 Fly Ash

Equipment used for the collection, handling and disposition of fly ash at Widows Creek Fossil Plant includes selective catalytic reduction (SCR) hoppers; precipitator hoppers; economizer hoppers; hydroveyors; air separation tanks, and piping. The economizer hoppers are housed inside buildings at the plant. Other equipment is located outside.

2.2.2 Bottom Ash

Equipment used for the collection, handling and disposition of bottom ash at Widows Creek Fossil Plant includes wet bottoms; jet pumps; hydroveyors; and piping. The wet bottoms are housed inside buildings at the plant. Other equipment is located outside.

2.2.3 Boiler Slag

Information on equipment used to manage boiler slag at Widows Creek Fossil Plant was not available.

2.2.4 Flue Gas Desulfurization Sludge

Equipment used for the collection, handling and disposition of Flue Gas Desulfurization Sludge at Widows Creek Fossil Plant includes limestone preparation facilities, absorbers, tanks, pumps and piping. The limestone preparation facilities are housed inside buildings at the plant. Other equipment is outside

2.3 SIZE AND HAZARD CLASSIFICATION

The embankment impounding the Main Ash Complex consists of a starter dike and raised dike with an average height of about 35 feet and a total storage capacity of about 572 acre-feet.

The Gypsum Stack Stilling Pond is impounded by a clay dike with a height of about 30 feet and a total storage capacity of about 990 acre-feet.

The classification for size, based on the height of the embankment and the impoundment storage volume of the Main Ash Complex and Gypsum Stack is "Small" based on the U.S. Army Corps of Engineers (USACE) Recommended Guidelines for Safety Inspection of Dams, ER 1110-2-106 criteria summarized in Table 2.2a:

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Table 2.2a: USACE ER 1110-2-106 Size Classification		
	Impoundments	
Category	Storage (Ac-ft)	Height (ft)
Small	50 and < 1,000	25 and < 40
Intermediate	1,000 and < 50,000	40 and < 100
Large	> 50,000	> 100

Dewberry conducted a qualitative hazard classification based on Federal Guidelines for Dam Safety, dated April, 2004. The hazard assessment classifications are summarized on Table 2.2.b.

Table 2.2b: FEMA Federal Guidelines for Dam Safety Hazard Classification		
	Loss of Human Life	Economic, Environmental, Lifeline Losses
Low	None Expected	Low and generally limited to owner
Significant	None Expected	Yes
High	Probable. One or more expected	Yes (but not necessary for classification)

Based on the height of the gypsum stacks above the crest of the perimeter embankment, TVA previously judged the Gypsum Stack impoundment to be a high hazard facility. Subsequently TVA retained Stantec Consulting Services to conduct a dam break analysis to assess the hazard presented by a catastrophic failure of the gypsum stack (See Appendix A - Doc 03). Briefly, the results indicated the dam break inundation zone included several homes and local, rural roadways.

Based on the results of the dam break analysis TVA acquired and demolished the homes in the inundation area. TVA also surveyed area roads and determined that the inundation would not overtop the roadway embankments. As a result, TVA revised its hazard rating for the gypsum Stack to Significant.

Based on the mitigation measures taken by TVA, a failure of the CCR impoundments, including the Main Ash Complex and the Gypsum Stack, at the Widows Creek Fossil Plant is not expected to result in a loss of human life. Economic and environmental losses are expected due to the potential for released materials reaching the Tennessee River. Therefore Dewberry evaluated the Main Ash Complex and the Gypsum Stack each as being a "**Significant hazard potential.**"

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

Materials stored in Main Ash Pond A include both fly ash and bottom ash. There are two sluiced ash discharges at two locations in the Bottom Ash Stack: sluice from Units 1 through 6 discharge to a ditch on the east side of the Bottom Ash Stack, and sluice from Units 7 and 8 discharge into a ditch at the west side of the Bottom Ash Stack. Photograph 2.4.1 shows the sluiced ash discharge from Units 7 and 8.



Photograph 2.4.1: Sluiced Ash Discharge from Plant Units 7 and 8 to Bottom Ash Stack.

The ditches converge near the north end of the Bottom Ash Stack and flow into Main Ash Pond A.

The bottom ash dredged from the sluice ditches and Main Ash Pond A is stacked to form the Bottom Ash Stack. Water stored in the Bottom Ash Stack consists of sluiced water in the ditches, water retained in the bottom ash after dredging and transport to the stack, and surface runoff from precipitation events.

The Dredge Cell is currently inactive with occasional use as a staging area for construction activities in other parts of the Main Ash Complex. Gypsum/scrubber sludge and dredged material from Main Ash Pond A previously placed in the dredge cell are being allowed to dewater.

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Design development is currently underway to close the Main Ash Complex.

The Gypsum Stack Stilling Pond receives water from two main sources:

- Discharge from the gypsum stack including excess sluice water and discharge from the internal drainage system
- Surface runoff from the gypsum stack and direct rainfall.

Information on the surface area, storage capacity, and elevation of each CCR pond is summarized in Table 2.3.

Table 2.3: Maximum Capacity of Unit

Ash Pond Name	Main Ash Pond A	Gyp Stack Stilling Pond
Surface Area (acre)[1]	156	6.5
Current Storage Capacity (cubic yards)[2]	390,456	79,655
Current Storage Capacity (acre-feet)	572	49
Total Storage Capacity (cubic yards)[2]	1,591,338	157,335
Total Storage Capacity (acre-feet)	986	98
Crest Elevation (feet)	636	621.4
Normal Pond Level (feet)	627	614.6

[1] Data taken from Stantec geotechnical exploration reports of February, 2010
[2] Data taken from Widows Creek Incremental Volume survey data dated May 2, 2011 (Appendix A – Doc 04)

2.5 PRINCIPAL PROJECT STRUCTURES

2.5.1 Earth Embankment

Main Ash Complex

The earth embankment impounding the Bottom Ash Stack, Main Ash Pond A, and Dredge Cell consists of an earth fill starter dike and raised dike about 18,400 feet long. The embankment forming the Bottom Ash Stack impoundment was constructed in the early to mid-1950's to store CCR from the plant's original six generating units. The Bottom Ash Stack impoundment was expanded horizontally in the late 1950's to early 1960s to include Main Ash Pond A, and the Dredge Cell. The expansion was constructed in conjunction with the addition of generating units 7 and 8 to the plant.

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Gypsum Stack

The earth embankments impounding the Gypsum Stack consists of an earth fill dike about 10,250 feet long. The Gypsum Stack was constructed in the mid 1980's in conjunction with the plant's change to a forced oxidation scrubber system.

2.5.2 Outlet Structures

Main Ash Complex

Sluiced ash discharge enters the Main Ash Complex at two locations near the south end of the Bottom Ash Stack. The sluice flows through two ditches to the Main Ash Complex. A spillway system of five 4-foot diameter concrete risers with 10-foot diameter corrugated metal pipe skimmers is present, but no longer in service. Water level in Main Ash Pond A is controlled by a siphon system consisting of six 12-inch diameter steel siphons. The siphons discharge to the Upper Stilling Pond.

Gypsum Stack Stilling Pond

Stormwater runoff, internal drains and excess sluice water drain to the Gypsum Stack Stilling Pond. The Gypsum Stack Stilling Pond spillway consists of a 30-inch diameter steel pipe through the embankment. The spillway discharges to a concrete lined channel that empties into the Tennessee River.

The Gypsum Stack Stilling Pond also has an emergency spillway that empties into Widow Creek.

2.6 CRITICAL INFRASTRUCTURE WITHIN FIVE MILES DOWN GRADIENT

Critical infrastructure information was not provided to Dewberry for review.

Based on the available topographic maps (See Appendix A – Doc 05) surface drainage at the plant is to the southeast towards the Tennessee River. Localized site drainage on the east side of the Main Ash Complex is westward toward Widows Creek. Similarly, drainage from the Gypsum Stack is to the south – southwest toward Widows Creek. Widows Creek flows into the Tennessee River near the southeast corner of the Main Ash Complex Dredge Cell.

There are no recognized critical structures within five miles of the CCR management units.

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3.0 SUMMARY OF RELEVANT REPORTS, PERMITS, AND INCIDENTS

TVA provided representative daily, weekly, monthly and quarterly inspection reports prepared by TVA personnel for the Main Ash Complex and the Gypsum Stack. TVA also provided reports of dam safety inspections conducted by Stantec, including:

- "2010 Annual Inspection Report of CCP Facilities and Ponds, Widows Creek Fossil Plant", dated May 14, 2010 (See Appendix A –Doc 06)
- "2011 Annual Inspection Report of CCP Facilities and Ponds, Widows Creek Fossil Plant", dated July 27, 2011(See Appendix A – Doc 07)

Neither inspection report indicated findings of significance. Recommendations presented in the 2010 report were generally related to routine maintenance issues including:

- Continue mowing operations
- Repair of animal burrows
- Repair of erosion rills
- Remove pipe through Dredge Cell impoundment dike. Pipe originally installed during Widows Creek dredging, Pipe is acting as rim ditch discharge which is not its purpose.
- Replace culvert pipe under road from Main Ash Pond A which is partially filled with sediment

The 2011 annual inspection report identified changes in the Main Ash Complex. The most significant changes included:

- Construction of a new siphon spillway system to control water elevation in Main Ash Pond A.
- Construction of about 4,800 feet of stone buttress and inverted filter/stone buttress along the Main Ash Pond A starter dike embankment exterior slopes. The new construction was conducted in response to findings and recommendation presented in hydrologic and hydraulic analysis reports, and geotechnical reports prepared in 2010.

- TVA provided technical reports prepared by Stantec related to the safety and stability of the CCR impoundments. Those reports associated with safety and stability are provided in Appendix A

The results of the hydrologic and hydraulic analyses are discussed in Section 6.0 of this report. The results of the geotechnical exploration and analysis are discussed in Section 7.0 of this report.

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3.1 SUMMARY OF LOCAL, STATE, AND FEDERAL ENVIRONMENTAL PERMITS

The State of Alabama has not implemented a dam safety program; therefore there is no local or state permit.

Discharge from the impoundments is regulated by the Alabama Department of Environmental Management and the impoundments have been issued a National Pollutant Discharge Elimination System Permit: Permit No. AL0003875 issued March 8, 2005. (See Appendix A – Doc 08). The permit expired on March 31, 2010. TVA reportedly submitted the application for permit renewal prior to September 30, 2009. Review of the renewal application by the Alabama Department of Environmental Management has not been completed.

3.2 SUMMARY OF SPILL/RELEASE INCIDENTS

Data reviewed by Dewberry did not indicate any spills, unpermitted releases, or other performance related problems with the Main Ash Complex over the last 10 years.

Data reviewed by Dewberry indicated one unpermitted release and one contained spill from the Gypsum Stack within the last three years. The incidents were:

- On January 9, 2009 an abandoned weir failed releasing gypsum fly ash material into the Gypsum Stack Stilling Pond and Widows Creek.
- On January 4, 2010 a newly constructed spillway pipe from a stormwater pond in the gypsum stack washed out causing gypsum fly ash material to slide down the stack and into the Stilling Pond. Available data indicates that the material was retained in the Gypsum Stack Stilling Pond.

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4.0 SUMMARY OF HISTORY OF CONSTRUCTION AND OPERATION

4.1 SUMMARY OF CONSTRUCTION HISTORY

4.1.1 Original Construction

Main Ash Complex

The original plant construction in the early 1950's included six generating units. Available data indicate an original ash disposal area designated "Ash Disposal Area Units 1-6" or "Ash Pond 1". Data provided to Dewberry for review indicates that Ash Pond 1 was filled and taken out of service in 1969.

As construction of Ash Pond 1 was completed, design began on "Ash Area 2."The footprint of Area 2 included the Bottom Ash Stack, Main Ash Pond A and the Dredge Cell.

The Bottom Ash Stack was designed to receive sluiced ash from new generating units 7 and 8. The original embankment forming the Bottom Ash Stack had a crest elevation of 626 feet. As the Bottom Ash Stack pond reached capacity, an interior dike was breached and sluiced ash was allowed to flow into the "future ash disposal area", currently designated Main Ash Pond A.

The Dredge Cell area originally identified as the Limestone Scrubber Pond was impounded by a dike having a crest elevation of 626 feet.

The original dikes impounding the Bottom Ash Stack, Main Pond A and Dredge Cell consists of compacted clay, likely excavated from on-site sources.

Gypsum Stack

The Gypsum Stack impoundment was constructed in the early to mid-1980s to store gypsum and fly ash from Units 7 and 8. The dike forming the Gypsum Stack impoundment consists of a rolled earth embankment.

A Stilling Pond was constructed along the southwest portion of the Gypsum Stack impoundment. The Stilling Pond was constructed as a rolled earth embankment having a crest elevation on the west dike of 620 feet. The west embankment of the Gypsum Stack dike acts as the east dike of the Stilling Pond.

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4.1.2 Significant Changes/Modifications in Design since Original Construction

Main Ash Complex

In the 1960s the capacity of the Main Ash Complex was increased by raising the impounding dike crest elevation form 626 feet to 636 feet. The Main Ash Pond A raised dike consists of compact clay. The raised dike was constructed beginning at the inside edge of the starter dike crest. As a result the subgrade of the raised dike consists of silt and ash.

The Bottom Ash Stack raised dike consists of compacted clay with the inside portion of the dike supported on sandy bottom ash. Based on data in the Phase 2 geotechnical report (See Appendix A – Doc 01) localized sections of the raised dike consist of stacked bottom ash. It is expected that those areas may have been excavated for various operating and maintenance activities, and backfilled with material from the bottom ash stack

The Upper and Lower Stilling Ponds were added in 1986. The Stilling Ponds receive discharge from the Main Ash Pond A siphon system. The Upper and Lower Pump Ponds are located at the southeast corner of Main Ash Pond A on the southwest side of the Dredge Cell. The Pump Pond was also added in 1986 and receives decant spillway discharge from the Lower Stilling Pond. Recirculation pumps feed water from the Pump Pond to the Condenser Cooling Water Intake. Three spillway pipes discharge water directly into the Tennessee River. The Pump Pond is located southeast of and adjacent to the Lower Stilling Pond.

The Red Water Pond, located along the west side of the Bottom Ash Stack and Main Ash Pond A was added in 2007 to collect surface water run-off and seepage from the CCR dikes, including the Copper Pond and Iron Pond.

The drawings indicate that the area currently occupied by the Copper and Iron Ponds, also referred to as the chemical ponds in the new impoundment, are included in the original ash storage area for Units 7 and 8. Drawings from the early 1980's show a new embankment constructed to form the chemical ponds and separate them from ash storage operations in the Bottom Ash Stack.

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Gypsum Stack

The Gypsum Stack and Stilling Pond dikes have not undergone significant changes or design modifications since their original construction.

4.1.3 Significant Repairs/Rehabilitation since Original Construction

As part of a general assessment of CCR impoundment stability TVA retained Stantec to conduct geotechnical explorations at the Main Ash Complex and the Gypsum Stack. Based on the recommendations presented in the Stantec report, TVA has undertaken a program to rehabilitate embankments where required. Concurrently TVA has retained an engineering firm to develop closure plans for the Main Ash Complex and the Gypsum Stack.

Main Ash Complex

During 2010 and 2011 rehabilitation work at the Main Ash Complex consisted of adding inverted seepage filters and lower slope buttresses along a section of the Main Ash Pond A starter dike. The work included:

- About 1,700 linear feet of rock buttress along the northeastern length of the starter dike exterior slope.
- About 3,100 linear feet of seepage blanket/rock buttress along the northwestern length on the starter dike exterior slope parallel to the Red Water Pond
- About 100 linear feet of riprap along the southeastern interior slope

During the same period the existing decant riser spillway system was taken out of service and replaced with a siphon system. The siphon system consists of six 12-inch diameter steel pipes extending through the impoundment dike to discharge into the Upper Stilling Pond.

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Gypsum Stack

During 2010 and 2011 rehabilitation work at the Gypsum Stack included adding seepage filters and /or stone buttresses along sections of the Gypsum Stack impoundment dike, as follows.

- Slope filter blankets/rock buttresses added:
  - below bench/perimeter road at elevation 620 feet
  - About 3,400 feet along northwest slope and northern corner
  - About 800 feet along the southeast slope
  - About 650 feet along southwest slope
- Seepage control blankets at:
  - Seep 20 - 120 feet long by 10 feet in elevation
  - Seep 27 - 250 feet long by 5 feet in elevation
  - Seep 28 –100 feet long by 5 feet in elevation
  - Seep 34 –35 feet long by 5 feet in elevation

4.2 SUMMARY OF OPERATIONAL PROCEDURES

4.2.1 Original Operational Procedures

Main Ash Complex

Original operations of the Main Ash Complex consisted of sluicing CCR, primarily bottom ash, to the area currently designated as the Bottom Ash Stack. As that area approached capacity, an interior divider dike was breached to allow a hydraulic connection with the area designated for future ash storage, currently designated Main Ash Pond A.

The Dredge Cell initially received sluiced gypsum/scrubber sludge discharged into the southwest corner of the impoundment.

Gypsum Stack

Initial operations consisted of sluicing gypsum into internal cells within the impoundment. Solids were allowed to settle and clarified excess water was decanted to the stilling pond.

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4.2.2 Significant Changes in Operational Procedures and Original Startup

Main Ash Complex

Changes in the operation of the Main Ash Complex were implemented to accommodate the increasing volume of material requiring storage. Originally designed to store sluiced ash from generating units 7 and 8, upon closure of Ash Area 1, ash from Units 1 – 6 was also sluiced to the Bottom Ash Stack. As the Bottom Ash Stack neared capacity, the "Future Ash Storage" section of Area 2 was opened to form Main Ash Pond A. Ash being sluiced to the Bottom Ash Stack was directed via two interior ditches from the south end of the Bottom Ash Stack northeastward to the Main Ash Pond A.

In the 1980's TVA changed to a Forced Oxidation Gypsum Scrubber and opened the Gypsum Stack across Widows Creek from the Dredge Cell. The Dredge Cell was then used to store ash from Main Ash Pond A.

Gypsum Stack

In the mid-1990s the operation was converted to a wet stacking method. An interior perimeter dike were constructed over wet ash to increase the capacity of the unit. Each stack layer included an internal drainage system to allow sluice water to drain from the stack in preparation for the next lift. Water from the internal drainage system was directed to perimeter ditches and discharged to the Gypsum Stack Stilling Pond.

As discussed in Section 4.2.3, an alternate stacking method is currently in use.

4.2.3 Current Operational Procedures

Main Ash Complex

Sluiced ash from the plant is discharged into the Bottom Ash Stack and directed to Main Ash Pond A via internal surface ditches. Main Ash Pond A interior dikes are used to route sluice flow to the northern end of the impoundment from where it flows southward into the main pond area. Photograph 4.2.3.1 shows the drainage ditch through the Bottom Ash Stack to Main Ash Pond A, and Photograph 4.2.3.2 shows the ditch discharge into the main stilling basin of Main Ash Pond A.

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Photograph 4.2.3.1: Ditch Draining Sluice Ash Through Bottom Ash Stack to Main Ash Pond A



Photograph 4.2.3.2: Sluice Ditch Through Bottom Ash Stack at Discharge to Main Ash Pond A.

The dredge cell is currently inactive and being allowed to dewater

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Gypsum Stack

Currently sluiced gypsum is discharged into either an east or west rim ditch; excavated and rolled into windrows. Once dried, the material is transported to interior cells at the top of the stack. A pond formed at the top of the stack serves as a clarifier pond for stormwater runoff at the top of the pond.

Stormwater runoff from the Gypsum Stack and discharge from the stack's internal drainage system is directed to the Gypsum Stack Stilling Pond located along the west side of the Gypsum Stack. Photograph 4.2.3.3 shows the Gypsum Stack perimeter ditch discharge pipes into the Gypsum Stack Stilling Pond.



Photograph 4.2.3.3: Gypsum Stack Drainage Collector Pipes Discharge to Gypsum Stack Stilling Pond

4.2.4 Other Notable Events since Original Startup

No additional information has been provided to Dewberry concerning other notable events impacting operation of the Main Ash Complex or the Gypsum Stack.

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5.0 FIELD OBSERVATIONS

5.1 PROJECT OVERVIEW AND SIGNIFICANT FINDINGS

Dewberry personnel Joseph P. Klein, III, P.E. and Frank B. Lockridge P.E. performed a site visit on Tuesday September 13, 2011 in company with persons listed in Section 1.3.1.

The site visit began at 9:00 AM. The weather was clear and warm. Photographs were taken of conditions observed. Please refer to the Dam Inspection Checklist in Appendix B for additional information. Selected photographs are included here for ease of visual reference. All pictures were taken by Dewberry personnel during the site visit.

The overall assessment of the CCR management units was that the units were in satisfactory condition and no significant findings were noted.

5.2 MAIN ASH COMPLEX

5.2.1 Crest

The crest of the embankment impounding the Main Ash Pond A, Bottom Ash Stack, and Dredge Cells comprising the CCR components of the Main Ash Complex is a gravel and crushed stone paved roadway. The crest had no signs of significant depressions, tension cracks, or other indications of settlement or shear failure. Photograph 5.2.1.1 shows the embankment crest along the west side of the Main Ash Complex. Photograph 5.2.1.2 shows the embankment crest along the west side of the Bottom Ash Stack. Photograph 5.2.1.3 shows the embankment crest along the south side of the Dredge Cell.

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Photograph 5.2.1.1: Embankment Crest at Main Ash Pond A - Northwest Corner



Photograph 5.2.1.2: Embankment Crest along West Side of Bottom Ash Stack with Stacked Ash on Right Side

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Photograph 5.2.1.3: Embankment Crest along South Side of Dredge Cell with Stacked Dredged Ash on Left

5.2.2 Upstream/Inside Slope

The inside slope of the Main Ash Pond A is armored with crushed stone to protect against wind related wave erosion. Photograph 5.2.2.1 shows the embankment interior slope at the northwest corner of Main Ash Pond A.

The interior slope of the Bottom Ash Stack is covered with ash to near the crest elevation making observation of the inside slope infeasible. Figure 5.2.2.2 shows the interior of the Bottom Ash Stack impoundment, including the ditch used to transport sluiced ash through the Bottom Ash Stack to the Main Ash Complex.

The perimeter portion of the Dredge Cell adjacent to the embankment has been filled with ash which has covered much of the inside slope of the embankment. The exposed portion of the embankment slope is covered with grass and weeds. Figure 5.2.2.3 shows a section of the inside slope.

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Photograph 5.2.2.1: Embankment Interior Slope at Northwest Corner of Main Ash Pond A



Photograph 5.2.2.2: Embankment Inside Slope along West Side of Bottom Ash Stack. Ditch Carrying Sluiced Ash to Main Ash Complex Shown on Left

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Photograph 5.2.2.3: Embankment Inside Slope at South End of Dredge Cell (Inside Slope on Left Side of Crest Roadway)

5.2.3 Downstream/Outside Slope and Toe

The outside slopes of the Main Ash Complex embankment are vegetated with several species of grass and weeds. Photographs 5.2.3.1 and 5.2.3.2 show the typical vegetation on the impoundment outside slopes.



Photograph 5.2.3.1: Main Ash Complex Embankment Outside Slope East Area of Dredge Cell North Dike

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Photograph 5.2.3.2: Main Ash Complex Embankment Outside Slope Northwest Corner of Main Ash Complex.

Based on findings presented in the Stantec geotechnical report (See Appendix A – Doc 01) inverted filter drain/buttresses have been constructed along the starter dike portion of three sections of the outside slopes.

Four previously identified seepage areas on the outside slope have been repaired using inverted filter drains.

Photograph 5.2.3.3 shows a section of embankment outside slope with both the inverted filter drain/buttress and an inverted filter drain.

Photograph 5.2.3.4 shows a seepage area with a vegetated seepage blanket previously identified from continued monitoring.

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Photograph 5.2.3.3: Outside Embankment near Southeast Corner Main Ash Complex - Inverted Filter Drain/Buttress at Bottom on Slope and Inverted Filter Drain (Seep #18) at Mid-Slope. Widows Creek Located along Toe of Slope.



Photograph 5.2.3.4: Seepage Area with Vegetated Filter Blanket. Seepage #35 located at South End of Dredge Cell.

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No significant, unreported seepage was observed along the toe of the embankment during Dewberry's site visit. Areas of soft, wet soil and/or standing water appeared to be the result of recent heavy rains in the day preceding the site visit. As shown in Photograph 5.2.3.5 one area of potential seepage was observed at the toe of the embankment at the southwest corner of the Bottom Ash Pond



Photograph 5.2.3.5: Possible Seepage Area at Bottom Ash Stack Embankment Toe

5.2.4 Abutments and Groin Areas

The Main Ash Complex is formed by a dike on four sides; therefore, there are no abutments. Neither erosion nor uncontrolled seepage was observed along the groins. Photographs 5.2.4.1 and 52.4.2 show typical conditions observed at the outside groins.

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Photograph 5.2.4.1: Outside Groin at Northwest Corner of Main Ash Complex



Photograph 5.2.4.2: Outside Groin at Southwest Corner of Bottom Ash Stack

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5.3 GYPSUM STACK

5.3.1 Crest

The crest of the stacked ash portion of the Gypsum Stack is gravel and crushed stone paved roadway. The crest of the Stilling Pond has a compacted ash roadway. The crest had no signs of significant depressions, tension cracks, or other indications of settlement or shear failure. Photograph 5.3.1.1 and 5.3.1.2 show typical conditions observed along the crest of the embankment around the gypsum stack and Stilling Pond.



Figure .3.1.1: Embankment Crest at North Side of Gypsum Stack. Stacked Gypsum to left of Gravel paved Crest Roadway.

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Photograph 5.3.1.2: Embankment Crest on West Side of Stilling Pond

5.3.2 Upstream/Inside Slope

Except for the Stilling Pond, dry ash is stacked to an elevation above the crest of the embankment throughout the Gypsum Stack: therefore, observation of the inside slope of the embankment is infeasible.

The inside slope of the Stilling Pond is armored with crushed stone riprap to protect against erosion caused by windblown waves. Photograph 5.3.2.1 shows interior slopes of the Stilling Pond.

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Figure 5.3.2.1: Interior Slopes of Gypsum Stack Stilling Pond View to South.

5.3.3 Downstream/Outside Slope and Toe

Based on findings presented in the Stantec geotechnical report (See Appendix A – Doc 02) inverted filter drain/buttresses have been constructed along three sections of the outside slopes. The approximate length and location of the inverted filter drain/buttress sections are:

- 3,400 feet along north section of west slope and northwest corner
- 800 feet along the southeast corner
- 650 feet along the southeast slope

Other sections of the embankment are vegetated with various species of grass and weeds. Photographs 5.3.3.1 and 5.3.3.2 show typical outside slope conditions observed during Dewberry's site visit. Photograph 5.3.3.3 shows a section of inverted filter drain under construction.

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Figure 5.3.3.1: West Embankment Outside Slope with Inverted Filter Drain and Stone Buttress.



Photograph 5.3.3.2: South Embankment Outside Slope at Transition from Inverted Filter Drain to Vegetated Slope.

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Photograph 5.3.3.3: Construction of Inverted Filter Drain Used to Control Seepage

Small- to medium-sized trees were observed along the north end of the east embankment. It was unclear if the trees were on natural ground or the slope of the starter dike. The trees are shown in Photograph 5.3.3.4



Photograph 5.3.3.4: Trees along North Section of East Embankment

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No significant, unreported seepage was observed along the embankment or toe area during Dewberry's site visit. Areas of soft wet soils and/or standing water observed appeared to be the result of recent heavy rains shortly before the site visit. Photograph 5.3.3.5 shows a typical soft, wet area observed at the toe along the embankment.



Photograph 5.3.3.5: Soft, Wet Surface Soils at Toe near South End of West Embankment

5.3.4 Abutments and Groin Areas

Erosion or uncontrolled seepage was not observed along embankment groins or abutments. Photograph 5.3.4.1 shows the exterior slope at the northeast groin with the terminus of the north embankment inverted filter drain and buttress.

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Photograph 5.3.4.1: Exterior Groin with Inverted Filter Drain/Buttress at Northeast Corner of impoundment.

5.4 OUTLET STRUCTURES

5.4.1 Overflow Structure

The Main Ash Complex original overflow structure consisted of five 48-inch diameter concrete risers each connected to a 36-inch diameter concrete outlet pipe installed through the embankment to discharge to the Upper Stilling Pond.

In 2011 the spillway system was changed as part of a program to reduce the potential hazard posed by a release due to a failure or misoperation of the impoundment. The change consisted on taking the riser spillway out of service and constructing a new six pipe siphon system.

Seepage water from the stacked gypsum is collected through an extensive internal drainage system and discharge to perimeter ditches. Surface water runoff and collected internal drainage discharge is directed into the Stilling Pond. The overflow structure for the Stilling Pond consists of a 30-inch diameter riser pipe protected by steel, and a 42-inch diameter skimmer barrel. Photograph 5.4.1.1 shows the Stilling Pond overflow structure.

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Photograph 5.4.1.1: Gypsum Stack Stilling Pond Riser Spillway

5.4.2 Outlet Conduit

The Main Ash Pond A siphon system discharges into the Upper Stilling Pond. Photograph 5.4.2.1 shows discharge from the siphon pipes to the Upper Stilling Pond.



Photograph 5.4.2.1: Siphon Pipe Discharge from Main Ash Complex to Upper Stilling Pond.

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The Gypsum Stack Stilling Pond spillway discharges from a 30-inch diameter concrete pipe into a partially lined ditch. The ditch flows into Widows Creek. Photographs 5.4.2.2 and 5.4.2.2 show the outlet pipe discharge and the discharge ditch, respectively.



Photograph 5.4.2.2: Gypsum Stack Stilling Pond Spillway Outfall.



Photograph 5.4.2.3: Gypsum Stack Stilling Pond Spillway Discharge Ditch Leading to Widows Creek

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5.4.3 Emergency Spillway

The Main Ash Complex does not have an emergency spillway.

A small low area in the north section of the Gypsum Stack Stilling Pond crest serves as an emergency spillway. Design is underway to deepen the emergency spillway section, line the channel with crushed stone and direct flow into Widows Creek. Photograph 5.4.3.1 shows the current area serving as the emergency spillway.



Photograph 5.4.3.1: Area of Gypsum Stack Stilling Pond Serving as Emergency Spillway.

5.4.4 Low Level Outlet

The Main Ash Complex does not have a low level outlet.

The Gypsum Stack Stilling Pond does not have a low level outlet.

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6.0 HYDROLOGIC/HYDRAULIC SAFETY

6.1 SUPPORTING TECHNICAL DOCUMENTATION

6.1.1 Flood of Record

No documentation has been provided about the flood of record.

6.1.2 Inflow Design Flood

TVA retained Stantec to conduct hydrologic and hydraulic analyses of the Main Ash Complex and the Gypsum Stack Stilling Pond (See Appendix A – Docs 09 and 10 respectively). The analyses included the 24-hour, 100-year storm (one percent probability of occurrence in any given year) and the 6-hour probable maximum precipitation (PMP) event.

The hydraulic analyses were based on the following assumptions:

- All process waters from the plant enter the impoundments normally
- All rainfall within the embankment perimeter flows into the impoundment
- No infiltration occurs

The results of the hydrologic and hydraulic analyses are summarized in Table 6.1.

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Table 6.1: Hydrologic and Hydraulic Analyses Summary				
	Main Ash Complex		Gypsum Stack Stilling Pond	
	100-Yr 24-Hour Storm	6-Hour PMP Storm	100-Yr 24-Hour Storm	6-Hour PMP Storm
Drainage Area (acres)	323		143	
Dam Crest Elev. (feet)	635.8		620	
Normal Pool Elev. (feet)	632.4		613.7	
Normal Operation Flow (MGD)	50		8.2	
Precipitation Depth (inches)	7.5	36.3	7.4	36.3
Maximum Water Surface Elev. (feet)	633.1	Overtopping	615.7	620.6 Overtopping
Flood Event Freeboard (feet)	2.7	Overtopping	4.3	Overtopping

6.1.3 Spillway Rating

No spillway hydraulic data were provided for review.

The overflow riser spillway configuration used in the Main Ash Complex hydraulic analysis has been taken out of service and replaced with a newly constructed siphon system. The Main Ash Pond A analysis report (See Appendix A – Doc 09) indicates that design of the system was underway by others at the time the hydraulic analysis was conducted.

6.1.4 Downstream Flood Analysis

TVA commissioned a dam breach analysis to evaluate the potential impact of a release due to a failure or misoperation of the Gypsum Stack (See Appendix A – Doc 03). The results indicated that a release would reach a limited number of residences and rural roads. TVA acquired and demolished the residences and conducted a survey of roadway elevations. The survey indicated the roadway elevations were above the anticipated flood depth.

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6.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Supporting documentation reviewed by Dewberry is adequate.

6.3 ASSESSMENT OF HYDROLOGIC/HYDRAULIC SAFETY

Based on the hydraulic analyses both the Main Ash Complex and the Gypsum Stilling Basin can retain the 24-hour, one percent probability storm event with a freeboard of 2.7 feet and 4.3 feet respectively (See Appendix A – Docs 09 and 10). Although the calculated freeboard is less than TVA's current design standard of 5-feet, the freeboard is considered satisfactory.

As previously discussed the spillway used for the Main Ash Complex hydraulic analysis has been replaced with a new six pipe siphon system. It is reasonable to expect that the new system was designed to meet the TVA 5-foot of freeboard standard during the 24-hour, 100-year storm event; however, documentation was not provided concerning the new spillway design.

The hydrologic and hydraulic analyses concluded that the Probable Maximum Flood (PMF) results in overtopping both the Main Ash Complex and the Gypsum Stack Stilling Pond. Federal guidelines allow selecting an inflow design flood less than the PMF based on policies established by statues specifying a specific fraction of the PMF. Although Alabama does not have a dam safety program, Tennessee regulations establish a design flood criteria of 1/3 PMP for Significant Hazard, small dams. As discussed in section 2 of this report, both the Main Ash Complex and Gypsum Stack Stilling Basin are classified as significant hazard, small dams.

Based on the conservative use of the PMF in the hydrologic and hydraulic analyses, the results of the dam break analysis, and the actions implanted by TVA to reduce the risk due a release caused by a failure or misoperation of the embankments, the hydrologic and hydraulic safety is considered SATISFACTORY.

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7.0 STRUCTURAL STABILITY

7.1 SUPPORTING TECHNICAL DOCUMENTATION

7.1.1 Stability Analyses and Load Cases Analyzed

Slope stability analyses were conducted by Stantec as part of the geotechnical explorations for the Main Ash Complex and the Gypsum Stack Pond (See Appendix A – Docs 01 and 02 respectively). The analyses used soil properties and shear strength values based on geotechnical borings and laboratory testing conducted as part of the explorations.

The stability analyses in the geotechnical reports considered static, long-term loading with steady-state flow parameters.

The analyses indicated slope stability Safety Factor deficiencies at the east and west dikes of the Dredge Cell embankment, and at the northeast section of the Main Ash Complex embankment. TVA retained URS Corp. to develop design alternatives to improve the long term static slope stability of those embankments. The improvement alternative selected consisted of adding toe buttresses to enhance embankment stability. An inverted graded filter blanket was incorporated into the design for the northern portion of the Main Ash Complex dike (See Appendix A – Doc. 14 and 15)

Based on discussions between TVA and EPA during the current assessment, TVA retained Stantec to conduct additional analyses to evaluate slope stability under seismic loading conditions (See Appendix A Doc 11). The initial seismic analyses performed represented the lowest current long-term factor of safety. Ground motion was based on an exceedance probability of 10 percent in 50 years.

Based on further discussions between TVA and EPA additional seismic analyses were conducted using a peak ground acceleration with a probability of exceedance of two percent in 50 years (See Appendix A – Doc 16).

7.1.2 Design Parameters and Dam Materials

Documentation provided to Dewberry for review indicated the stability analyses assumed six strata for the Main Ash Complex, and seven strata for the Gypsum Stack. The material properties used in the analyses are shown in Table 7.1.

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Table 7.1: Summary of Soil Properties Used in Stability Analyses						
Soil Strata	**Static Analysis**			**Seismic Analysis**		
	Unit Weight γ' (pcf)	Cohesion c' (psf)	Friction Angle Ø'	Unit Weight γ (pcf)	Cohesion c (psf)	Friction Angle Ø
Main Ash Complex						
Silty Sand & Gravel – Bottom Ash	119	0	33	119	0	33
Silt with Sand – Bottom Ash	112	0	33	112	0	33
Silty Sand – Fly Ash	112	0	27.5	112	0	21.8
Fill –Clay with Gravel	125	0	32	125	1,375	14.2
Residual Clay	125	0	32	125	650	15.7
Riprap Buttress	115	0	40	115	0	40
Gypsum Stack						
Cast Gypsum	113	0	40	113	0	40
Sedimented Gypsum	112	0	41	112	0	41
Weak Sedimented Gypsum	108	0	27.5			
Fat Clay	123	0	25	123	650	15.7
Crushed Stone	125	0	35			
Bedrock	125	0	N/A			
Sand Drains	Placed After Analysis			110	0	33

7.1.3 Uplift and/or Phreatic Surface Assumptions

The stability documentation provided to Dewberry did not specifically identify uplift forces acting on the base of the dike.

Embankment pore pressures for the long-term static loading condition were obtained from the seepage analyses conducted as part of the geotechnical explorations (See Appendix A – Docs 01 and 02).

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7.1.4 Factors of Safety and Base Stresses

The safety factors computed for the Main Ash Complex and Gypsum Stack are summarized in Table 7.2.

Table 7.2 Factors of Safety for Widows Creek Fossil Plant CCR Impoundment Embankments

Table 7.2: Slope Stability Factors of Safety Widows Creek Fossil Plant				
Location	**Long Term Static Loading**		**Seismic Loading**	
	Require Safety Factor (US Army Corps of Engineers	**Computed Minimum Safety Factor**	**Require Safety Factor (US Army Corps of Engineers**	**Computed Minimum Safety Factor**
Main Ash Complex				
Main Ash Complex	1.5	1.62[1]	>1.0	1.8[1]
Bottom Ash Pond	1.5	1.7	>1.0	1.9
Dredge Cell	1.5	1.50[2]	>1.0	1.5
Gypsum Stack[3]	1.5	1.7[4]	>1.0	1.9[5]

[1] Includes effects of riprap buttress and seepage filter blanket constructed after geotechnical exploration (see Appendix A – Doc 14)
[2] Includes effects of riprap buttress constructed after geotechnical exploration (See Appendix A – Doc 15)
[3] Computed minimum factor of safety found in stacked gypsum, not in impoundment dike.
[4]Minimum factor of safety sliding surface occurs in stacked gypsum slope below bench at elevation 656 feet.
[5]Minimum factor of safety sliding is for a deep surface from current top of slope elevation 668.

At the time of Dewberry's visit the remedial work for the Main Ash Complex was underway, and work at the Dredge Cell was completed.

7.1.5 Liquefaction Potential

The documentation reviewed by Dewberry did not include an evaluation of liquefaction. Soil types and conditions provided in the geotechnical reports (See Appendix A – Docs 01 and 02) indicate the soils supporting the starter dike and Main Ash Complex, and the Gypsum Stack Perimeter

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Dike are not expected to be susceptible to liquefaction. However, the Main Ash Pond A raised dikes consist of sandy silt, and gravelly sand-size bottom ash that are potentially susceptible to liquefaction. Significant sections of the Main Ash Pond A raised dike are supported on silt-size fly ash, which is also expected to be susceptible to liquefaction.

7.1.6 Critical Geological Conditions

The Widows Creek Fossil Plant is located in the Sequatchie Valley District of the Appalachian Plateau Physiographic Province. The site is underlain by a complex series of interbedded limestone and shale mapped as the Sequatchie Formation, Nashville Group, and Stone River Group. Shallow soils include alluvial deposits adjacent to the Tennessee River.

As indicated in the Gypsum Stack geotechnical report (See Appendix A – Doc 02) the USGS National Seismic Hazard Map for the Central and Eastern United States estimates the peak ground acceleration for the 10 percent probability of exceedance in 50 years as 0.058g. The seismic slope stability analyses (See Appendix A – Doc 11) used a ground acceleration of 0.038g as the 10 percent in 50 year exceedance value based on a hazard study report conducted by AMEC Geometrix, Inc. Dewberry has not received the AMEC Geometrix for review.

7.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Structural stability documentation is adequate to support the results and conclusions provided. However, the susceptibility of soils to liquefaction resulting from a seismic event has not been assessed.

7.3 ASSESSMENT OF STRUCTURAL STABILITY

Overall, the structural stability of the dams for the Main Ash Complex management unit appears to be FAIR based on the lack of documentation of liquefaction potential affecting the embankments. The Gypsum Stack is rated SATISFACTORY.

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8.0 ADEQUACY OF MAINTENANCE AND METHODS OF OPERATION

8.1 OPERATING PROCEDURES

The operation and maintenance procedures for the Widows Creek CCR impoundments are defined in the Tennessee Valley Authority (TVA) Routine Handling Operations and Maintenance (RHO&M) Operating Support Document, Widows Creek Fossil Plant, July 2011 (appendix A - Doc 13).

Main Ash Complex

The Main Ash Complex consists of three primary units:

- Main Ash Complex
- Bottom Ash Stack
- Dredge Cell

The Main Ash Complex is configured to receive fly ash and bottom ash sluiced from Units 1 through 6 discharged to a ditch at the eastern side of the Bottom Ash Stack, and bottom ash from Units 7 and 8 discharged to a ditch at the western side of the Bottom Ash Stack. The ditches merge at the southwest corner of the Main Ash Complex. The single ditch flows through precipitated ash to the north corner of the Main Ash Complex where it is directed south into the main stilling basin. Water elevation in the Main Ash Complex is controlled by a newly construction siphon system located in the southwest corner of the pond.

The Bottom Ash Stack currently is being operated as a storage area for stacked ash and as the route of ditches transmitting sluiced ash to the Main Ash Complex.

The Dredge Cell is currently inactive.

Gypsum Stack

The Gypsum Stack currently receives Flue Gas Desulfurization gypsum – fly ash materials from Plant Units 7 and 8 via a system of six 18-inch diameter HDPE pipes that discharge to either the east or west rim ditches. Discharge is alternated between the ditches. Material from the ditch not receiving discharge is excavated and rolled into windrows. Once dried, the material is transported to cells in the stack.

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8.2 MAINTENANCE OF THE DAM AND PROJECT FACILITIES

Inspection reports provided to Dewberry for review identified seepage through embankments as the most significant maintenance issue. In response, Widows Creek Fossil Plant retained Stantec to prepare a maintenance plan addressing seepage (See Appendix A – Doc 12). Dewberry was provided a copy of the plan for review.

During Dewberry's site visit the embankments were generally clear of vegetation, except along the west side of the Main Ash Complex/Bottom Ash Stack where small- to medium-sized trees were observed. The embankment crests were free of cracks, scarps, or other visual indications of distress.

Seepage areas identified during routine inspections were marked in the field with flags. Most of the seepage areas had been repaired by installation of an inverted filter blanket. Scattered seepage areas were marked and being monitored. Dewberry engineers did not identify any significant seepage areas that were not previously identified by TVA.

8.3 ASSESSMENT OF MAINTENANCE AND METHODS OF OPERATIONS

8.3.1 Adequacy of Operating Procedures

Based on the assessments of this report, operating procedures appear to be adequate.

8.3.2 Adequacy of Maintenance

Based on the assessments of this report, maintenance procedures appear to be adequate.

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9.0 ADEQUACY OF SURVEILLANCE AND MONITORING PROGRAM

9.1 SURVEILLANCE PROCEDURES

The TVA surveillance procedures for the Widows Creek Fossil Plant CCR impoundments include documented routine inspections daily, weekly, monthly, quarterly and annually. Special inspections are required after significant storms (i.e. rain events exceeding the 10-year recurrence intensity, and after significant earthquakes).

Reports of daily, weekly, monthly, quarterly and annual inspections conducted in 2011 were provided to Dewberry for review (See Appendix A – Doc 06 and 07)

9.2 INSTRUMENTATION MONITORING

The embankments for both the Main Ash Complex and the Gypsum Stack impoundments are monitored with slope inclinometers and groundwater piezometers. Reports of monitoring results were provided to Dewberry for review (See Appendix A – Docs 01 and 02).

9.3 ASSESSMENT OF SURVEILLANCE AND MONITORING PROGRAM

9.3.1 Adequacy of Inspection Program

Based on the data reviewed by Dewberry, including observations during the site visit, the inspection program is adequate.

9.3.2 Adequacy of Instrumentation Monitoring Program

Based on the data reviewed by Dewberry, including observations during the site visit, the instrumentation monitoring program is adequate.

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APPENDIX A

Document 1

Phase 2 Geotechnical Exploration, Ash Pond Complex, Widows Creek Fossil Plant, Stevenson Alabama, Stantec Consulting Services, February 4, 2010

Report of Phase 2 Geotechnical Exploration

Ash Pond Complex
Widows Creek Fossil Plant
Stevenson, Alabama

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One Team. Infinite Solutions
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Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

February 4, 2010

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February 4, 2010 rpt_003_175569036

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

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

Dear Mr. Snider:

As requested, Stantec Consulting Service Inc. (Stantec) has completed our Geotechnical Exploration for the Ash Pond Complex at the Widows Creek Fossil Plant. This report documents the subsurface conditions encountered during the exploration, results of laboratory testing, findings from the historical document reviews, results of our analyses and evaluation, and recommendations for the facility. These services were performed under Engineering Service Request ESR/TAO 909 in accordance with the terms and provisions established in our System-Wide Services Agreement dated December 22, 2008.

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

Sincerely,

STANTEC CONSULTING SERVICES INC.

Robert D. Fuller, PE
Project Manager

/cmw/rdr



Report of Phase 2 Geotechnical Exploration

Ash Pond Complex
Widows Creek Fossil Plant
Stevenson, Alabama

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

February 4, 2010

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

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Executive Summary

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

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

Stability Analysis Results for As Found Conditions

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

Seepage Analysis Results for As Found Conditions

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

Stability Analysis Results for Final Ash Stack Height

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

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

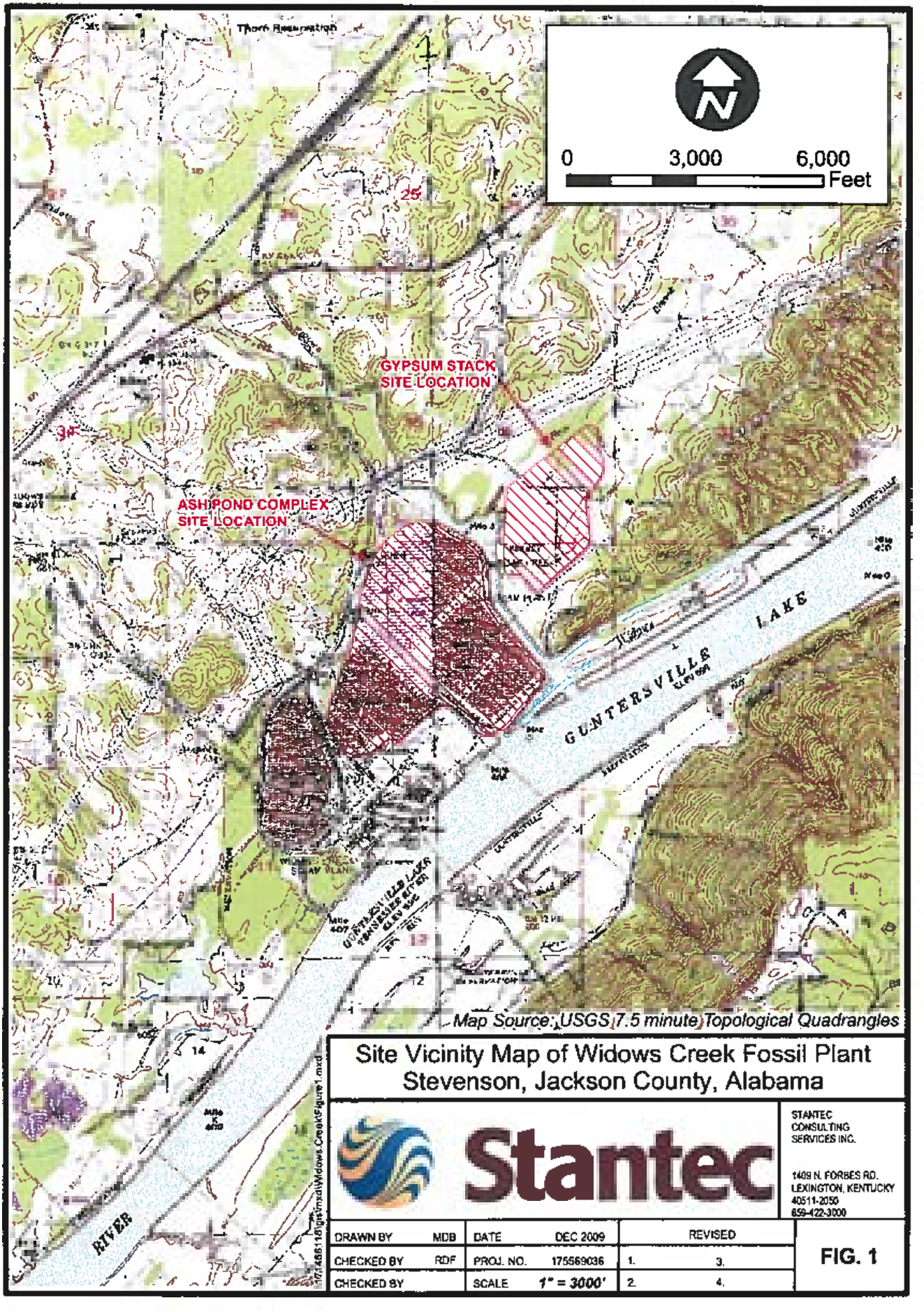
Report of Phase 2 Geotechnical Exploration
Ash Pond Complex
Widows Creek Fossil Plant
Stevenson, Alabama

1. Introduction

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

2. Ash Pond Complex

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



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

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

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

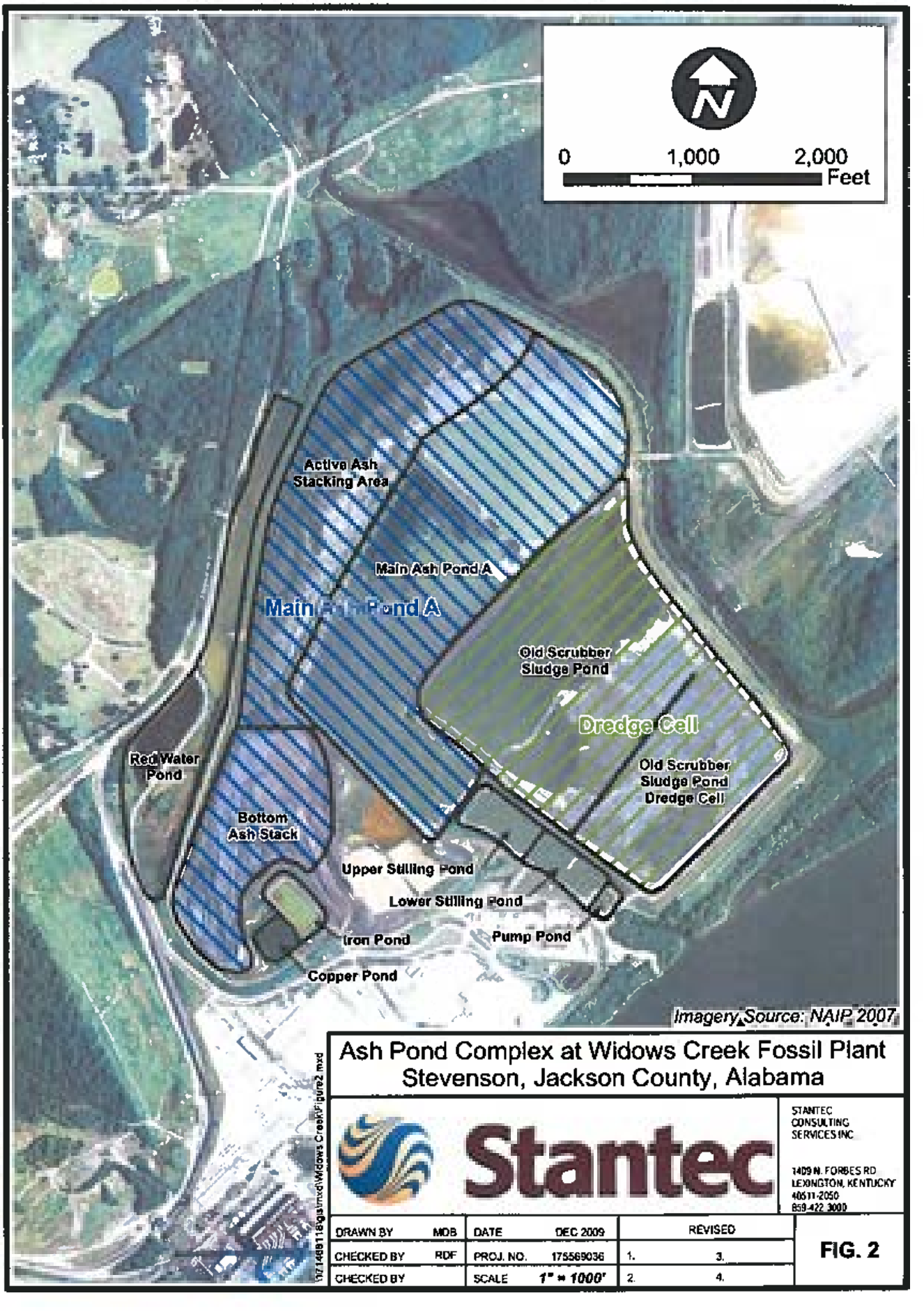
Table 1. Structures Within Ash Pond Complex

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

2.1.1. Dredge Cell

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

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



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

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CHECKED BY	RDF	PROJ. NO.	175569036	1.	3.
CHECKED BY		SCALE	1" = 1000'	2.	4.

FIG. 2

\\175569036\gis\mxd\Widows Creek\Figure2.mxd

Table 2. Former Names of the Dredge Cell

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

NA = Not Available

2.1.2. Red Water Pond

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

2.1.3. Upper/Lower Stilling Pond

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

2.1.4. Pump Pond

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

2.1.5. Main Ash Pond A

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

2.1.6. Bottom Ash Stack

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

3. Review of Available Information

3.1. General

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

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

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

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

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

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

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

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

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

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

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

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

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

3.2. Site History

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

Table 3. Summary of Events

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

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

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

3.3. 1952 Conceptual Design Recommendations

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

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

3.4. 1973 Record Modification

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

3.5. 1978 Chemical Treatment Ponds

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

3.6. 1980 Dredge Cell IFR Drawings

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

3.7. 1981 and 1983 Geotechnical Exploration

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

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

3.8. 1984 Slurry Cutoff Wall Addition

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

3.9. 1987 Redwater Containment Dike/Spillway Improvements

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

3.10. 2001 Preliminary Engineering Scope

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

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

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

3.11. 2003 and 2004 Mactec Geotechnical Exploration

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

3.12. Ash Pond Complex Design Layouts

3.12.1. Ash Disposal Area No. 1

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

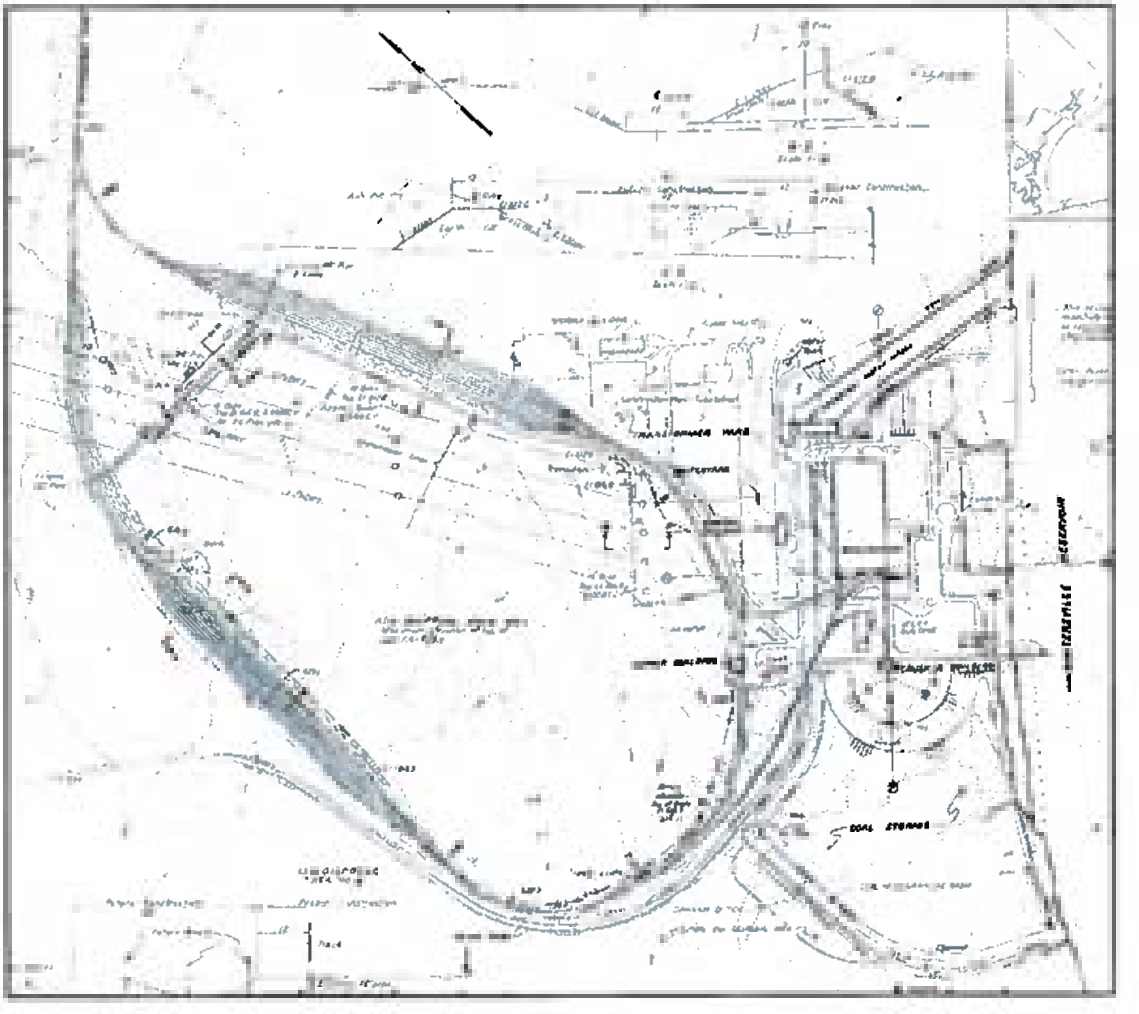


Figure 3. Ash Disposal Area, 1952

3.12.2. Ash Disposal Area No. 2

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

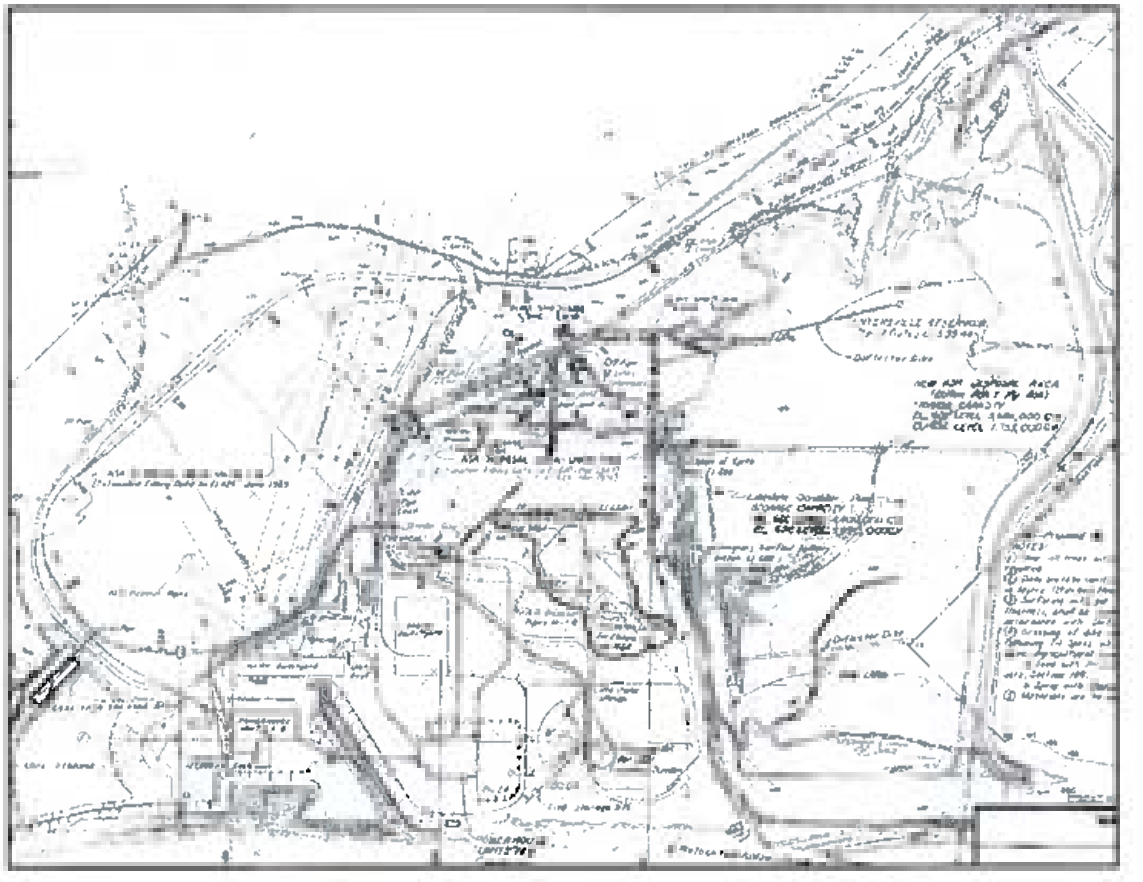


Figure 4. Ash Disposal Area, 1959

3.12.3. 1973 Dike Raising Typical Section

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

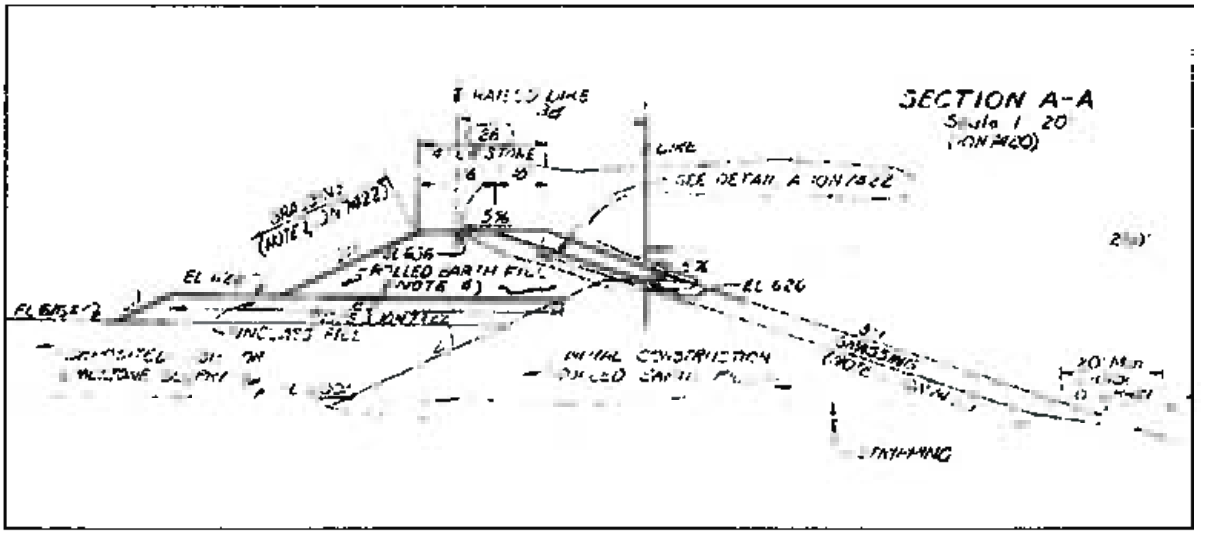


Figure 5. Typical Section 1973 Dike Raise

3.12.4. Slurry Cutoff Wall Layout

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

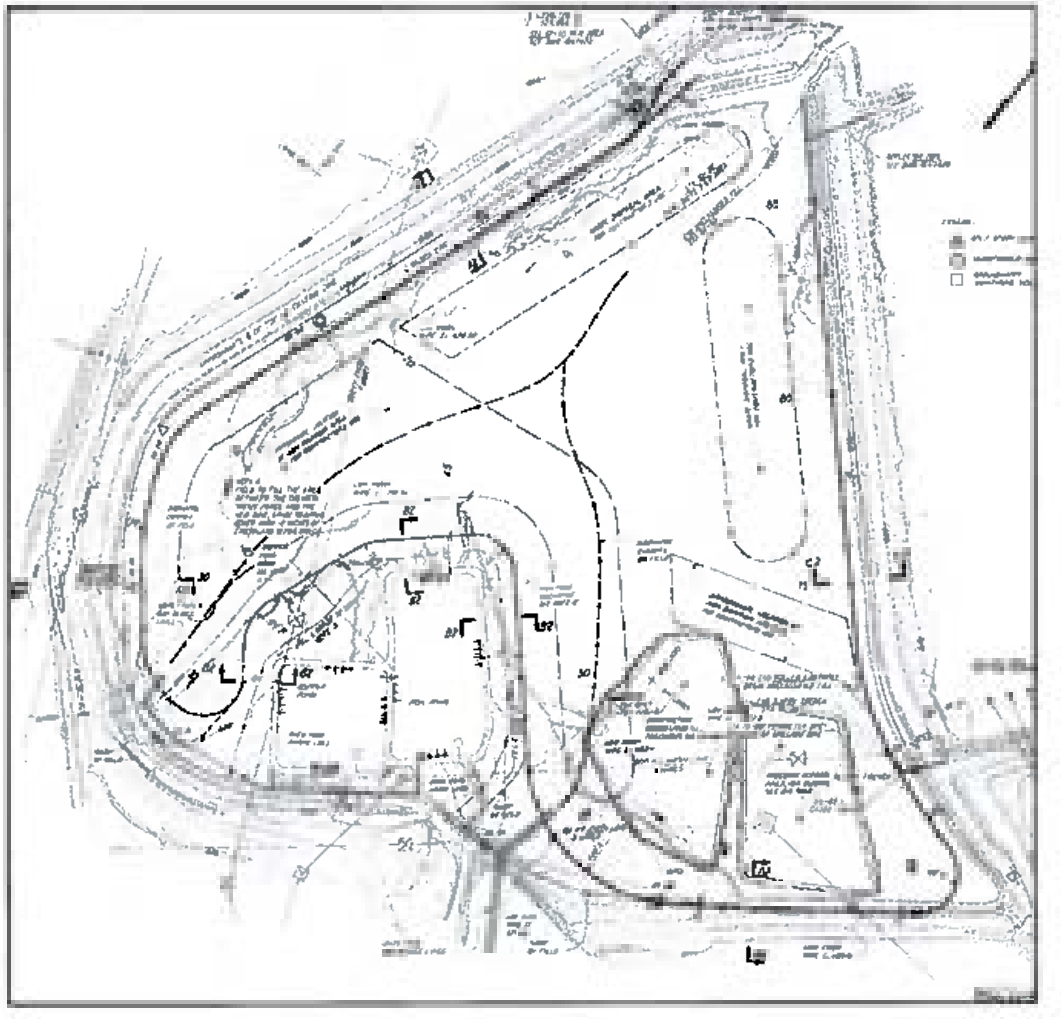


Figure 6. Slurry Cutoff Wall Plan View

3.12.5. Slurry Cutoff Wall Typical Section

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

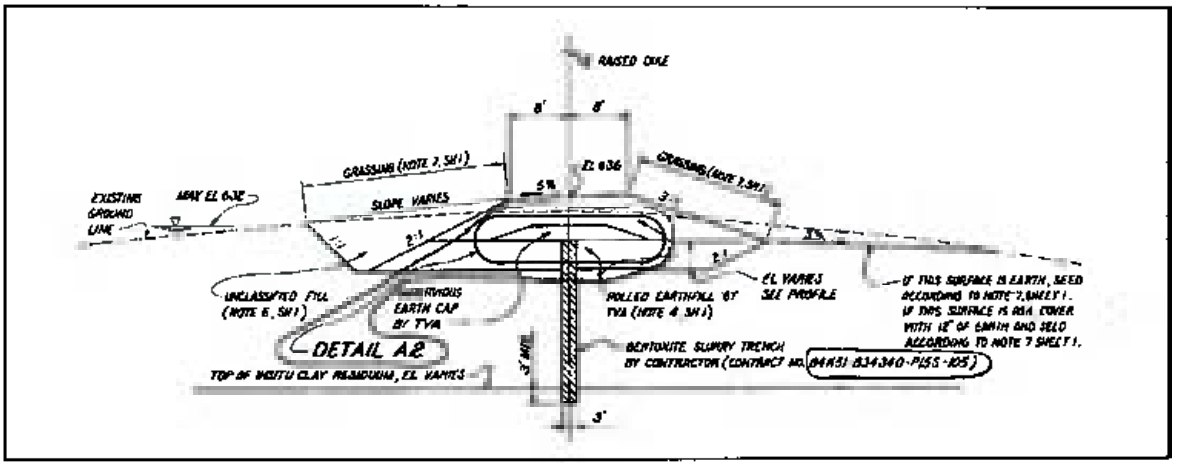


Figure 7. Slurry Cutoff Wall Typical Section

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

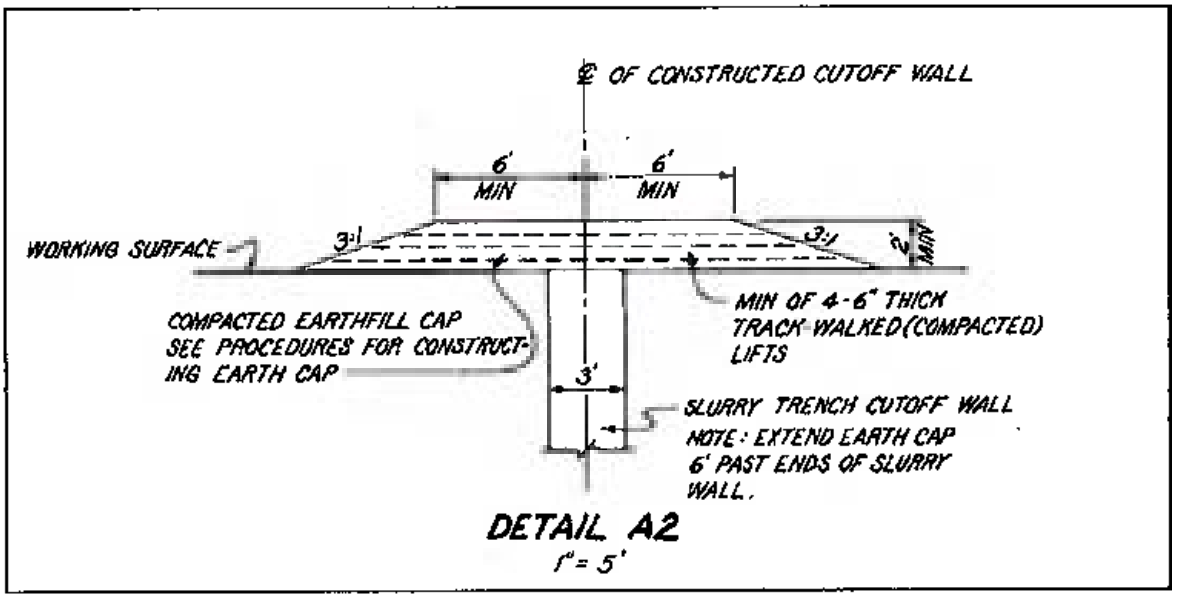


Figure 8. Slurry Cutoff Cap

3.12.6. Future Scrubber Sludge Pond Underdrain System

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

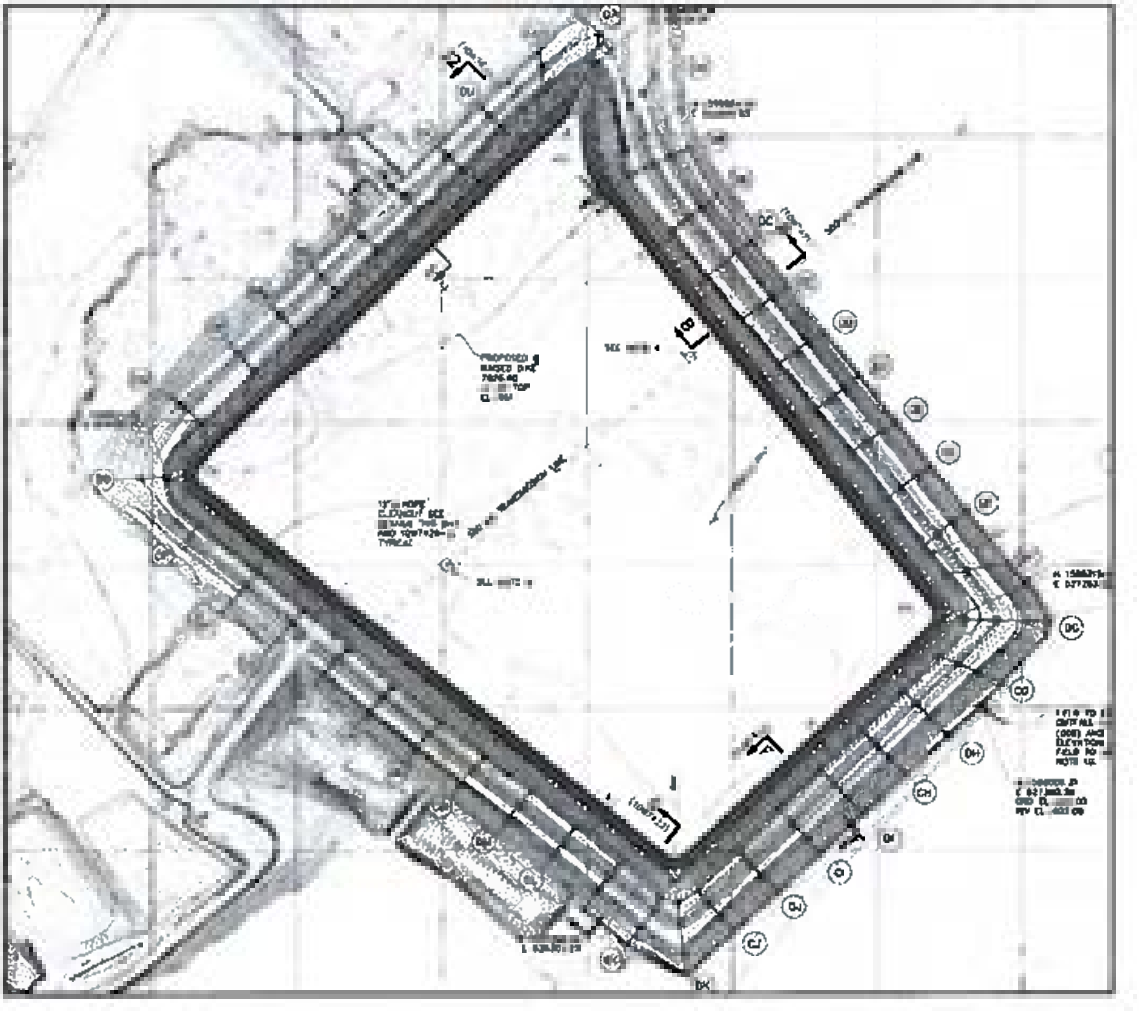


Figure 9. Scrubber Sludge Pond Underdrain Plan View

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

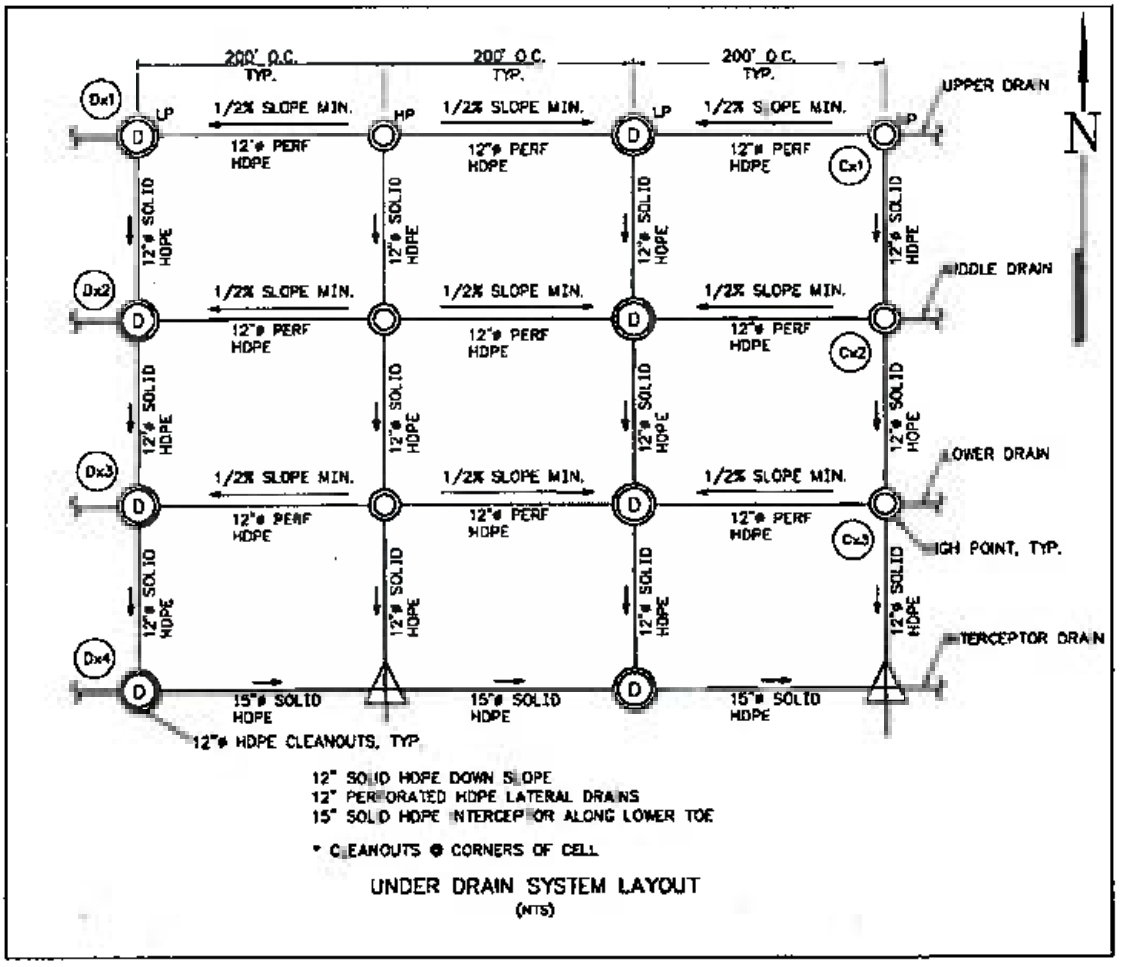


Figure 10. Scrubber Sludge Pond Underdrain Layout

3.12.7. Scrubber Sludge Pond Typical Section with Underdrains

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

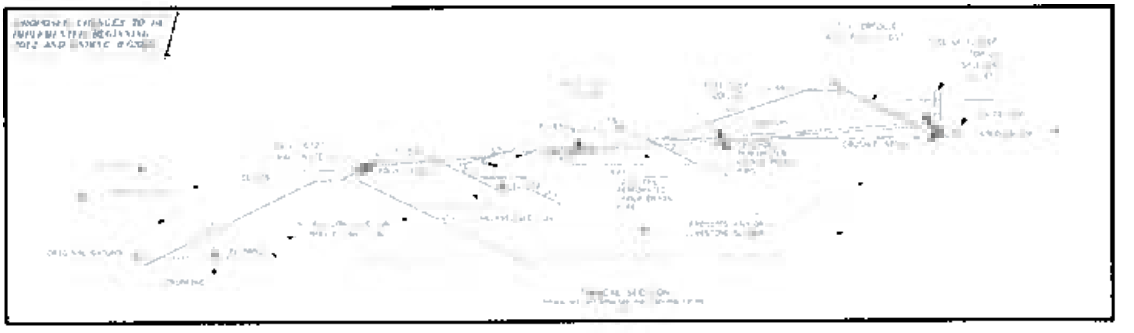


Figure 11. Typical Section, 2006

3.13. Surface Water Control

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



Figure 12. Typical Rill Erosion, 2009



Figure 13. Typical Surface Ditch, 2009



Figure 14. Typical Roadway/Surface Rills, 2009



Figure 15. Erosion Along Crest of Dredge Cell, 2009



Figure 16. Erosion Along Crest of Dredge Cell, 2009

3.14. Record Drawings

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

3.15. O&M Manual

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

3.16. Annual Reports

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

3.17. Underdrain System

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

3.18. Interim Risk Reduction Measure - Seepage Control

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



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



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

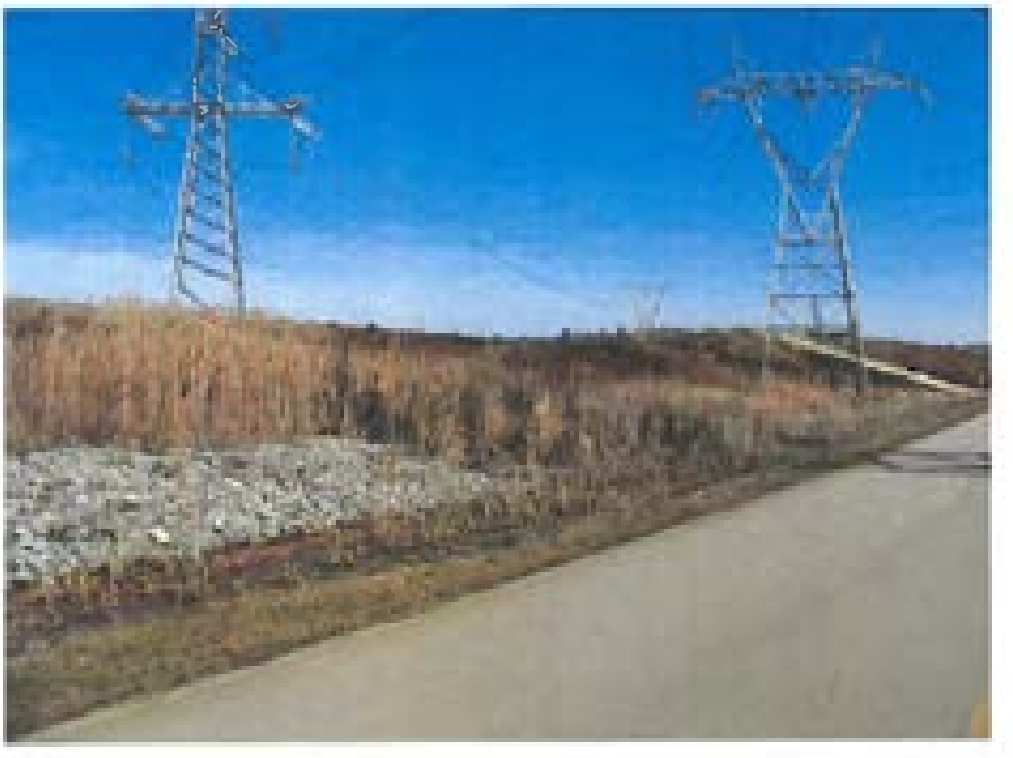


Figure 19. Ditch/Seepage Conditions Scrubber Road, 2009



Figure 20. Discharge Pipe along Scrubber Road, 2009



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

4. Scope of Work

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

The fieldwork for the geotechnical exploration was completed in late June, 2009 with the exception of one boring (STN-113) which was drilled by the Limestone Runoff Surge Pond. The scope of work included advancing a total of fifty-three (53) auger sample borings, eight (8) vane shear borings, and nine (9) CPT borings across the site at the approximate locations shown on the attached boring layout in Appendix C. The borings were drilled using a truck-mounted drill rig equipped with 4¼-inch (ID) hollow stem augers and NQ size rock coring equipment. All of the boring locations were staked in the field by Stantec personnel prior to drilling. Continuous standard penetration testing (SPT) were performed in most of

the borings advanced at the site, while undisturbed sampling, rock core sampling, cone penetration testing, and vane shear testing were performed in selected locations. Of the fifty-three (53) sample borings a total of thirty-nine (39) were instrumented with the following equipment; two (2) were instrumented with slope inclinometer casing and thirty-seven (37) were instrumented with standpipe piezometers with five-foot slotted screens. The slope inclinometers were installed to monitor any possible slope movement along the south exterior slopes of the Bottom Ash Stack near the Chemical Ponds and the standpipe piezometers were installed to determine the pore water pressures/groundwater levels at selected locations within the embankments. The final surface elevations and as-drilled locations for each boring were determined by TVA's survey crew upon completion of the drilling. Detailed boring logs and piezometer installation records can be found in Appendix B.

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

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

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

5. Results of Geotechnical Exploration

5.1. Summary of Borings

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

Table 4. Summary of Borings

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

Table 4. Summary of Borings

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

* Refusal, as used herein, refers to rock-like resistance to the advancement of the augers using a carbide-tipped-tooth bit. This may indicate the beginning of weathered bedrock, boulders, or rock remnants. An exact determination cannot be made without performing rock coring.

** Denotes cone penetration test (CPT) boring with assumed surface elevation.

*** Denotes vane shear (V) boring with assumed surface elevation.

NR No Refusal

5.2. Subsurface Soil Conditions

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

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

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

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

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

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

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

5.3. Standard Penetration Tests

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

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

5.4. Undisturbed Sampling

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

Table 5. Summary of Undisturbed Shelby Tube Samples

Boring Location	Boring Number	Sample Depth, ft	Recovery Length, ft
Main Ash Pond A	STN-72	18.0-20.0	1.4
	STN-74	9.0-11.0	2
		24.5-26.5	2
		40.0-42.0	2
	STN-75	25.5-27.5	1.7
		35.0-37.0	1.8
	STN-77	21.0-23.0	2
		38.0-40.0	2
	STN-78	10.5-12.5	1.7
		25.5-27.5	0
		36.5-38.5	1.4
	STN-79	15.0-17.0	1.8
	V-1	10.0-12.0	2
		20.0-22.0	2
		30.0-32.0	2
	V-2	5.0-7.0	2
		15.0-17.0	2
		25.0-27.0	2
Old Scrubber Sludge Pond	STN-69	38.0-40.0	0
	STN-71	30.0-40.0	1.4
		39.5-41.5	1.7
	STN-93	30.0-32.0	2
		39.5-41.5	0
		49.0-51.0	2
	STN-94	15.0-17.0	2
		24.5-26.5	2
		49.0-51.0	2
	STN-103	19.5-21.5	1.5
		29.0-31.0	1.2
		40.0-42.0	2
	STN-104	24.0-26.0	0
		35.0-37.0	0
		44.5-46.5	0.6
	STN-105	30.0-32.0	1.5
		39.5-41.5	2
	STN-106	24.0-26.0	1.7
		35.0-37.0	0
		50.5-52.5	0.6
	STN-107	30.0-32.0	2
		39.5-41.5	1.7
		49.0-51.0	0.9

Boring Location	Boring Number	Sample Depth, ft	Recovery Length, ft
Upper Stilling Pond	STN-89	10.0-12.0	0
		15.0-17.0	2
		20.0-22.0	2
		40.0-42.0	1.8
Lower Stilling Pond	STN-90	5.0-7.0	1
		10.0-12.0	2
		20.0-22.0	0.8
Old Scrubber Sludge Pond Dredge Cell	STN-65	10.0-12.0	0.8
		20.0-22.0	0
		32.0-34.0	1.2
		45.0-47.0	2
	STN-67	9.0-11.0	1.5
		20.0-22.0	1
		40.5-42.5	1.7
	STN-95	25.5-27.5	1.9
		30.5-32.5	1.3
		35.5-37.5	0.6
		40.5-42.5	
		45.5-47.5	0.9
		50.5-52.5	2
	STN-96	25.5-27.5	2
		35.0-37.0	2
		44.5-46.5	0
		52.5-54.5	2
	STN-97	19.5-21.5	1.9
		29.0-31.0	1.9
		40.0-42.0	2
	STN-98	18.0-20.0	2
	STN-99	19.5-21.5	1.5
		29.0-31.0	2
		40.0-42.0	2
	STN-100	30.0-32.0	2
		33.5-35.5	2
		44.5-46.5	1.5
		49.5-51.5	2
	STN-101	19.5-21.5	1.5
		29.0-31.0	1.8
		40.0-42.0	2
	STN-102	19.5-21.5	1.8
		29.0-31.0	1.5
		40.0-42.0	0.5

Table 5. Summary of Undisturbed Shelby Tube Samples

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

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

5.5. Vane Shear Testing

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

Table 6. Summary of Vane Sheer Testing

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

5.6. CPT Testing

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

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

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

5.7. Rock Core Samples

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

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

6. Field Instrumentation

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

6.1. Piezometers

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

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

Table 7. Summary of Piezometers

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

Table 7. Summary of Piezometers

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

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

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

Table 8. Summary of Instantaneous Change in Head Test Results

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

Table 8. Summary of Instantaneous Change in Head Test Results

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

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

6.2. Slope Inclinometers

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

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

Likewise, SI-86 also showed initial movement due to the installation/construction process. However, small cumulative displacements have continued in the upper five feet of the casing between 614.6 feet (ground surface) and 610.0 feet in elevation which may indicate a shear plane. At this time, ¼ inch of movement (after the initial settling) has been recorded at the

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

6.3. Measured Water Levels

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

Table 9. Average Piezometer Water Elevations

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

Table 9. Average Piezometer Water Elevations

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

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

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

7. Laboratory Testing and Analysis

7.1. Introduction

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

7.2. Testing of Standard Penetration Test (SPT) Samples

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

7.2.1. Natural Moisture Content

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

7.2.2. Particle Size Distribution and Fines Content

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

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

Table 10. Average Specific Gravity per Soil Type

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

7.2.3. Classification Testing

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

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

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

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

7.3.1. Dry and Wet Unit Weights

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

Table 11. Average Unit Weights per Location

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

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

Table 12. Average Unit Weights per Soil Type

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

7.3.2. Consolidated-Undrained (CU) Triaxial Testing

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

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

Table 13. Consolidated Undrained Triaxial Test Results per Soil Type

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

NA = Not Applicable

7.3.3. Unconsolidated-Undrained (UU) Triaxial Testing

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

Table 14. Unconsolidated Undrained Triaxial Test Results per Soil Type

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

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

7.3.4. Falling Head Permeability Testing

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

Table 15. Summary of Falling Head Permeability Test Results

Boring Number	Soil Horizon	Test Interval (feet)	Sample Center Elevation (MSL)	Initial Conditions				Coefficient of Permeability K_s (cm/sec)
				Dry Density (pcf)	Moisture Content (%)	Void Ratio (e)	Degree of Saturation (%)	
STN-67	Silty Sand (Fly Ash)	40.5-42.5	605.5	76.6	47.9	1.314	103.6	$1.79x10^{-6}$
STN-72	Silt with Sand (Bottom Ash)	18.0-18.5	618.4	105.0	17.1	0.587	77.7	$1.70x10^{-6}$
STN-78	Fill Clay	10.5-11.0	626.1	102.1	22.1	0.669	90.2	$2.79x10^{-8}$
STN-78	Residual Clay	36.5-37.0	600.1	104.0	22.1	0.614	96.9	$1.94x10^{-8}$
STN-80	Silt with Sand (Bottom Ash)	28.0-28.5	609.0	68.8	43	0.968	96.4	$4.84x10^{-5}$
STN-89	Residual Clay	40.0-42.0	595.3	116.2	16.7	0.462	98.4	$1.78x10^{-7}$
	Silt with Sand (Bottom Ash)		595.3	105.8	18.6	0.564	87.4	$1.83x10^{-5}$
STN-90	Fill Clay	10.0-10.5	603.0	98.0	26.0	0.727	96.9	$1.29x10^{-8}$
STN-96	Silt with Sand (Bottom Ash)	35.6-36.1	611.8	97.1	27.5	0.75	99.8	$1.78x10^{-5}$
STN-100	Residual Clay	49.5-50.0	594.4	86.4	33.9	0.937	97	$1.06x10^{-5}$
STN-101	Fill Clay	40.6-41.1	598.0	95.8	27.5	0.792	95.5	$1.80x10^{-8}$
STN-106	Silty Sand (Fly Ash)	50.5-51.0	595.2	73.3	41.8	0.943	101	$7.05x10^{-6}$

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

8. Engineering Analyses

8.1. General

The following geotechnical engineering analyses includes an evaluation of the soil strengths and permeability parameters for the five (5) predominant soil horizons, as well as, a seepage analyses and slope stability analyses for the as found slope conditions and a proposed final stack height in the active ash stacking area. Prior to beginning the seepage and slope stability analyses, twenty (20) initial cross sections were selected and the geometry of the existing embankment slopes and soil horizons were approximated using current and historical information. Once the geometry of the sections was determined, the geotechnical data for each section was reviewed and evaluated to determine the most critical reaches

around the Ash Pond Complex. The criteria for selecting the critical sections were based on the following: the geometry/steepness of the section cut, height/location of the phreatic surface, subsurface soil conditions/horizon thickness, and historical information/existing geotechnical data. Based on this evaluation, ten (10) critical cross sections (Section A, C, D, H, J, L, M, O, S and T) were selected for seepage and slope stability analyses. Results of the analyses and evaluations are summarized in the following paragraphs. The results of the seepage and slope stability analyses are included in Appendix G and the plan locations for each cross section is identified on the geotechnical drawings included in Appendix C.

8.2. Soil Horizons

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

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

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

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

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

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

8.3. Seepage Analyses

8.3.1. SEEP/W Model

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

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

8.3.2. SEEP/W Boundary Conditions

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

Table 16. SEEP/W Boundary Conditions

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

Table 16. SEEP/W Boundary Conditions

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

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

8.3.3. Seepage Properties

For each cross section analyzed, a representative subsurface profile was compiled based on boring logs, available record drawings, and the known project history. Material properties were estimated based on available laboratory data and typical values for similar soils. Material properties used in the seepage analysis are summarized in Table 17.

Table 17. Seepage Parameters

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

Significant engineering judgment is needed to select appropriate hydraulic properties for earth material. Unlike other key properties, hydraulic conductivity can vary over several orders of magnitude for a range of soils, often with substantial anisotropy for seepage in horizontal versus vertical directions. Laboratory test samples often do not represent important variations within a larger soil deposit. For the Ash Pond Complex, an iterative parametric calibration was used to arrive at final seepage design parameters. The results from trial SEEP/W simulations were compared to field data (measured piezometric levels). The material parameters were then varied until the solutions reasonably matched the field data for the representative cross sections. The final set of parameters identified in Table 18 was a result of the piezometer calibration process presented in Section 8.3.4.

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

8.3.4. Comparison to Field Observations

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

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

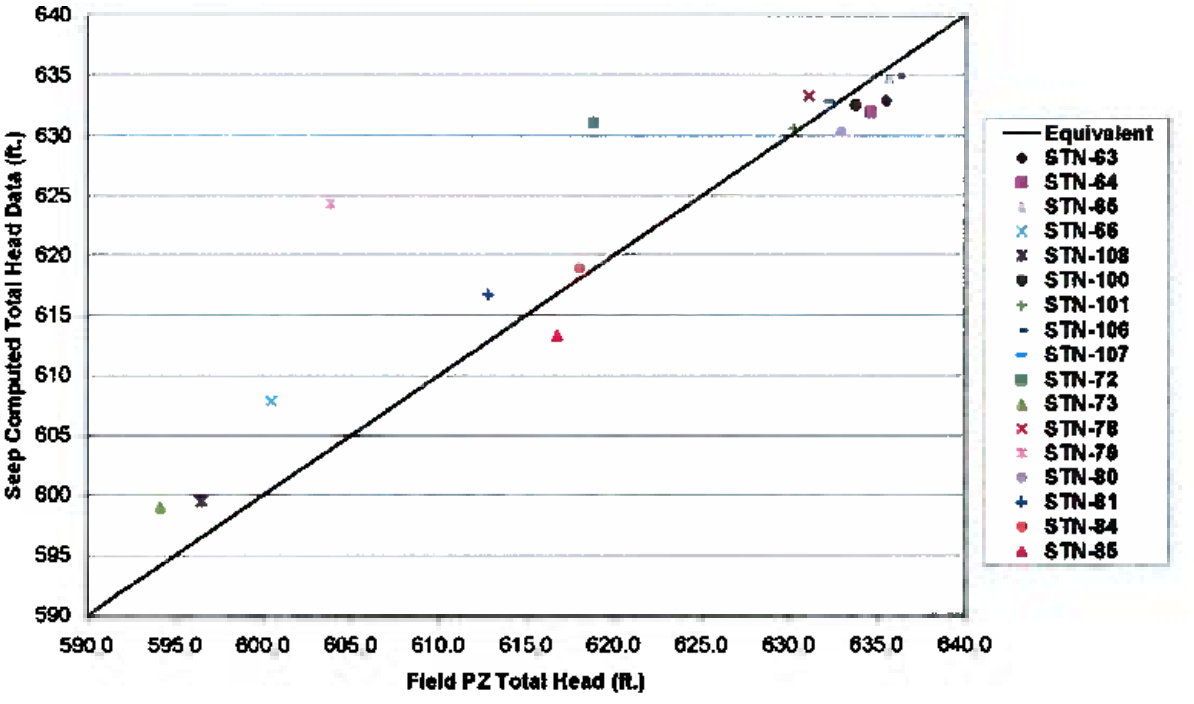


Figure 22. Comparison between the Field Piezometer Readings and Pore Water Pressures predicted in the SEEP/W Model

8.4. Strength Parameter Selection

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

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

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

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

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

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

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

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

The two Bottom Ash soils were primarily encountered above the residual clay soils and fill clay soil horizons. The type of bottom ash encountered, whether Silty Sand with Gravel or Silt with Sand was dependent on the location of sample. The coarser material was closer to the sluice line outfalls and internal dike expansions and the Silt with Sand (Bottom Ash) was encountered at lower depths within the ponds. Stantec performed CU triaxial tests on selected samples and the results were plotted on a scatter plot as described above. Again

the unit wet weight was determined by taking the average unit weight of the samples included within the soil horizon. These values were compared to the values obtained from the SPT correlation tables. However, since no CU triaxial tests were conducted on the Silty Sand with Gravel (Bottom Ash), the phi angle determined for this soil horizon was determined from the SPT correlation tables.

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

Table 19. Selected Strength Parameters for Stability Analysis

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

8.5. Slope Stability Analyses

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

9. Results

9.1. Seepage Exit Gradients for As Found Conditions

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

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

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

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

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

9.2. Slope Stability Results for As Found Conditions

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

There was no indication in the slope stability analyses that a noncircular failure surface would give a factor of safety lower than obtained for the optimized circular surfaces. Overall, the

geometry of the dike cross section and the foundation stratigraphy do not appear to be susceptible to sliding along a planar surface. The optimization scheme available within SLOPE/W was used to consider noncircular segments along the curved slip surfaces. The results in Table 21 and Appendix G represent factors of safety computed from the optimized, circular slip surface routine.

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

Table 21. Stability Analysis Results for As Found Conditions

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

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

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

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

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

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

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

9.3. Slope Stability Results for Planned Final Stack Conditions

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

Table 22. Stability Analysis Results for Planned Final Stack Conditions

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

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

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

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

10. Limitations of Study

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

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

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

11. Conclusions

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

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

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

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

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

12. Recommendations

12.1. Seepage and Stability Improvements

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

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

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

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

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

12.2. Dredge Cell

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

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

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

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

12.3. Bottom Ash Stack

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

12.4. Active Ash Stacking Areas

The active ash stacking areas within the Main Ash Pond A and Bottom Ash Stack should be operated in accordance with an updated Operations and Maintenance (O&M) manual and updated Stacking Plans. Stacking Plans should be designed using current site condition

using new topographic and hydrographic mapping and based on a planned fill date of January, 2015 at which time the waste disposal operations will cease at the existing facilities. The plans should also be developed in coordination with ongoing stability mitigation plans, inactive Dredge Cell closure, NPDES environmental compliance and spillway replacement projects.

12.5. Spillways

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

12.6. Additional Geotechnical Drilling and Instrumentation

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

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

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

13. Closure

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

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

Conditions at other locations may differ from those occurring at the boring locations. Also, the passage of time may result in a change in the subsurface conditions at the boring locations.

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

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

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

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

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US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 2

Phase 2 Geotechnical Exploration, Gypsum Stack, Widows Creek Fossil Plant, Stevenson Alabama, Stantec Consulting Services, February 5, 2010

Report of Phase 2 Geotechnical Exploration

Gypsum Stack
Widows Creek Fossil Plant
Stevenson, Alabama

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Tennessee Valley Authority
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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, PE
Project Manager

/rdr



Report of Phase 2 Geotechnical Exploration

Gypsum Stack
Widows Creek Fossil Plant
Stevenson, Alabama

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

February 5, 2010

Report of Phase 2 Geotechnical Exploration
Gypsum Stack
Widows Creek Fossil Plant
Stevenson, Alabama

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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

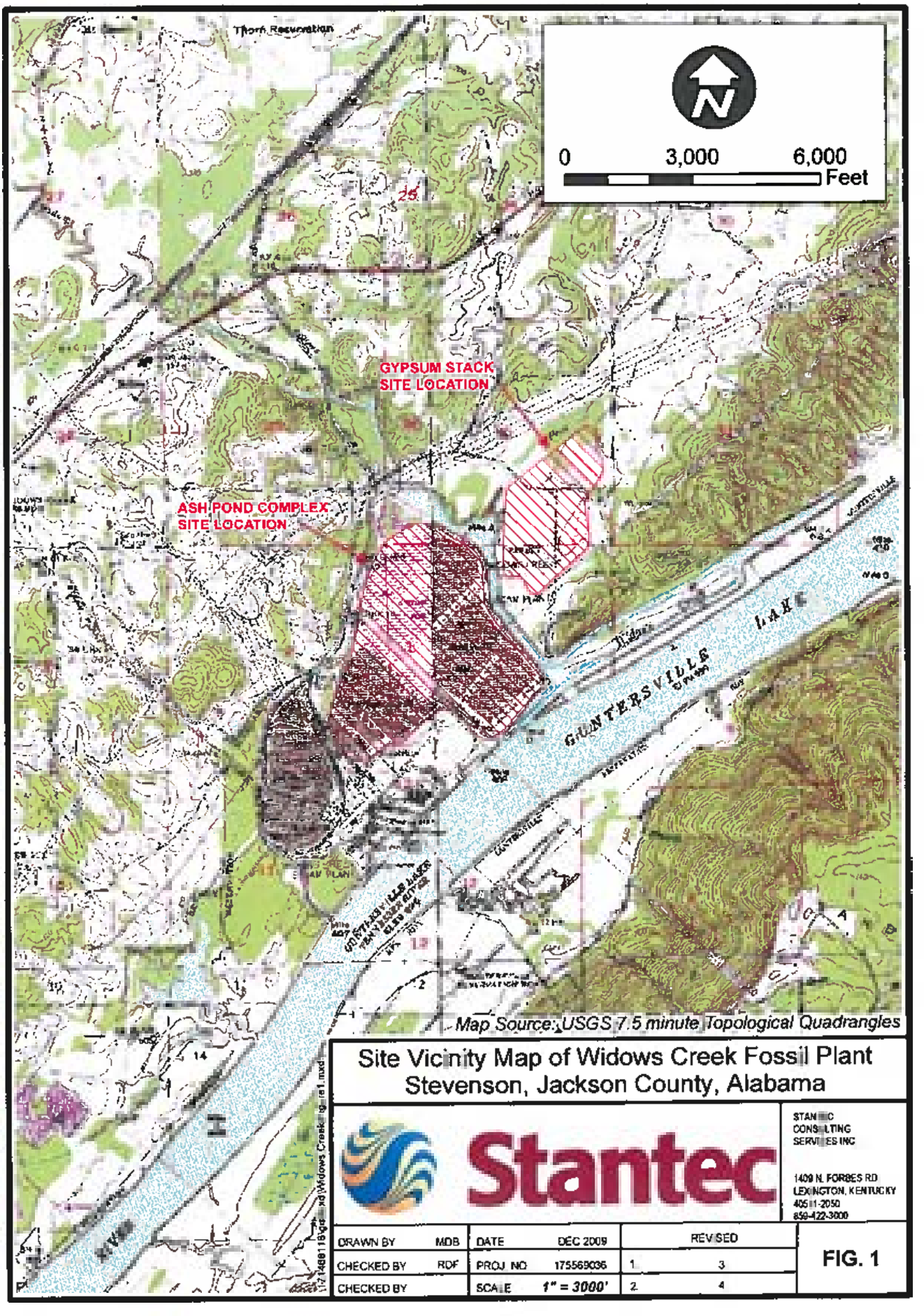
The Widows Creek Fossil (WCF) Plant is located in northeastern Alabama along the west shore of the Tennessee River and at the confluence of Widows Creek. More specifically, the plant is located at 2800 Steam Plant Road in Stevenson, Jackson County, Alabama approximately 40 miles southwest of Chattanooga, 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.



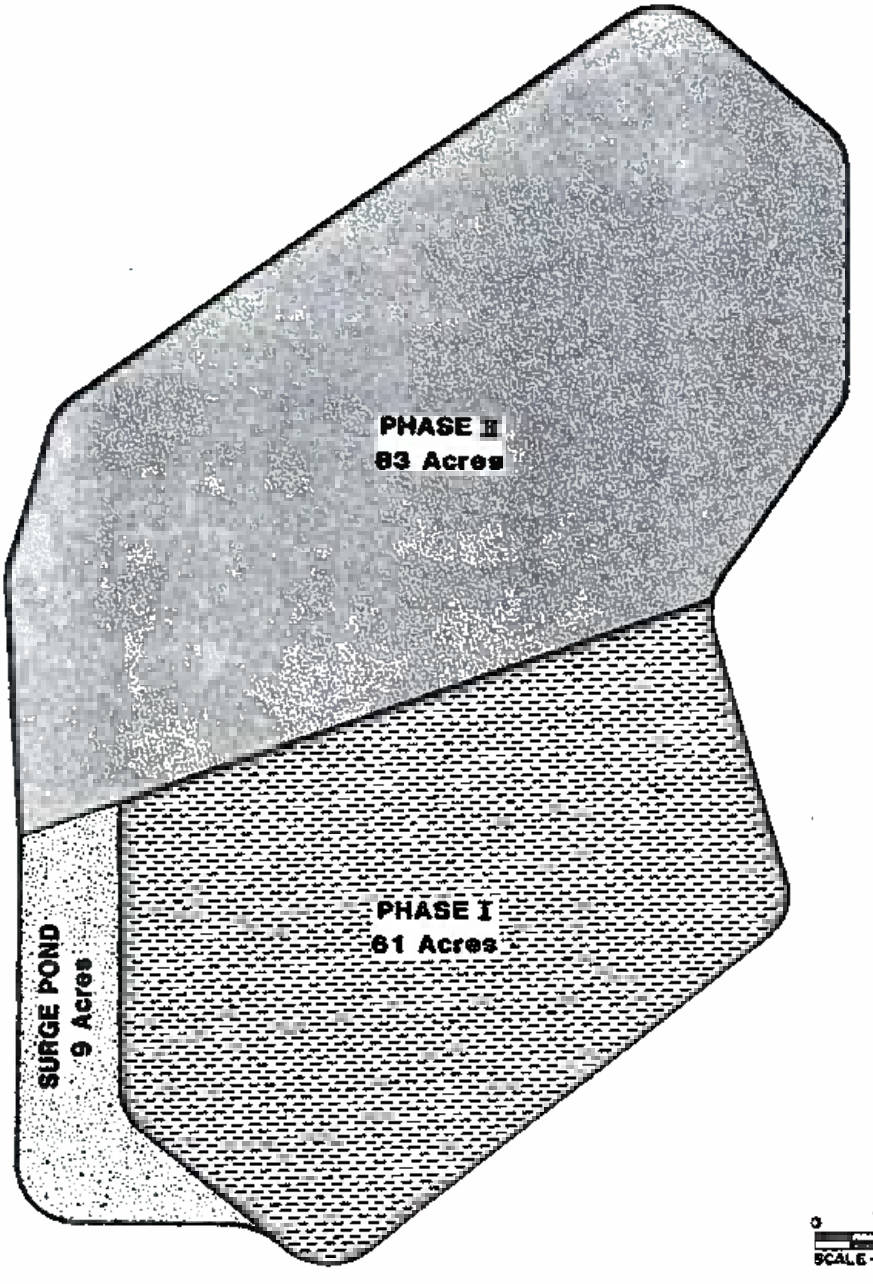
Site Vicinity Map of Widows Creek Fossil Plant
Stevenson, Jackson County, Alabama

Stantec

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859-422-3000

DRAWN BY	MDB	DATE	DEC 2009	REVISED	
CHECKED BY	RDF	PROJ NO	175569036	1	3
CHECKED BY		SCALE	1" = 3000'	2	4

FIG. 1



Widows Creek Fossil Plant
Gypsum Stack
Stevenson, Alabama

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

Stantec

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

Stantec

Figure 3. Widows Creek Fossil Plant Gypsum Stack Layout

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.

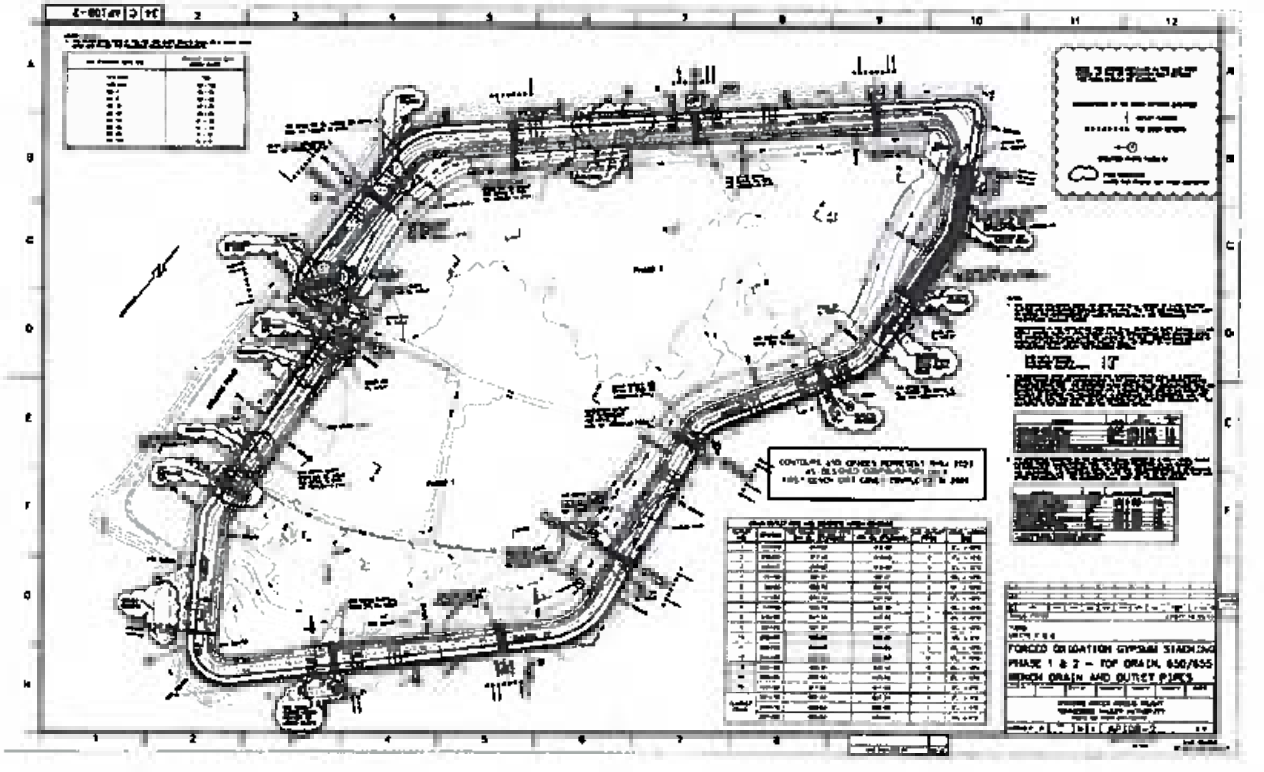


Figure 7. Toe Drain, 650/655 Bench Drain and Outlet Pipe

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.

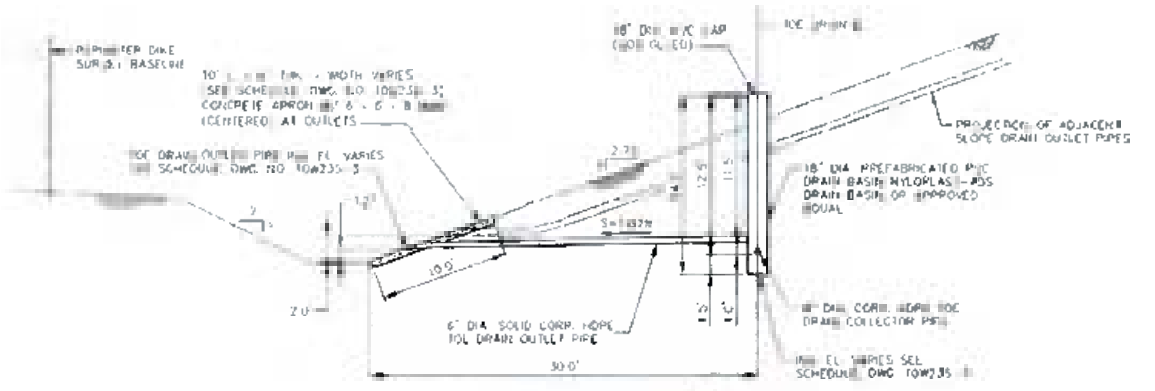


Figure 8. Toe Drain and Outlet Typical Section

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.

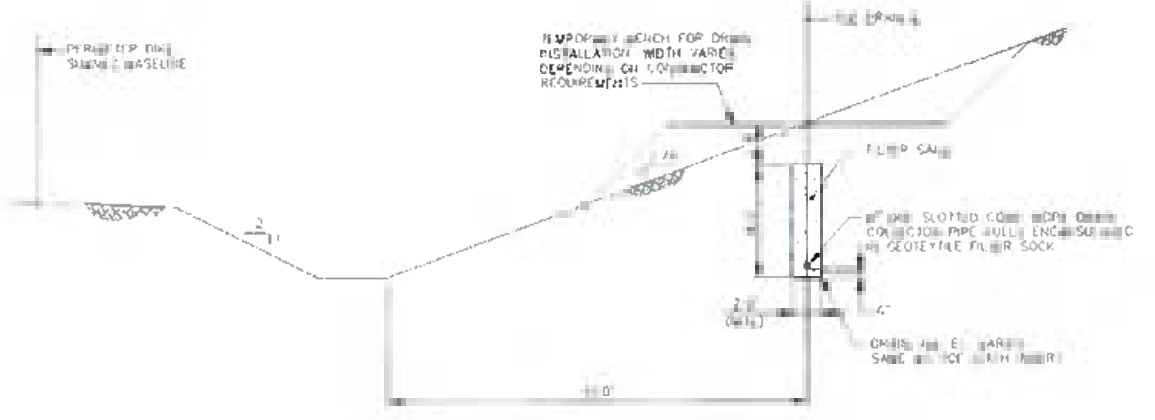


Figure 9. Toe Drain Typical Section

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.

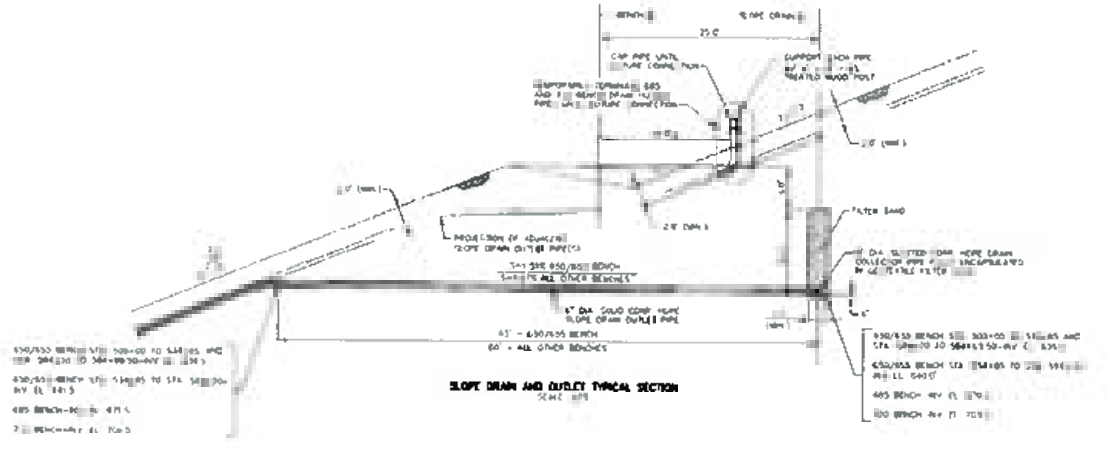


Figure 10. Slope Drain and Outlet Typical Section

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.

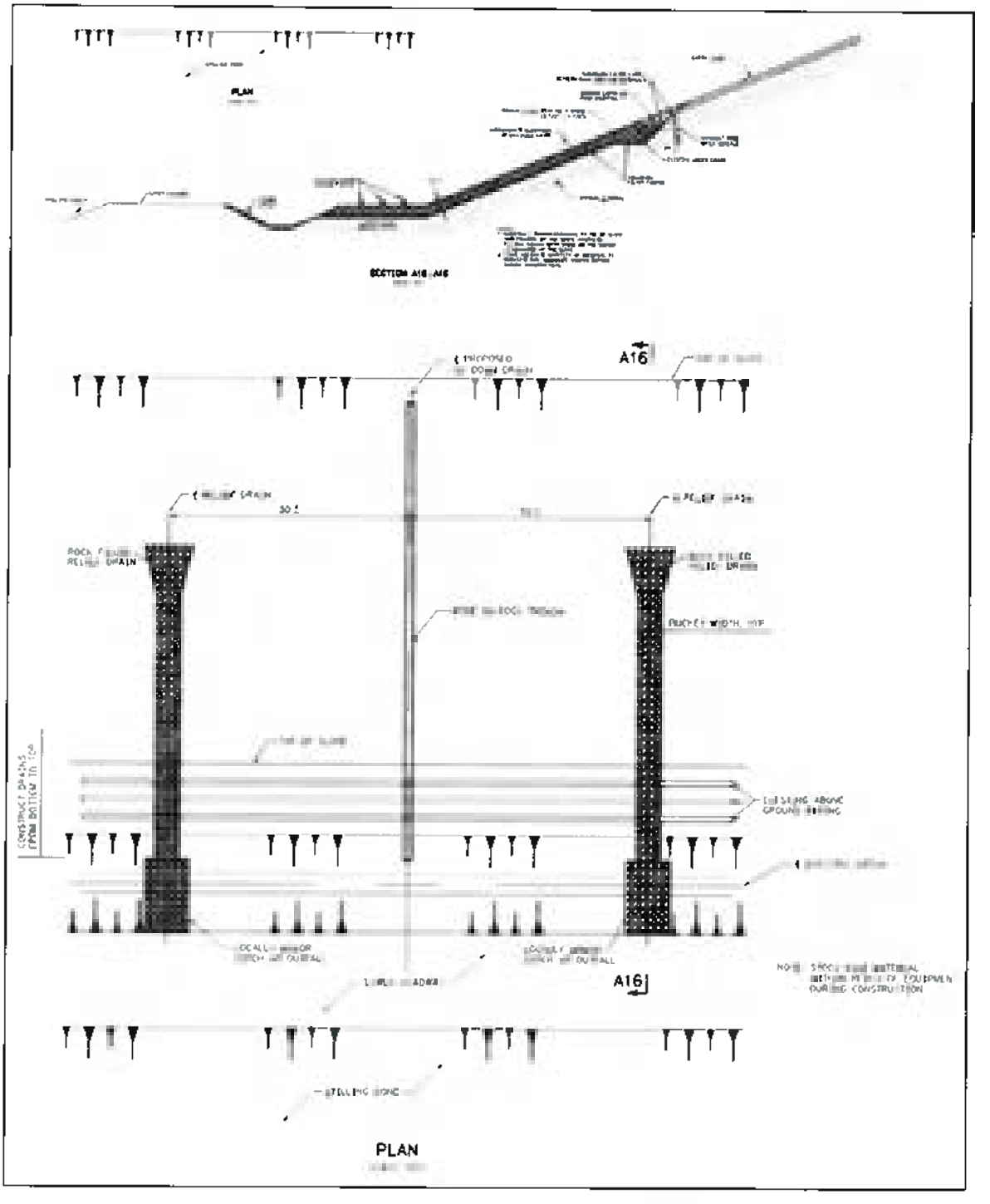
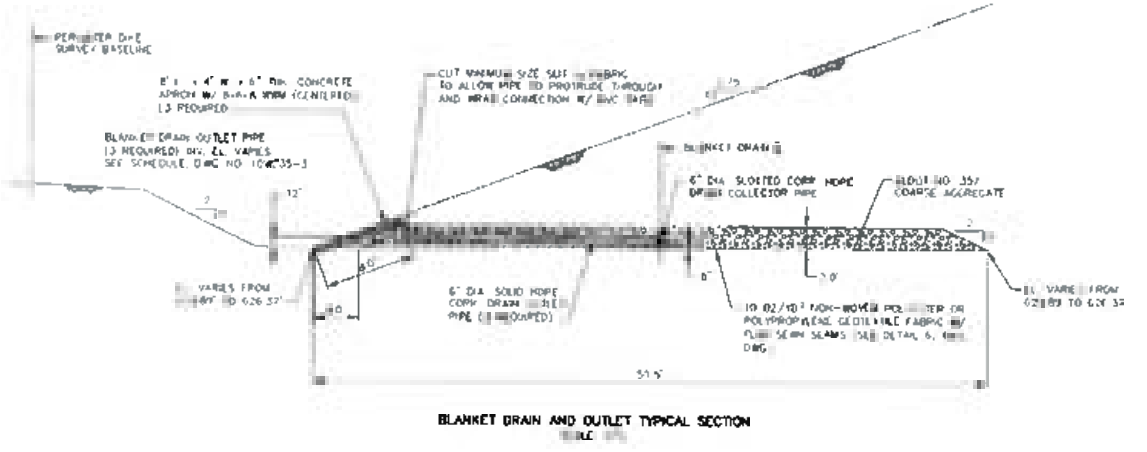


Figure 11. Modified Surface Water Collection

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.



BLANKET DRAIN AND OUTLET TYPICAL SECTION

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.

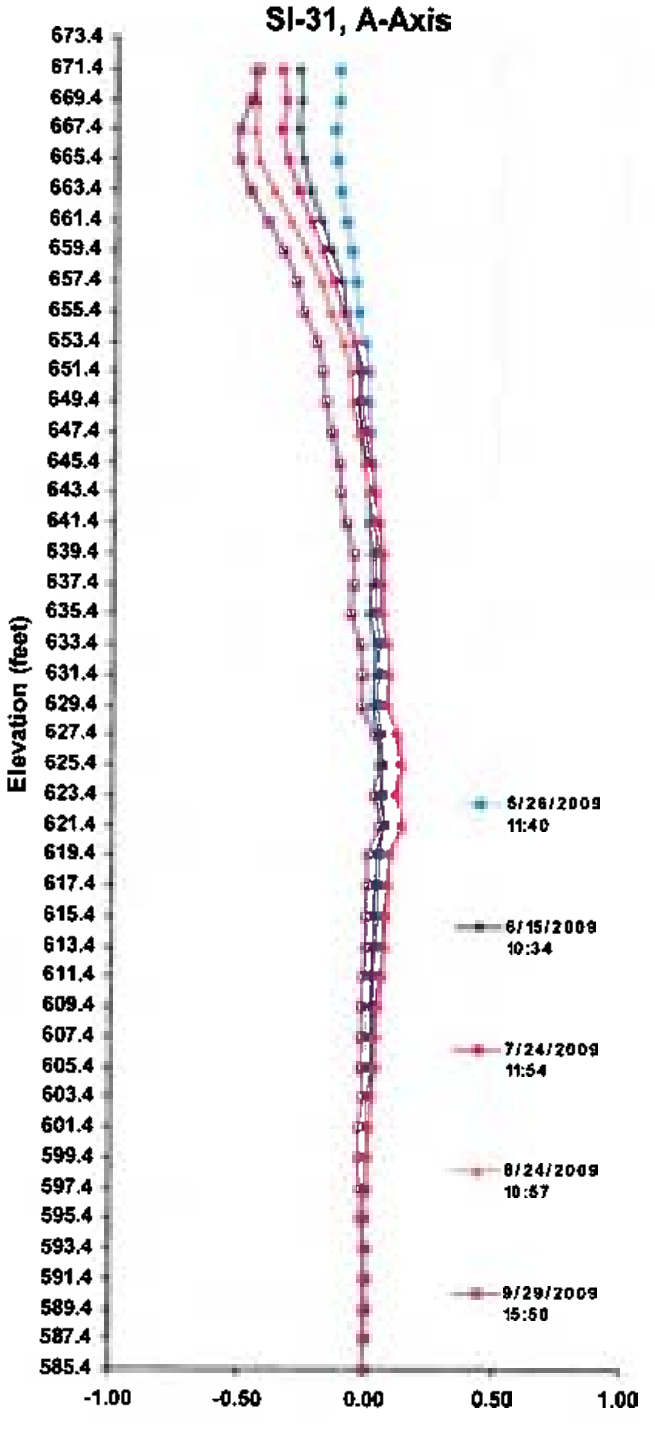
The results of the field and laboratory testing services were then used to develop critical stability sections around the Gypsum Stack at locations were possible signs of instability were observed. Stantec initially reviewed 15 cross sections (A through O) located at various distances along the perimeter 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.

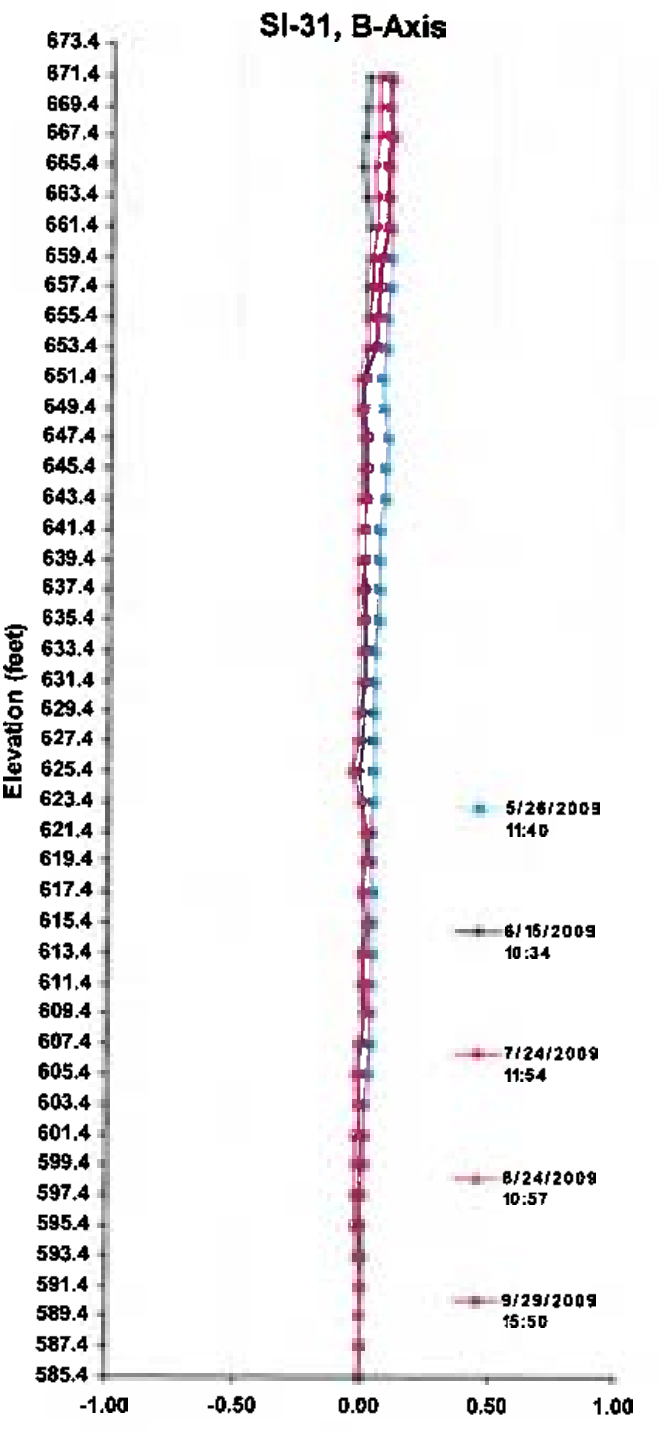
An inventory of the pipe penetrations through the perimeter of the stack was conducted. The assessment, remedial design, and construction work is being documented under a separate cover referred to as Work Plan 6.

5. Results of Geotechnical Exploration

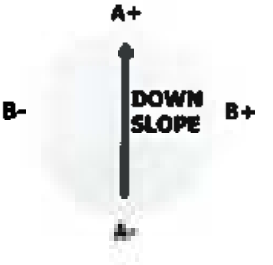
5.1. Summary of Borings

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.





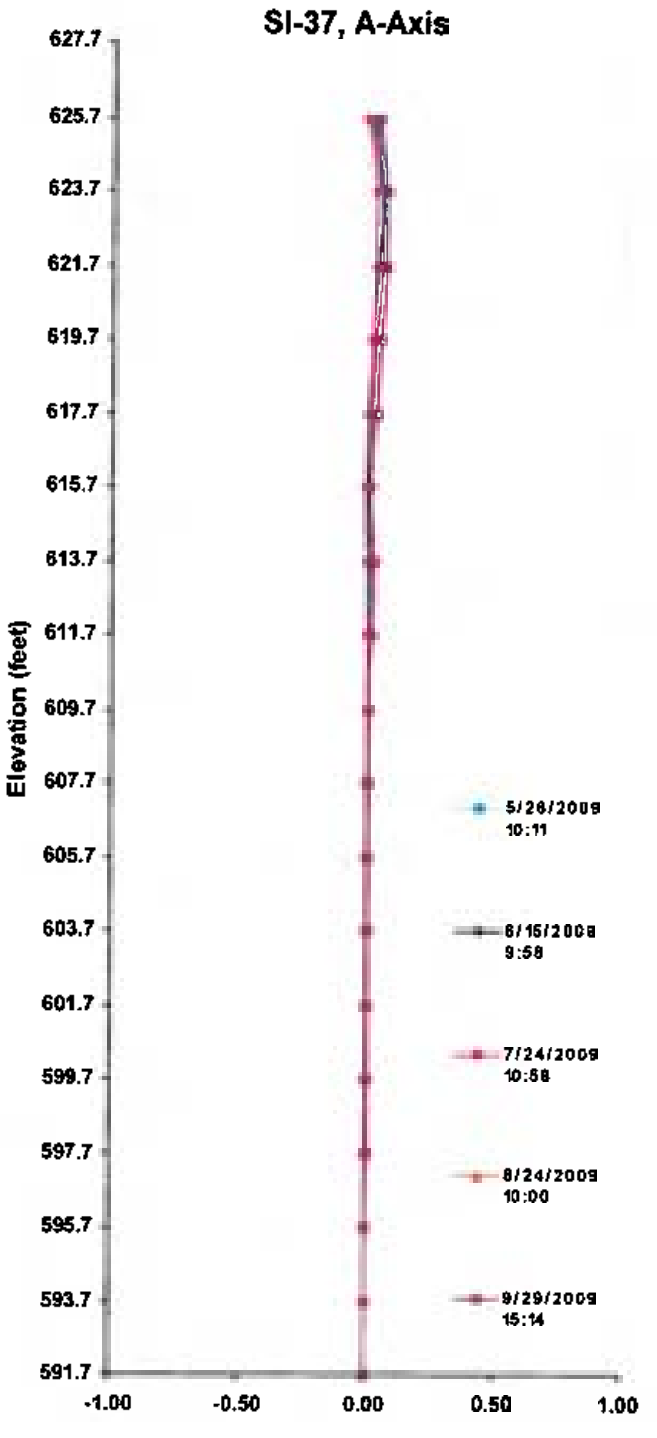
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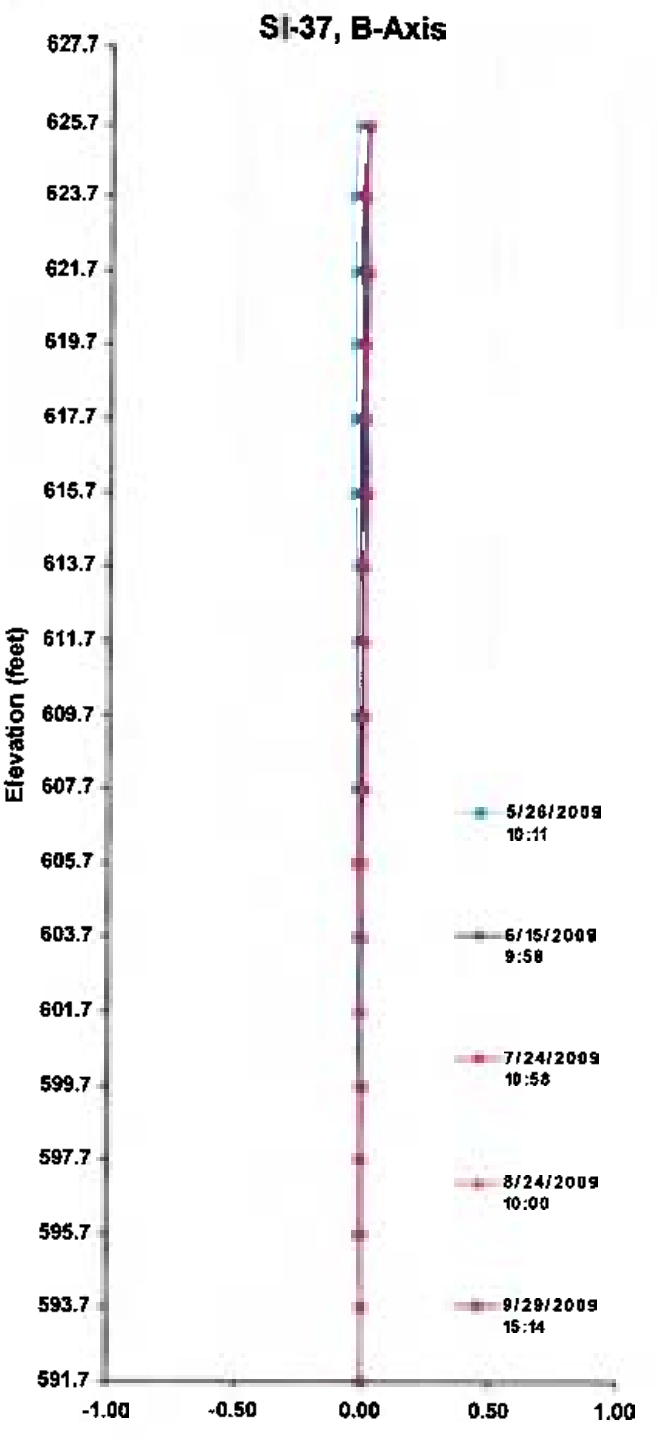


Widows Creek Fossil Plant
Gypsum Pond
Stevenson, AL

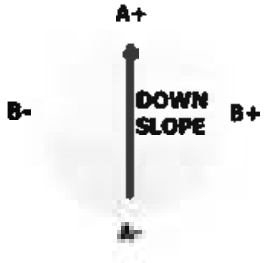
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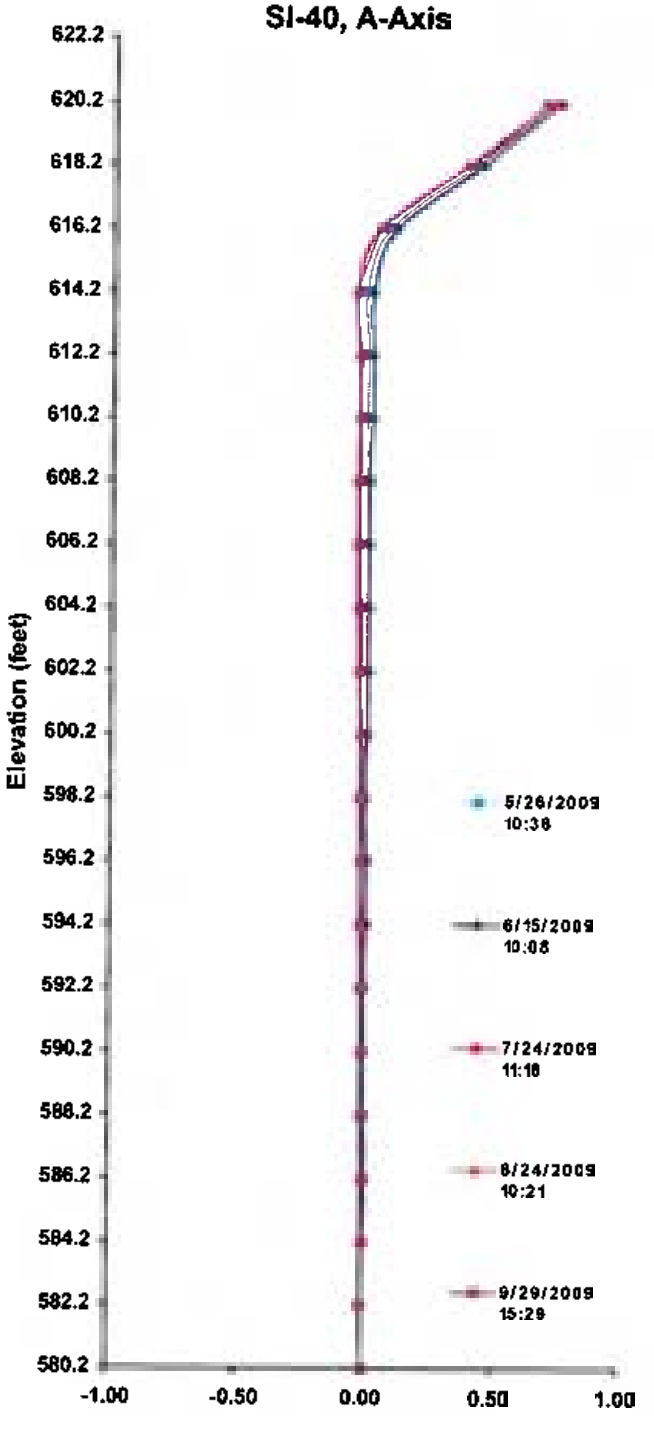
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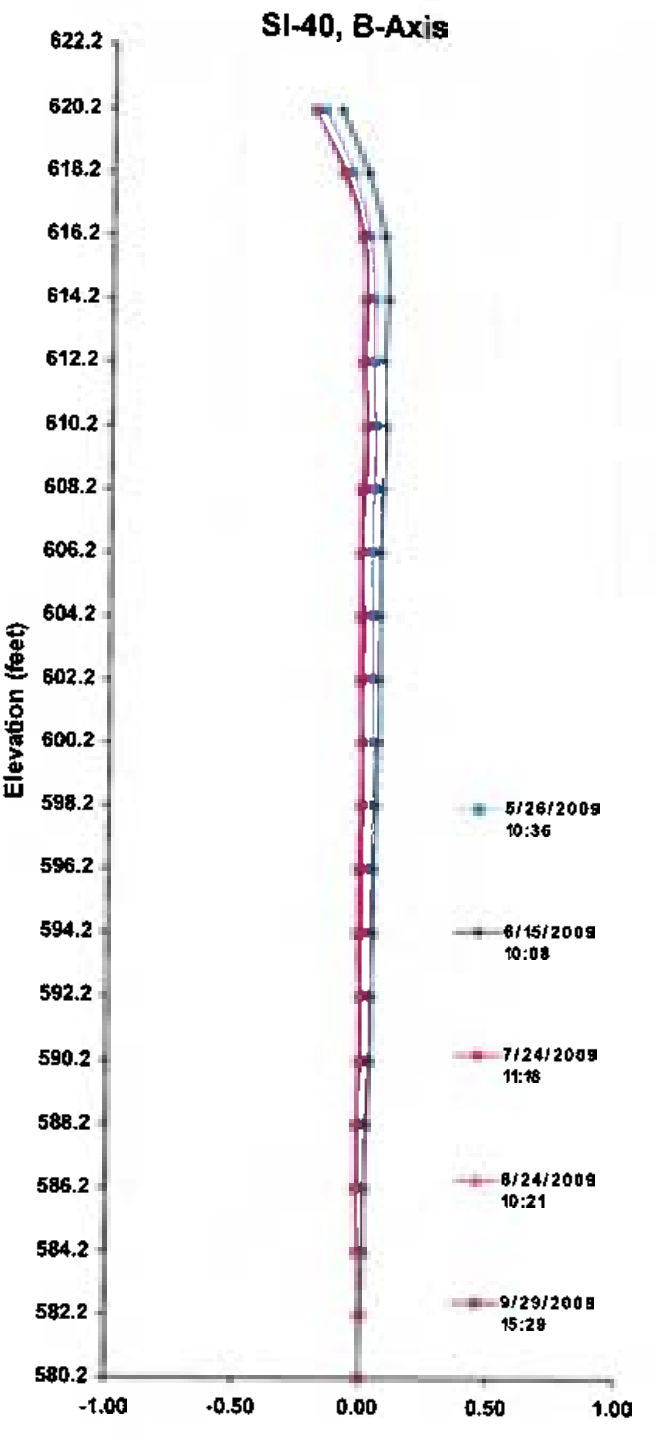


Widows Creek Fossil Plant
Gypsum Pond
Stevenson, AL

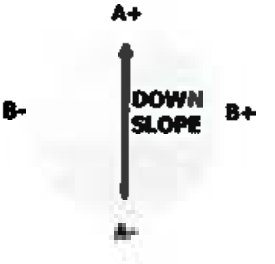
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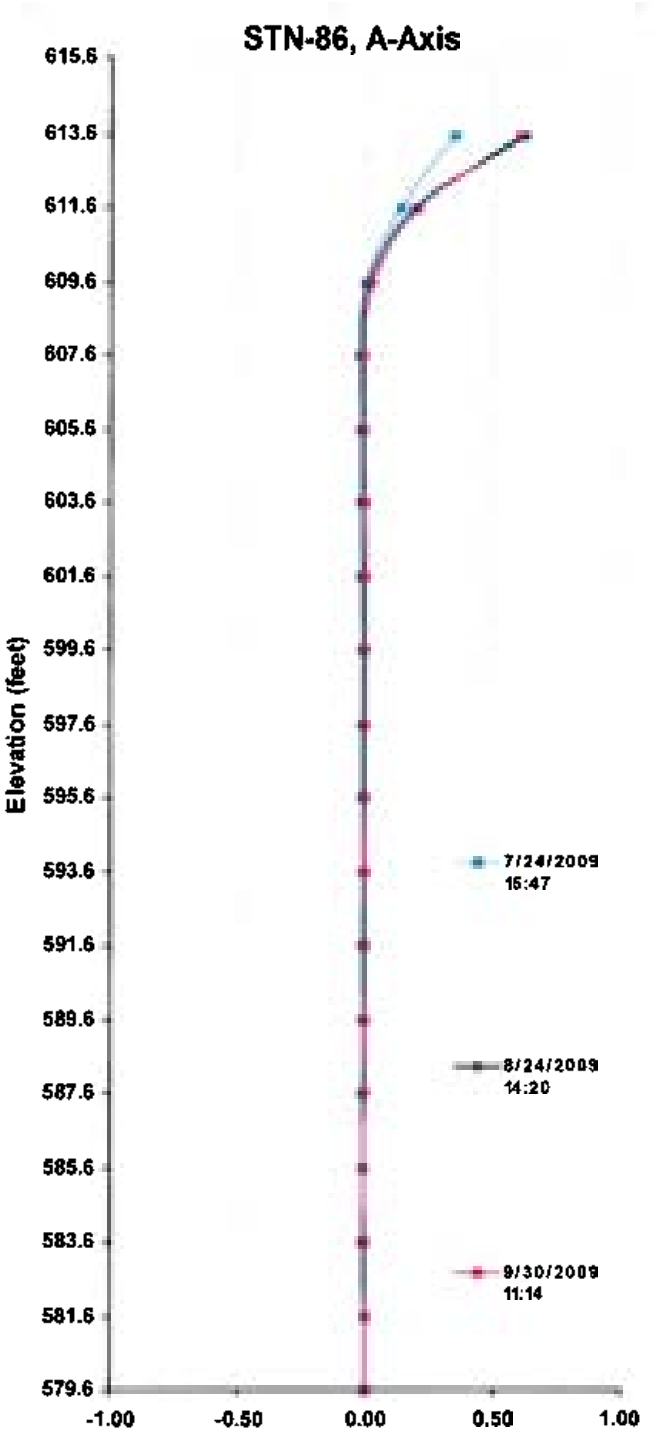


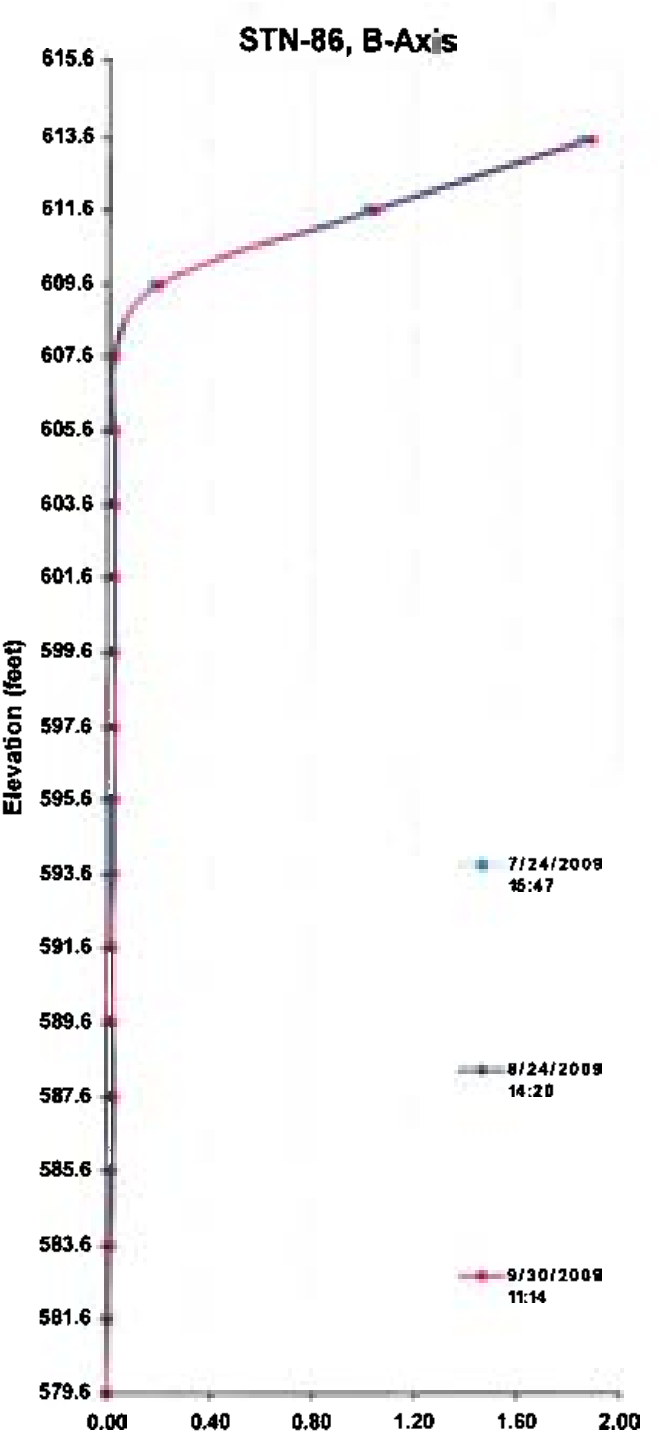


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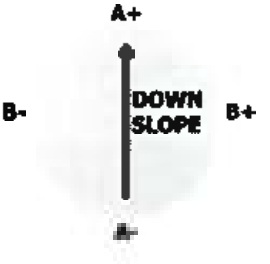


Widows Creek Fossil Plant
Gypsum Pond
Stevenson, AL
175569039
10/13/2009





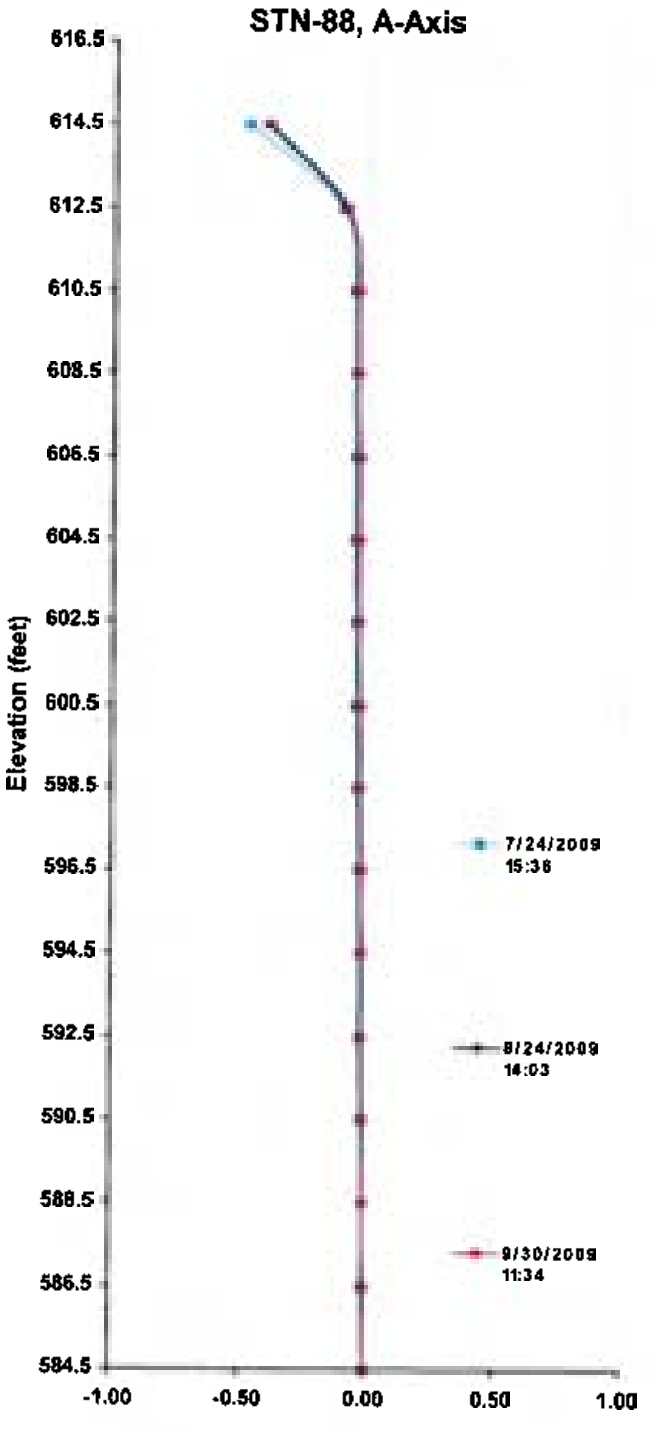
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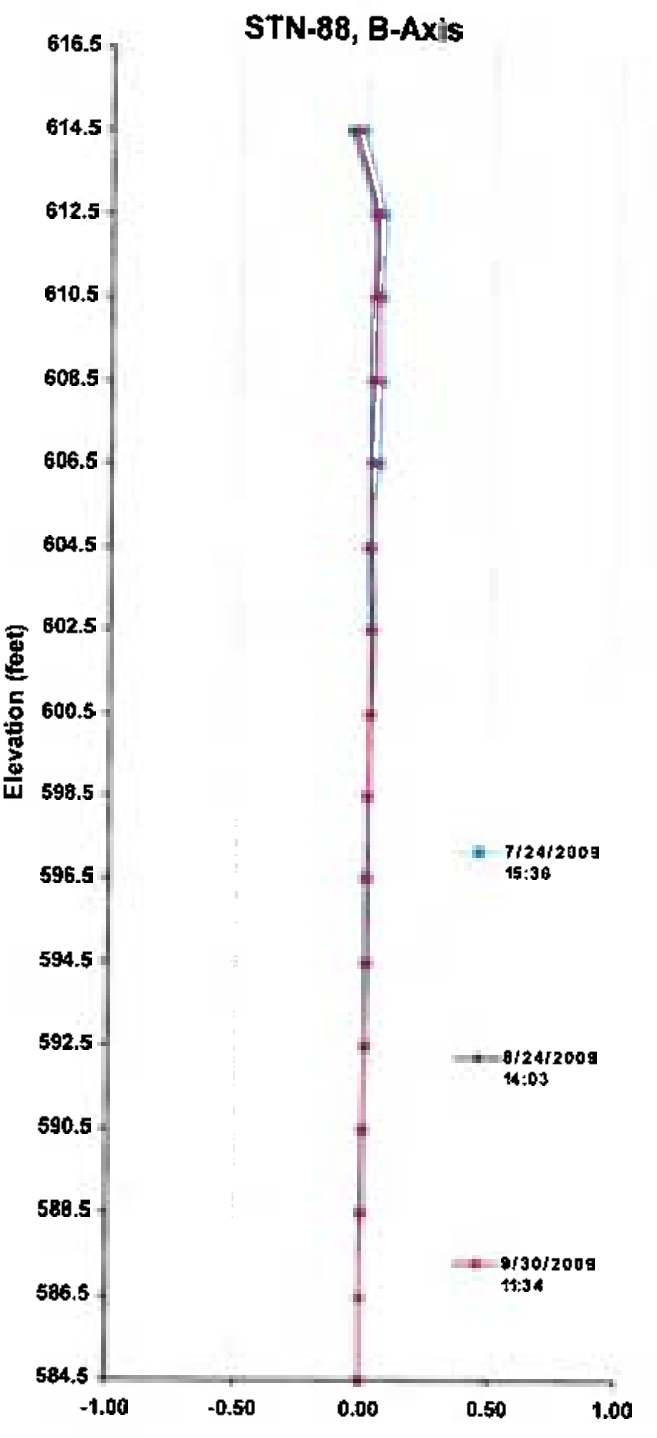


Widows Creek Fossil Plant
Main Ash Pond
Stevenson, AL

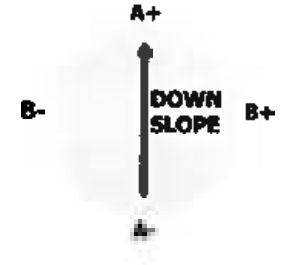
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Stantec



Widows Creek Fossil Plant
Main Ash Pond
Stevenson, AL
175569036
10/13/2009
Appendix E-5 of 10

Power System Operations
Surveying & Project Services

Project: **Widows Creek Fossil Plant**
Gypsum Stack Phase I Pond 2B
Slope Monitoring (Slope Between Phase 1 Pond 2B & Gypsum Stack Stilling Pond)
EPOCH-7
June 2009

Survey Date: 06/30/2009
Project: AJM952
Job: AJM952.1

Projection: Alabama Mercator East
Horizontal Datum: NAD 27 Order: GPS RTK
Coordinate Type: Ground
Vertical Datum: NGVD 29 Order: GPS RTK
Units: US Survey Feet

Printing Date: 07/15/2009

US Survey Feet

Monitoring Station	Change Since Previous Survey 6/10/2009 dN / Direction	dE / Direction	dZ	Change Since Initial Survey 1/27/2009 dN / Direction	dE / Direction	dZ	Remarks
WLS-1	0.0	0.0	0.0	0.0	0.0	0.0	
WLS-2	0.0	0.0	0.1	0.0	0.0	0.0	
WLS-3	0.0	0.0	0.0	0.0	0.0	0.0	
WLS-4	0.0	0.0	0.0	0.0	0.0	0.0	

Notes:
All monitoring stations were initially set as angle irons on 01/27/2009.
Horizontal movements are parallel and/or perpendicular to NAD 27 Alabama Mercator East state plane north.
Expected survey accuracy is ±0.05 Ft.

Power System Operations
Surveying & Project Services

Project: **Widows Creek Fossil Plant**
Gypsum Stack Phase I Pond 2B
Slope Monitoring (Slope Between Phase 1 Pond 2B & Gypsum Stack Stilling Pond)
EPOCH-6
June 2009

Survey Date: 06/10/2009
Project: AJM948
Job: AJM948.1

Projection: Alabama Mercator East
Horizontal Datum: NAD 27 Order: GPS RTK
Coordinate Type: Ground
Vertical Datum: NGVD 29 Order: GPS RTK
Units: US Survey Feet

Printing Date: 07/14/2009

US Survey Feet

Monitoring Station	Change Since Previous Survey 6/2/2009 dN / Direction	dE / Direction	dZ	Change Since Initial Survey 1/27/2009 dN / Direction	dE / Direction	dZ	Remarks
WLS-1	0.0	0.0	0.0	0.0	0.0	0.0	
WLS-2	0.0	0.0	-0.1	0.0	0.0	-0.1	
WLS-3	0.0	0.0	0.0	0.0	0.0	0.0	
WLS-4	0.0	0.0	0.0	0.0	0.0	0.0	

Notes:
All monitoring stations were initially set as angle irons on 01/27/2009.
Horizontal movements are parallel and/or perpendicular to NAD 27 Alabama Mercator East state plane north.
Expected survey accuracy is ±0.05 Ft.

Power System Operations
Surveying & Project Services

Project: **Widows Creek Fossil Plant**
Gypsum Stack Phase I Pond 2B
Slope Monitoring (Slope Between Phase 1 Pond 2B & Gypsum Stack Stilling Pond)
EPOCH-4
April 2009

Survey Date: 04/29/2009
Project: EAD539
Job: EAD539.1

Printing Date: 05/13/2009

Projection: Alabama Mercator East
Horizontal Datum: NAD 27 Order: GPS RTK
Coordinate Type: Ground
Vertical Datum: NGVD 29 Order: GPS RTK
Units: US Survey Feet

US Survey Feet

Monitoring Station	Change Since Previous Survey 3/23/2009 dN / Direction	dE / Direction	dZ	Change Since Initial Survey 1/27/2009 dN / Direction	dE / Direction	dZ	Remarks
WLS-1	0.0	0.0	0.0	0.0	0.0	0.0	
WLS-2	0.0	0.0	0.0	0.0	0.0	0.0	
WLS-3	0.0	0.1 West	0.0	0.0	0.0	0.0	
WLS-4	0.0	0.1 West	0.0	0.0	0.1 West	0.0	

Notes:
All monitoring stations were initially set as angle irons on 01/27/2009.
Horizontal movements are parallel and/or perpendicular to NAD 27 Alabama Mercator East state plane north.
Expected survey accuracy is ±0.05 Ft.

Power System Operations
Surveying & Project Services

Project: **Widows Creek Fossil Plant**
Gypsum Stack Phase I Pond 2B
Slope Monitoring (Slope Between Phase 1 Pond 2B & Gypsum Stack Stilling Pond)
EPOCH-3
March 2009

Survey Date: 03/23/2009
Project: PCF513
Job: PCF513.1

Projection: Alabama Mercator East
Horizontal Datum: NAD 27 Order: GPS RTK
Coordinate Type: Ground
Vertical Datum: NGVD 29 Order: GPS RTK
Units: US Survey Feet

Printing Date: 03/25/2009

US Survey Feet

Monitoring Station	Change Since Previous Survey 2/23/2009 dN / Direction	dE / Direction	dZ	Change Since Initial Survey 1/27/2009 dN / Direction	dE / Direction	dZ	Remarks
WLS-1	0.0	0.0	0.0	0.0	0.0	0.0	
WLS-2	0.0	0.0	0.0	0.0	0.0	0.0	
WLS-3	0.0	0.1 East	0.0	0.0	0.0	0.0	
WLS-4	0.0	0.1 East	0.0	0.0	0.0	0.0	

Notes:
All monitoring stations were initially set as angle irons on 01/27/2009.
Horizontal movements are parallel and/or perpendicular to NAD 27 Alabama Mercator East state plane north.
Expected survey accuracy is ±0.05 Ft.

Power System Operations
Surveying & Project Services

Project: **Widows Creek Fossil Plant**
Gypsum Stack Phase I Pond 2B
Slope Monitoring (Slope Between Phase 1 Pond 2B & Gypsum Stack Stilling Pond)
EPOCH-5
June 2009

Survey Date: 06/02/2009
Project: PCF529
Job. PCF529.1

Printing Date: 06/05/2009

Projection: Alabama Mercator East
Horizontal Datum: NAD 27 Order: GPS RTK
Coordinate Type: Ground
Vertical Datum: NGVD 29 Order: GPS RTK
Units: US Survey Feet

US Survey Feet

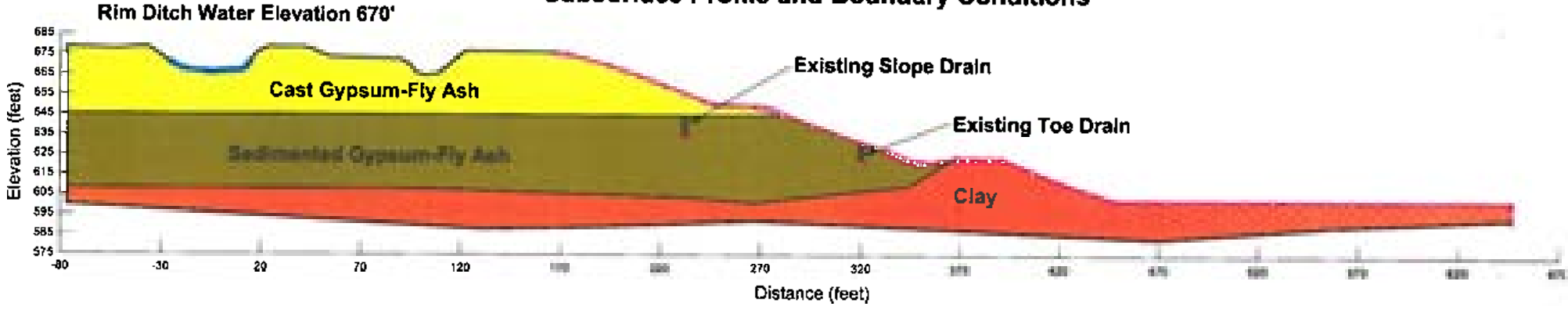
Monitoring Station	Change Since Previous Survey 4/29/2009 dN / Direction	dE / Direction	dZ	Change Since Initial Survey 1/27/2009 dN / Direction	dE / Direction	dZ	Remarks
WLS-1	0.0	0.1 West	0.0	0.0	0.0	0.0	
WLS-2	0.0	0.0	0.0	0.0	0.0	0.0	
WLS-3	0.0	0.0	0.0	0.0	0.0	0.0	
WLS-4	0.0	0.0	0.0	0.0	0.0	0.0	

Notes:
All monitoring stations were initially set as angle irons on 01/27/2009.
Horizontal movements are parallel and/or perpendicular to NAD 27 Alabama Mercator East state plane north.
Expected survey accuracy is ±0.05 Ft.

Widows Creeek Fossil Plant

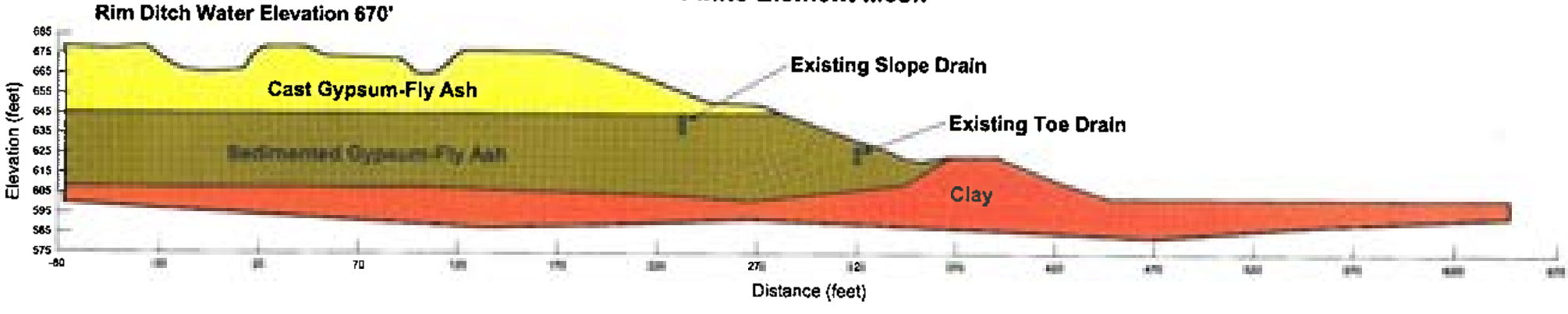
Gypsum Stack - Section A with Drains

Subsurface Profile and Boundary Conditions



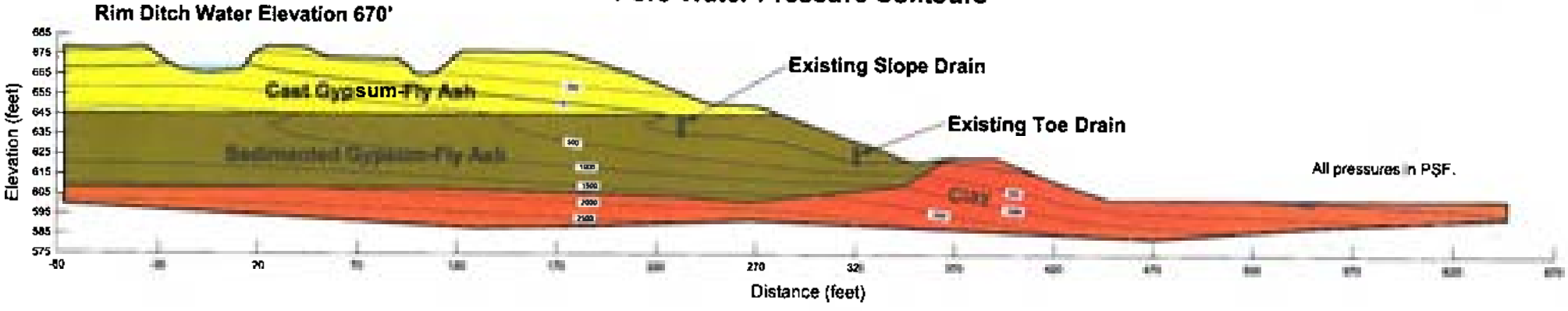
Widows Creeek Fossil Plant

Gypsum Stack - Section A with Drains
Finite Element Mesh



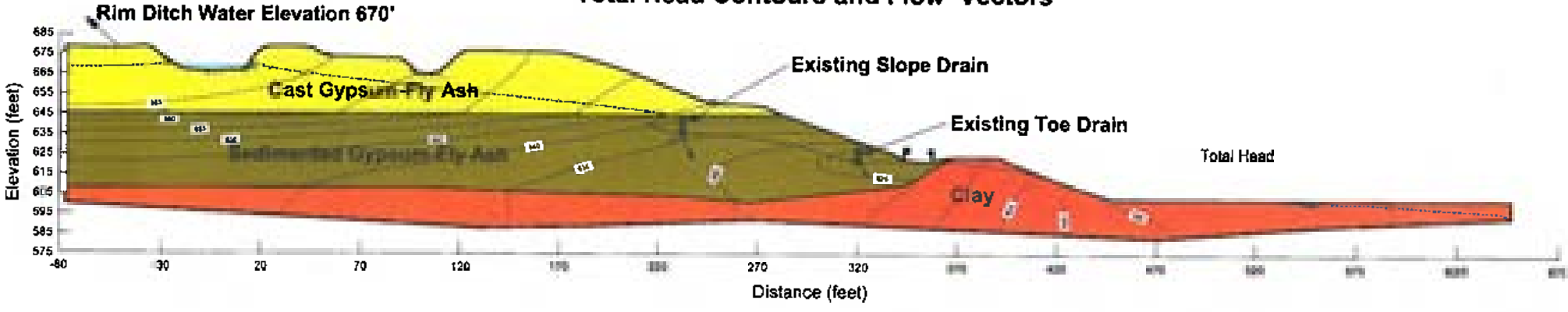
Widows Creeek Fossil Plant

Gypsum Stack - Section A with Drains
Pore-Water Pressure Contours



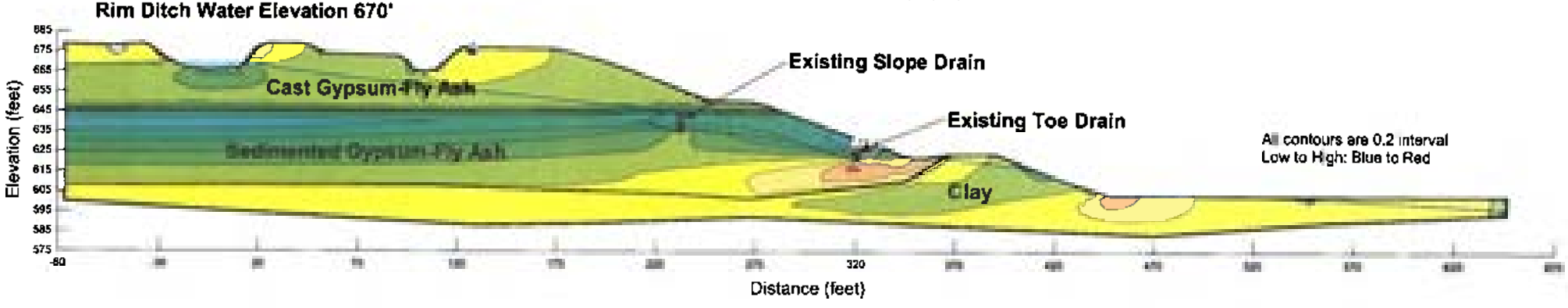
Widows Creeek Fossil Plant

Gypsum Stack - Section A with Drains
Total Head Contours and Flow Vectors

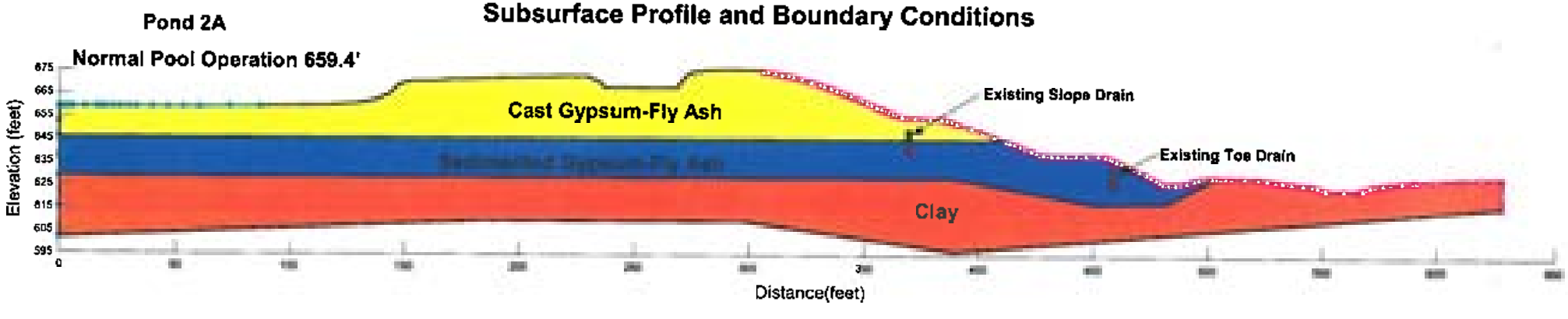


Widows Creeek Fossil Plant

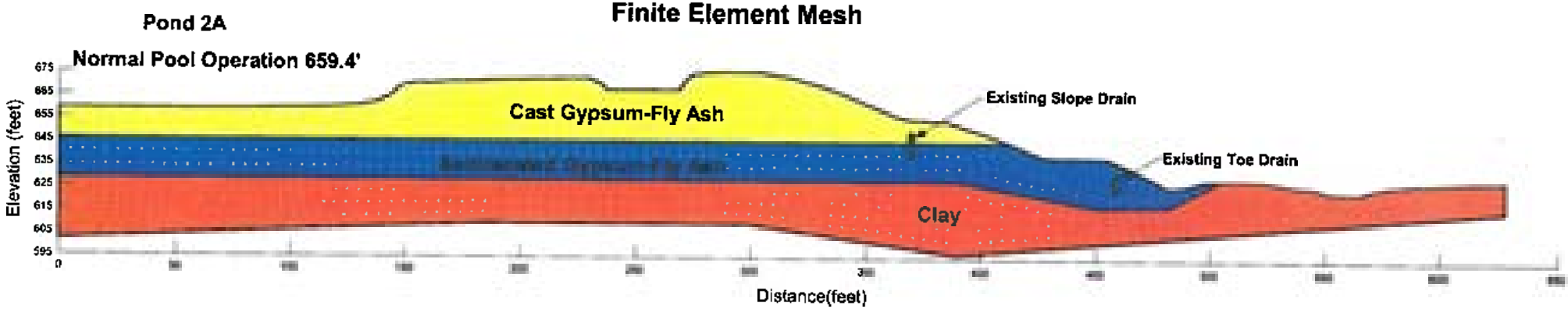
Gypsum Stack - Section A with Drains
Vertical Gradient Contours



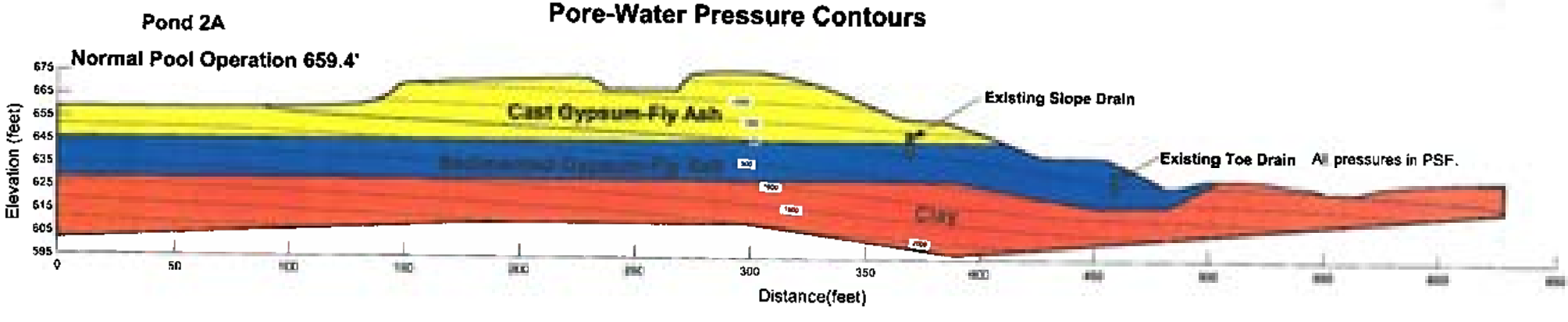
Widows Creek Fossil Plant
Gypsum Stack - Section D with Drains
Subsurface Profile and Boundary Conditions



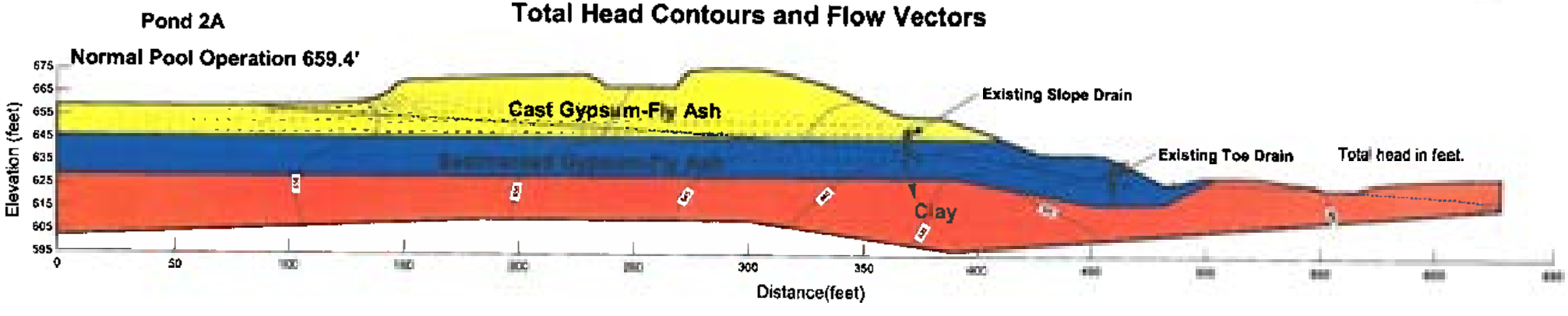
Widows Creek Fossil Plant
Gypsum Stack - Section D with Drains
Finite Element Mesh



Widows Creek Fossil Plant
Gypsum Stack - Section D with Drains
Pore-Water Pressure Contours



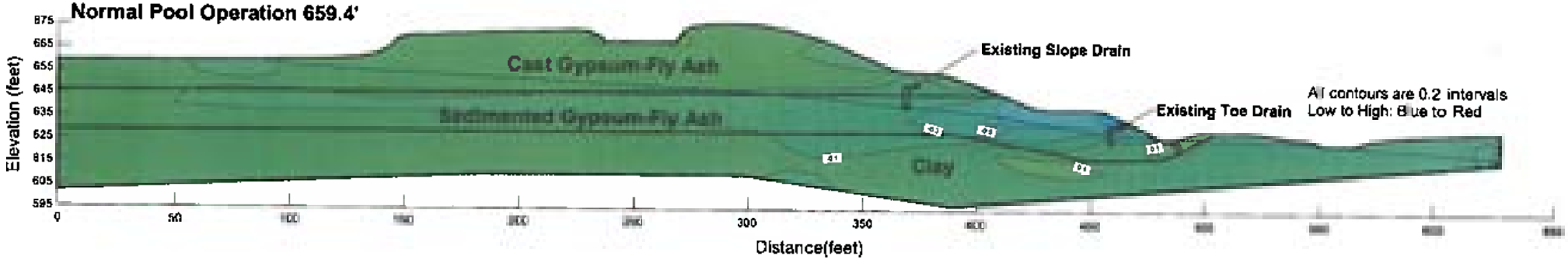
Widows Creek Fossil Plant
Gypsum Stack - Section D with Drains
Total Head Contours and Flow Vectors



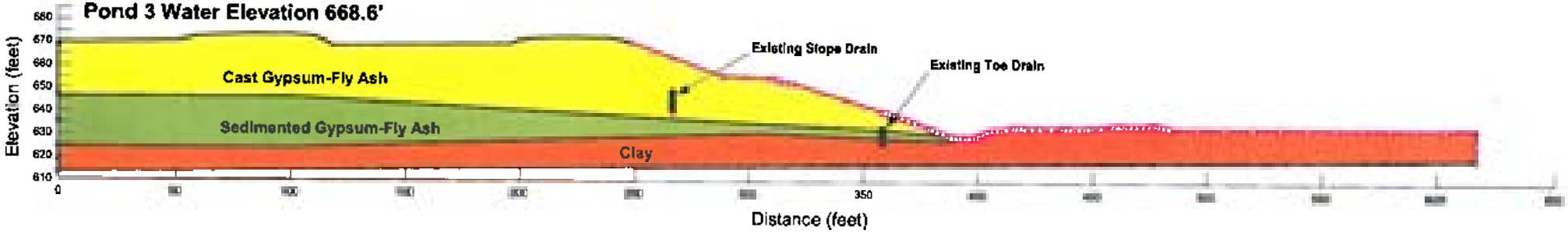
Widows Creek Fossil Plant
Gypsum Stack - Section D with Drains
Vertical Gradient Contours

Pond 2A

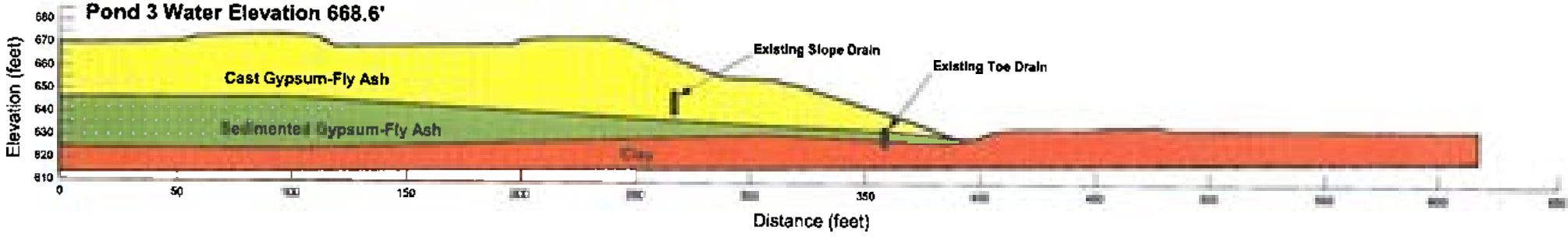
Normal Pool Operation 659.4'



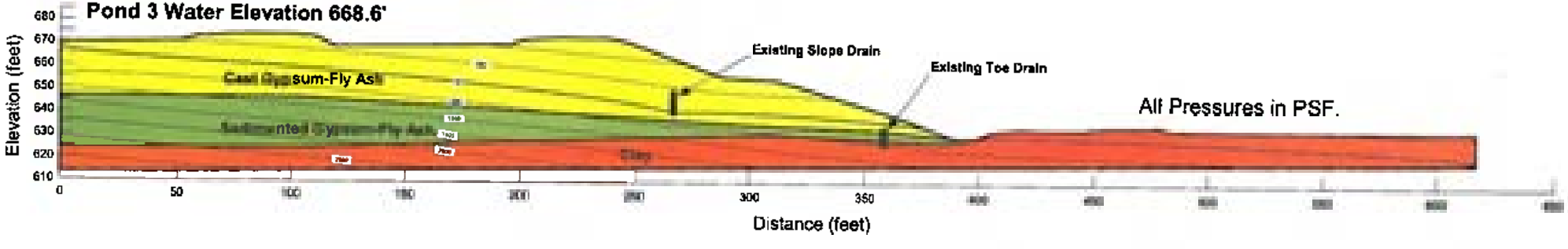
Widows Creek Fossil Plant
Gypsum Stack - Section F with Drains
Subsurface Profile and Boundary Conditions



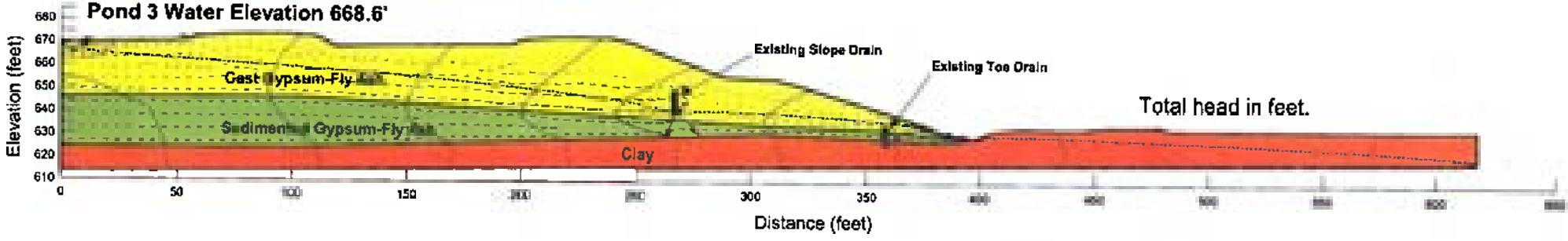
Widows Creek Fossil Plant
Gypsum Stack - Section F with Drains
Finite Element Mesh



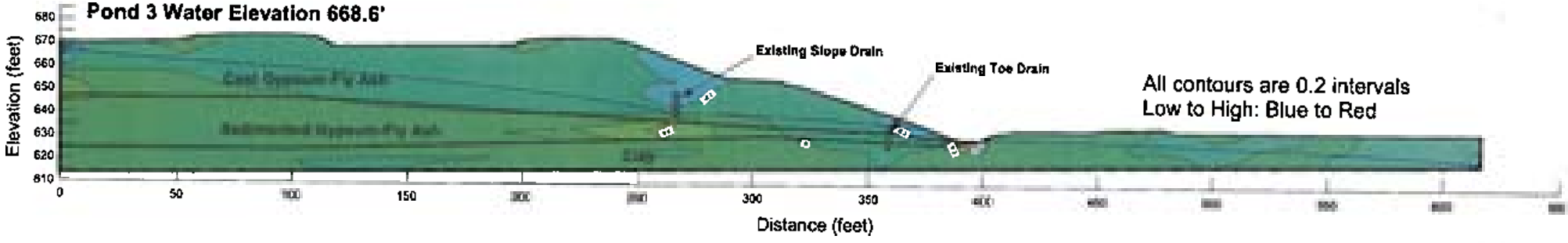
Widows Creek Fossil Plant
Gypsum Stack - Section F with Drains
Pore-Water Pressure Contours



Widows Creek Fossil Plant
Gypsum Stack - Section F with Drains
Total Head Contours and Flow Vectors

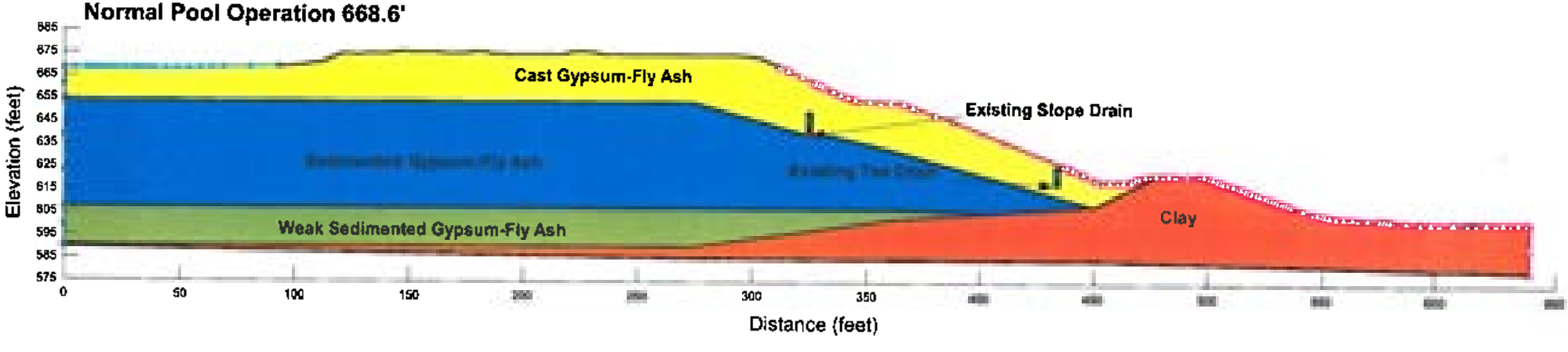


Widows Creek Fossil Plant
Gypsum Stack - Section F with Drains
Vertical Gradient Contours



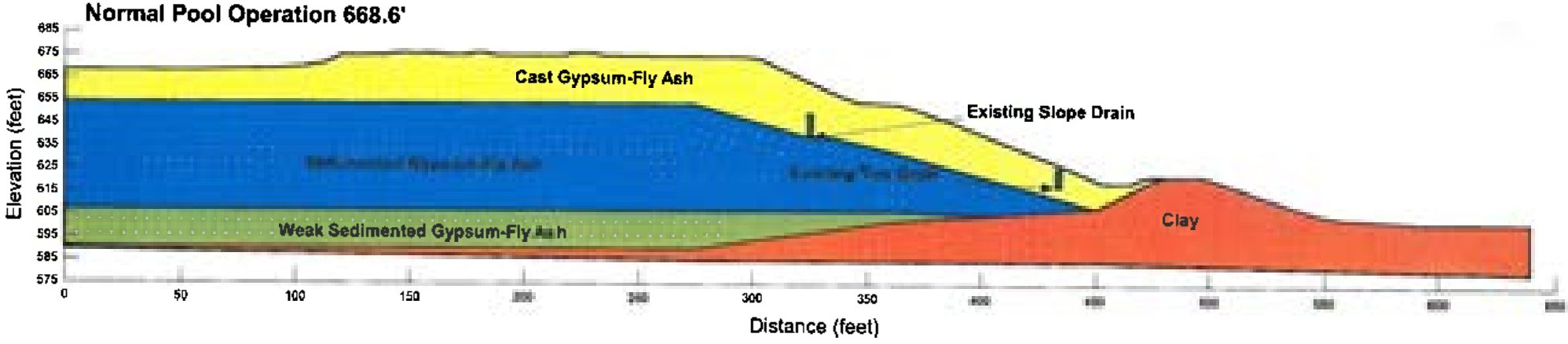
Widows Creek Fossil Plant
Gypsum Stack - Section H with Drains
Subsurface Profile and Boundary Conditions

Pond 3



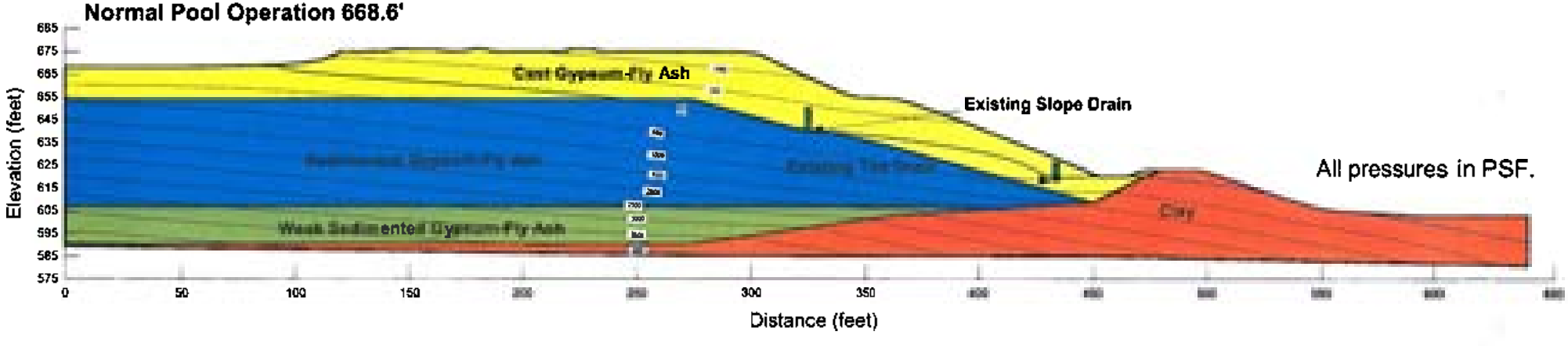
Widows Creek Fossil Plant
Gypsum Stack - Section H with Drains
Finite Element Mesh

Pond 3



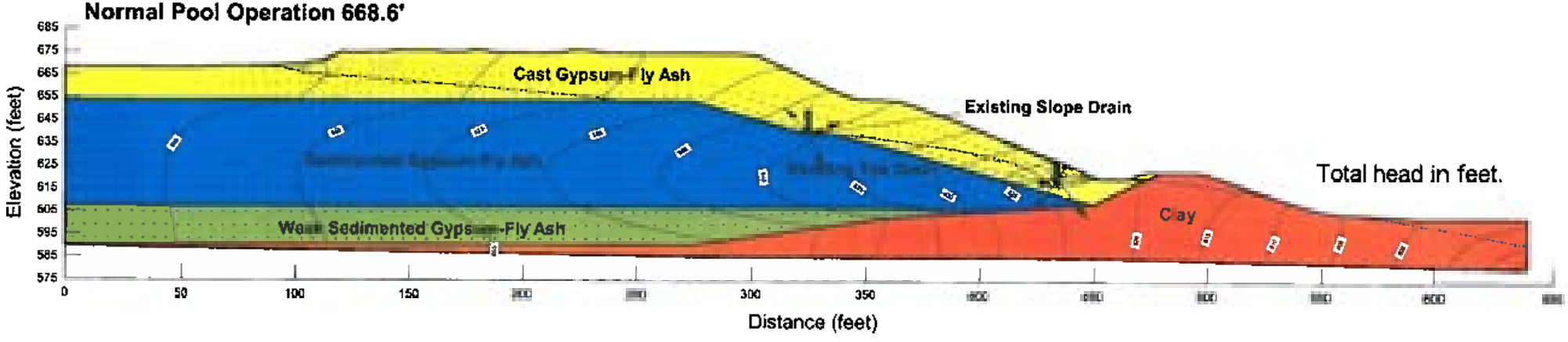
Widows Creek Fossil Plant
Gypsum Stack - Section H with Drains
Pore-Water Pressure Contours

Pond 3



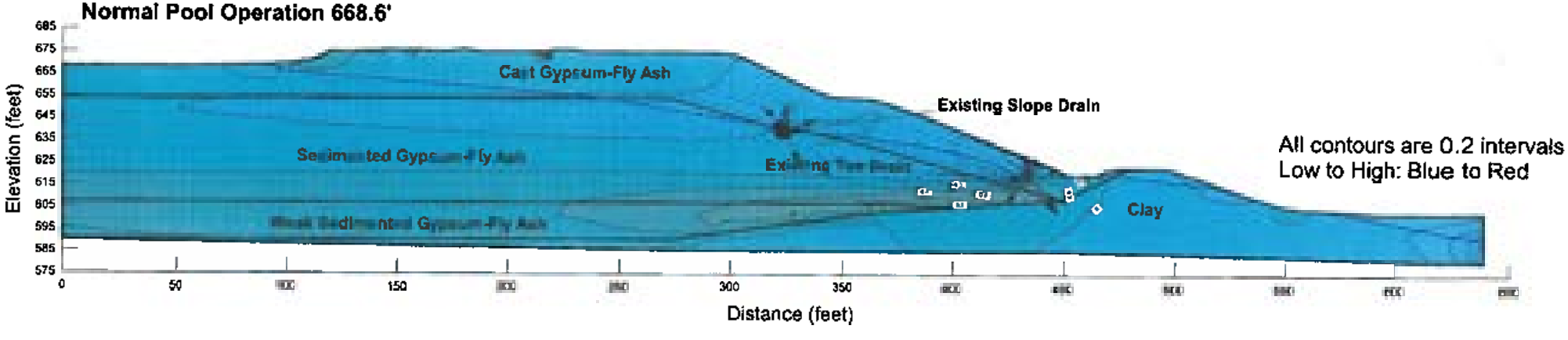
Widows Creek Fossil Plant
Gypsum Stack - Section H with Drains
Total Head Contours and Flow Vectors

Pond 3

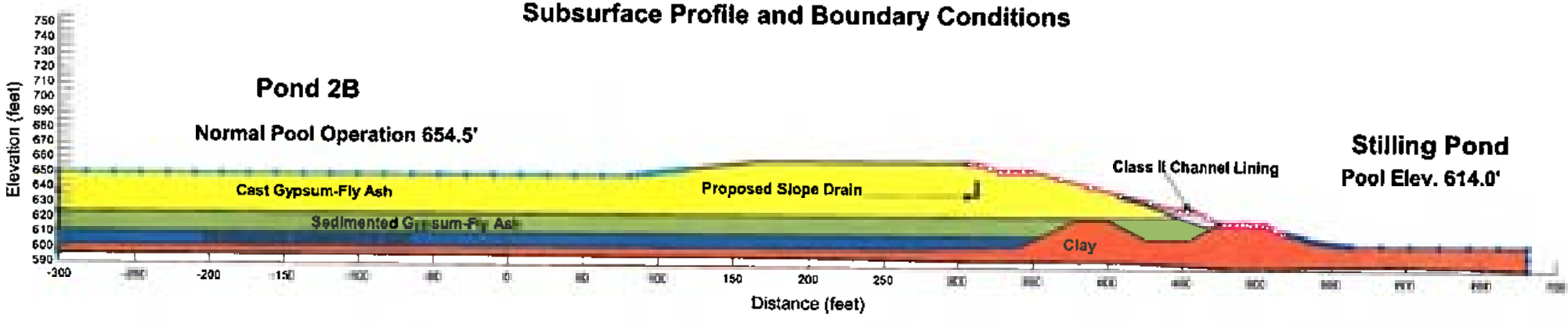


Widows Creek Fossil Plant
Gypsum Stack - Section H with Drains
Vertical Gradient Contours

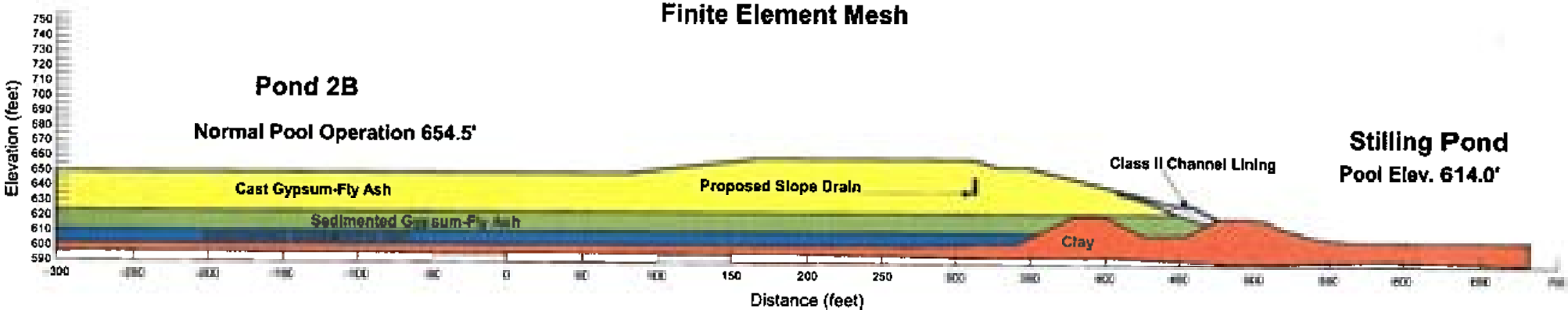
Pond 3



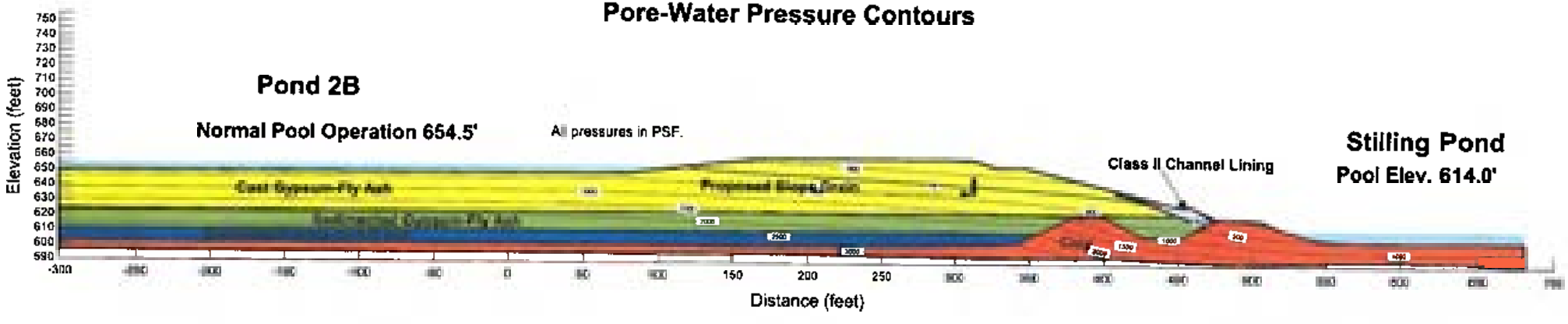
Widows Creek Fossil Plant
Gypsum Stack - Section K
Subsurface Profile and Boundary Conditions



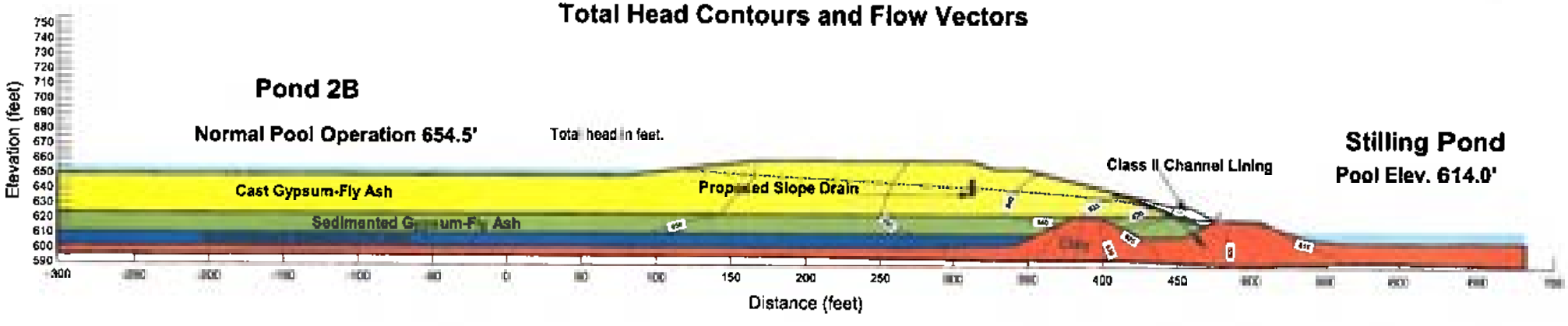
Widows Creek Fossil Plant
Gypsum Stack - Section K
Finite Element Mesh



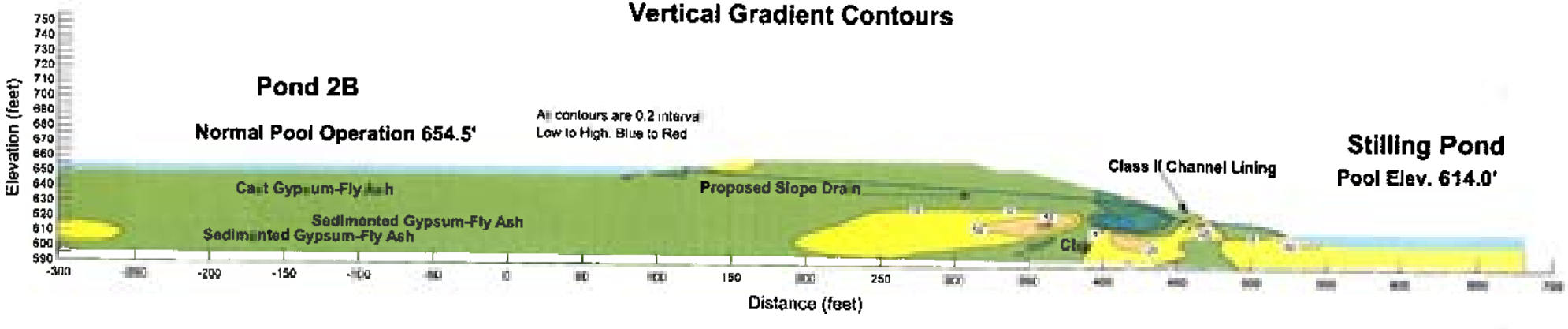
Widows Creek Fossil Plant
Gypsum Stack - Section K
Pore-Water Pressure Contours

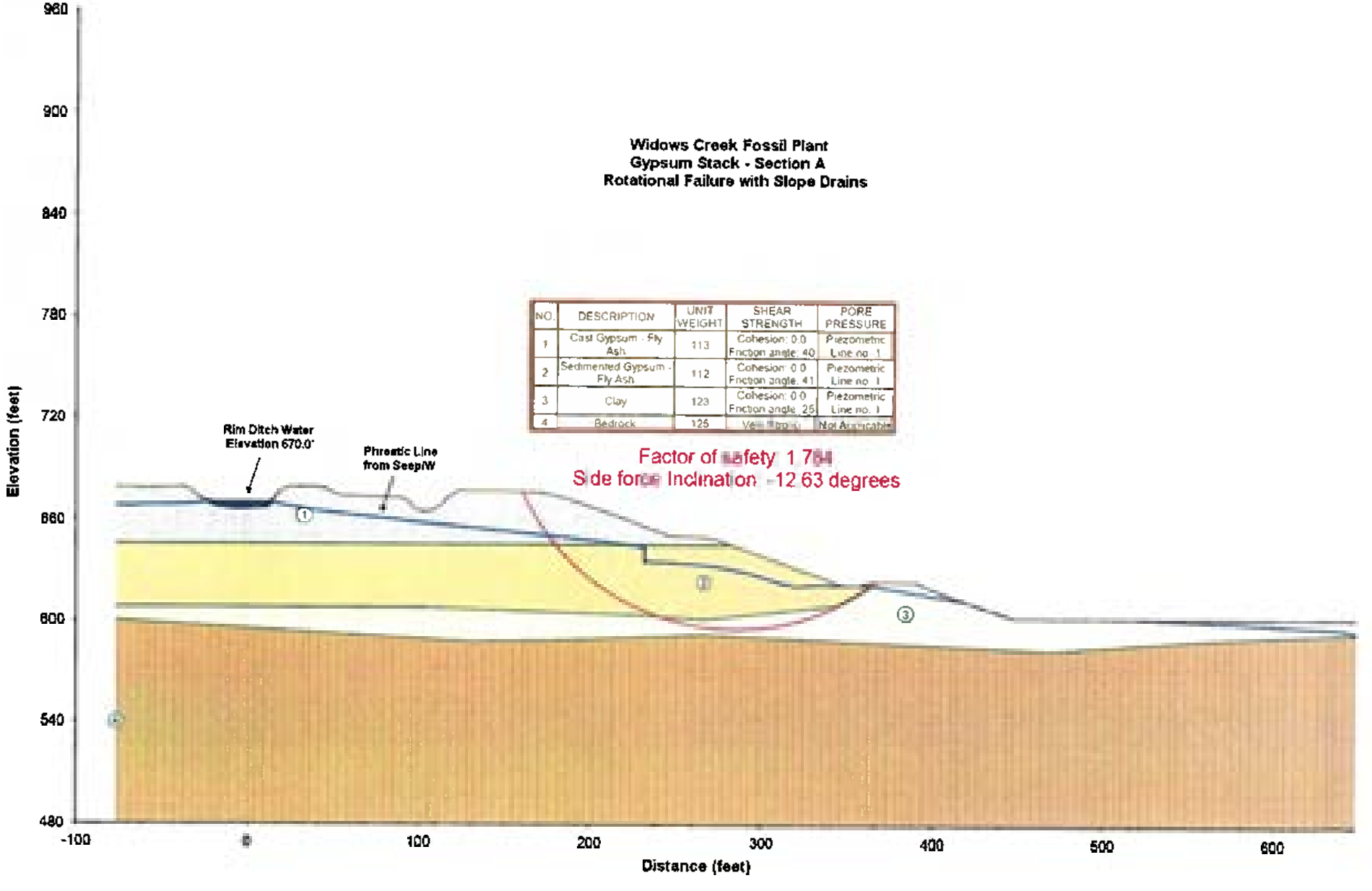


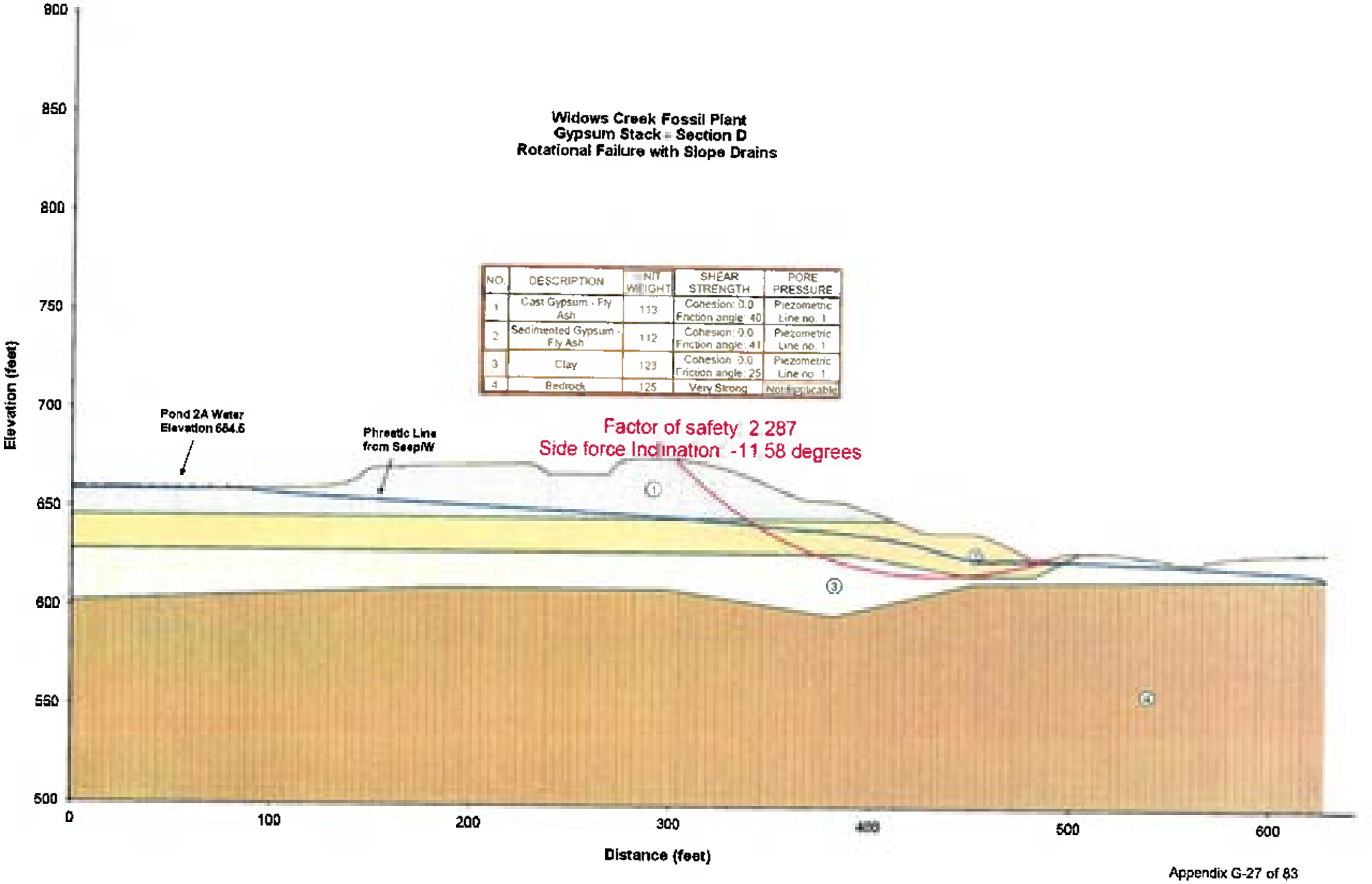
Widows Creek Fossil Plant
Gypsum Stack - Section K
Total Head Contours and Flow Vectors



Widows Creek Fossil Plant
Gypsum Stack - Section K
Vertical Gradient Contours

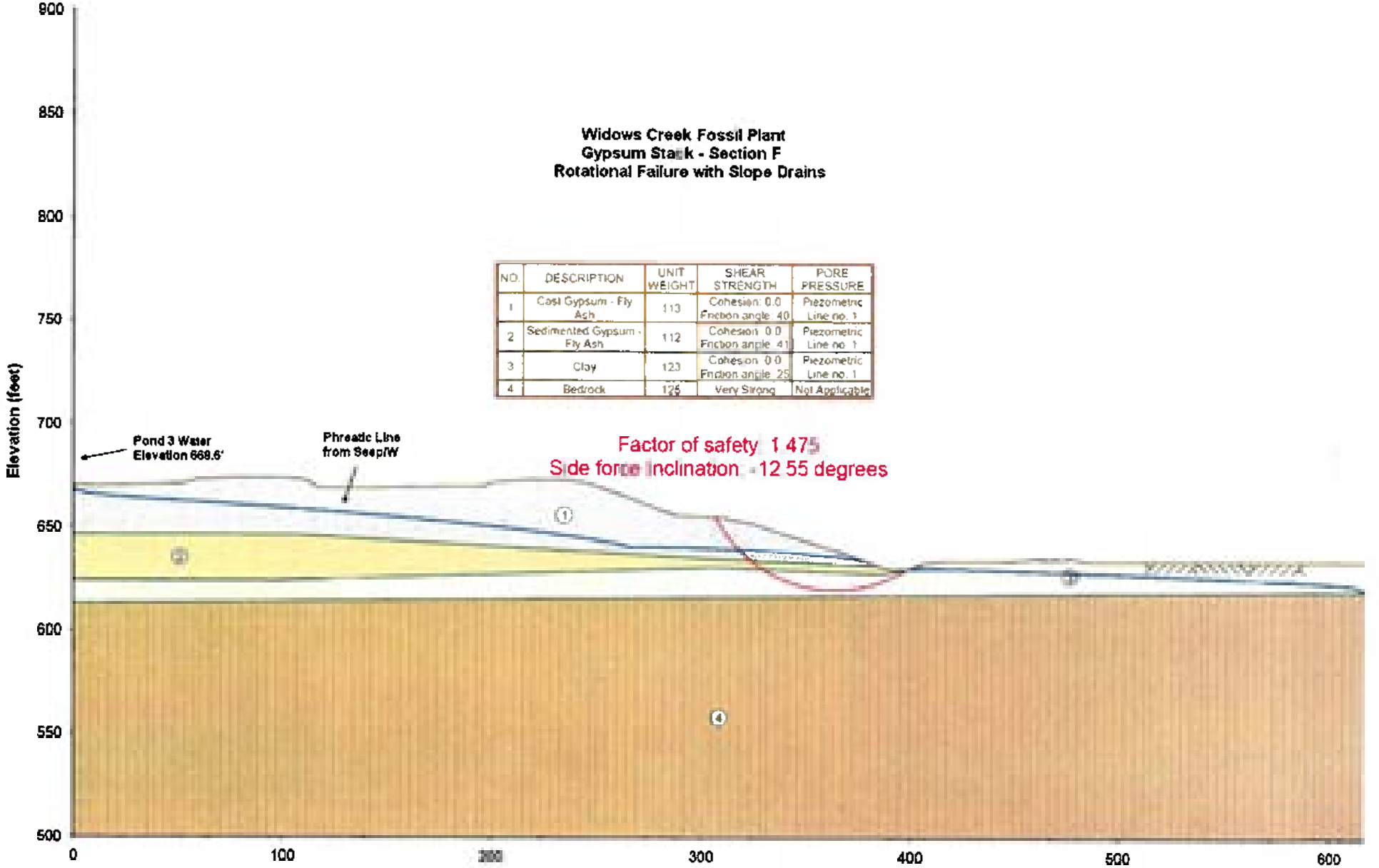



Widows Creek Fossil Plant
Gypsum Stack - Section A
Rotational Failure with Slope Drains
NO. | DESCRIPTION | UNIT WEIGHT | SHEAR STRENGTH | PORE PRESSURE
1 | Cast Gypsum - Fly Ash | 113 | Cohesion: 0.0 Friction angle: 40 | Piezometric Line no. 1
2 | Sedimented Gypsum - Fly Ash | 112 | Cohesion: 0.0 Friction angle: 41 | Piezometric Line no. 1
3 | Clay | 123 | Cohesion: 0.0 Friction angle: 25 | Piezometric Line no. 1
4 | Bedrock | 125 | Not Applicable
Rim Ditch Water Elevation 670.0'
Phreatic Line from SeepW
Factor of Safety 1.764
Side force Inclination -12.63 degrees
Elevation (feet)
960
900
840
780
720
660
600
540
480
-100
0
100
200
300
400
500
600
Distance (feet)


Widows Creek Fossil Plant
Gypsum Stack - Section D
Rotational Failure with Slope Drains
NO. | DESCRIPTION | UNIT WEIGHT | SHEAR STRENGTH | PORE PRESSURE
1 | Cast Gypsum - Fly Ash | 113 | Cohesion: 0.0 Friction angle: 40 | Piezometric Line no. 1
2 | Sedimented Gypsum - Fly Ash | 112 | Cohesion: 0.0 Friction angle: 41 | Piezometric Line no. 1
3 | Clay | 123 | Cohesion: 0.0 Friction angle: 25 | Piezometric Line no. 1
4 | Bedrock | 125 | Very Strong | Not Applicable
Pond 2A Water Elevation 684.6
Phreatic Line from SeepW
Factor of safety 2.287
Side force Inclination -11.58 degrees
Elevation (feet)
900
850
800
750
700
650
600
550
500
0
100
200
300
400
500
600
Distance (feet)

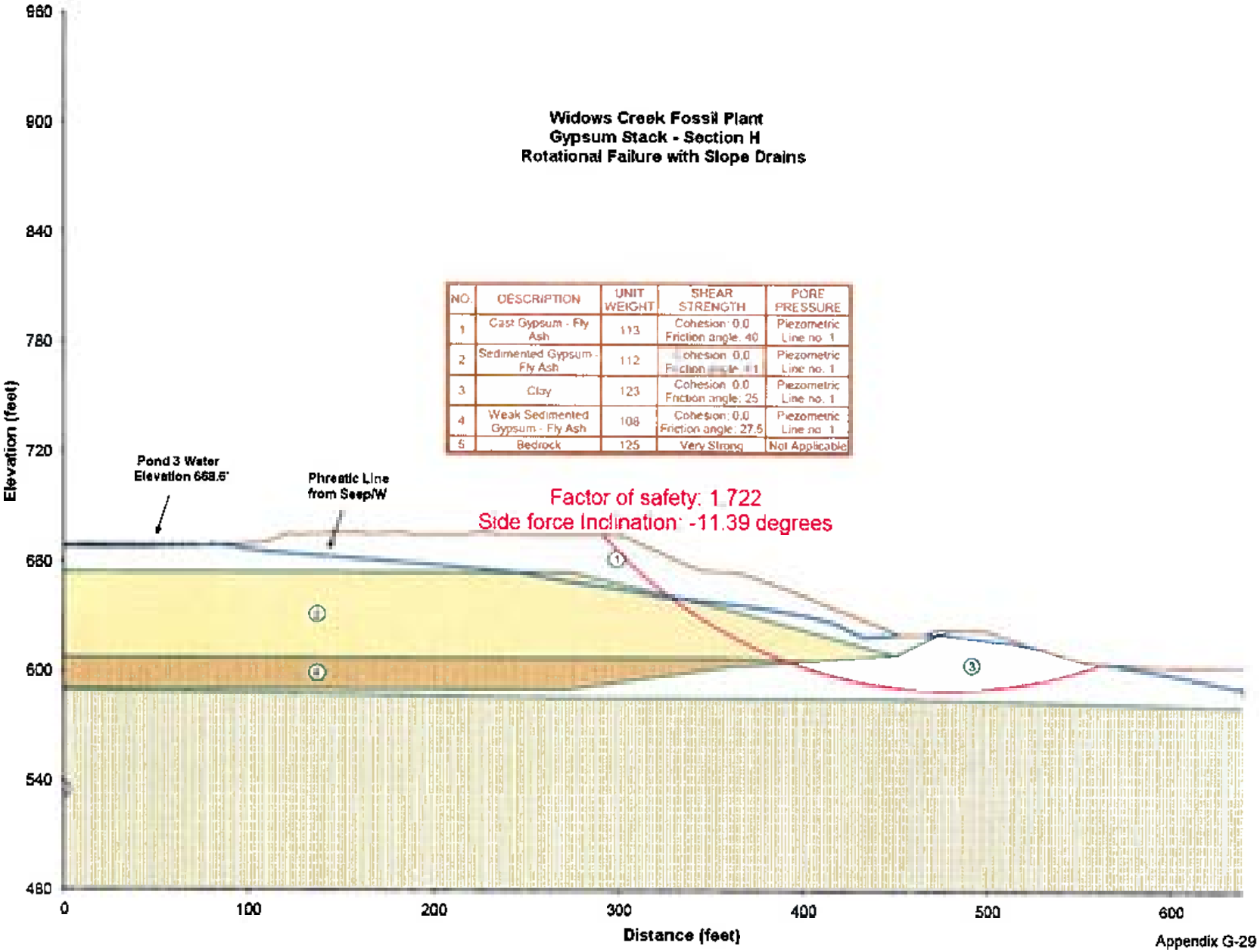
Widows Creek Fossil Plant
Gypsum Stack - Section F
Rotational Failure with Slope Drains

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Cast Gypsum - Fly Ash	113	Cohesion: 0.0 Friction angle: 40	Piezometric Line no. 1
2	Sedimented Gypsum - Fly Ash	112	Cohesion: 0.0 Friction angle: 41	Piezometric Line no. 1
3	Clay	123	Cohesion: 0.0 Friction angle: 25	Piezometric Line no. 1
4	Bedrock	125	Very Strong	Not Applicable



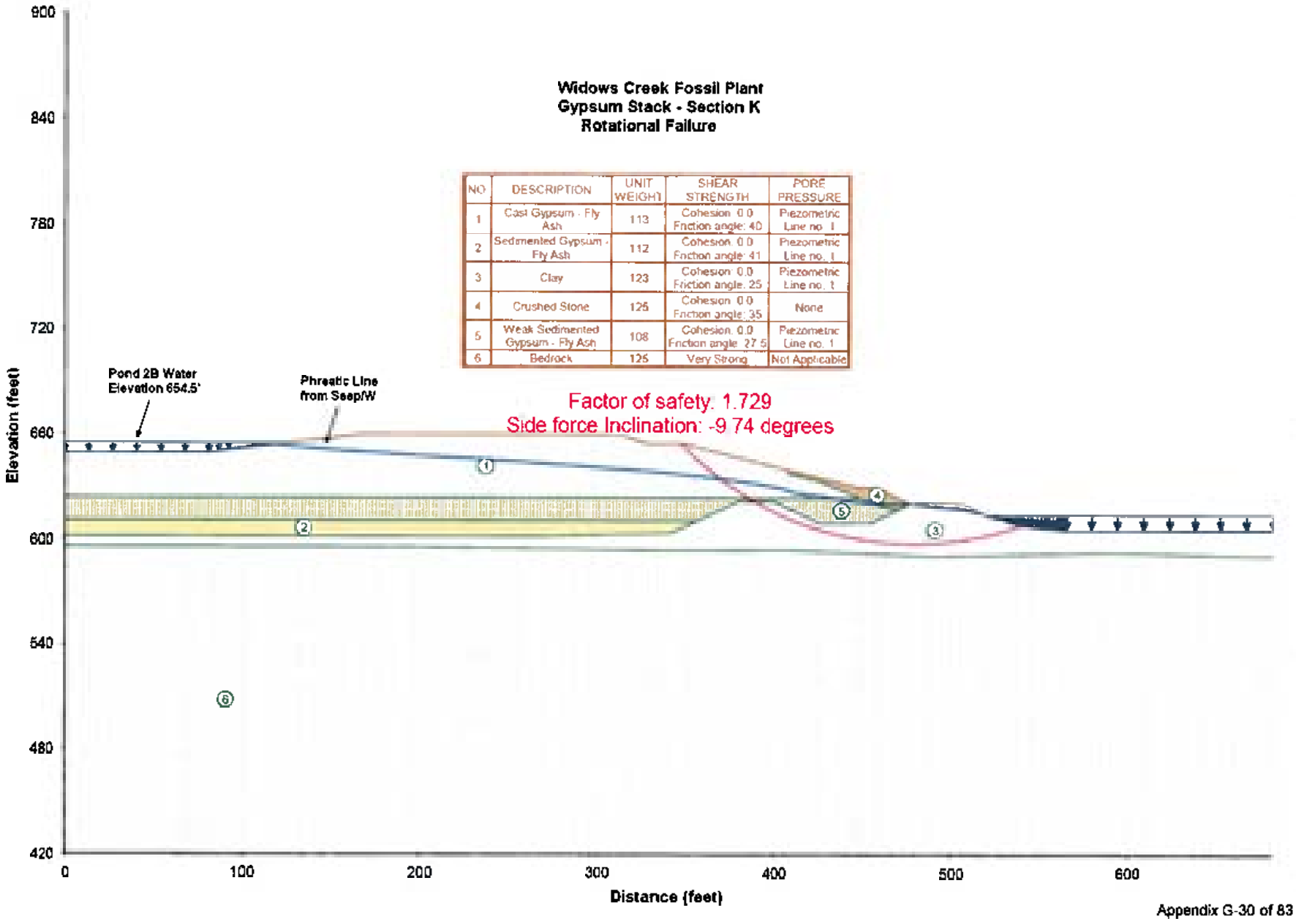
Widows Creek Fossil Plant
Gypsum Stack - Section H
Rotational Failure with Slope Drains

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Cast Gypsum - Fly Ash	113	Cohesion: 0.0 Friction angle: 40	Piezometric Line no. 1
2	Sedimented Gypsum - Fly Ash	112	Cohesion: 0.0 Friction angle: 31	Piezometric Line no. 1
3	Clay	123	Cohesion: 0.0 Friction angle: 25	Piezometric Line no. 1
4	Weak Sedimented Gypsum - Fly Ash	108	Cohesion: 0.0 Friction angle: 27.5	Piezometric Line no. 1
5	Bedrock	125	Very Strong	Not Applicable



Widows Creek Fossil Plant
Gypsum Stack - Section K
Rotational Failure

NO	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Cast Gypsum - Fly Ash	113	Cohesion: 0.0 Friction angle: 40	Piezometric Line no. 1
2	Sedimented Gypsum - Fly Ash	112	Cohesion: 0.0 Friction angle: 41	Piezometric Line no. 1
3	Clay	123	Cohesion: 0.0 Friction angle: 25	Piezometric Line no. 1
4	Crushed Stone	125	Cohesion: 0.0 Friction angle: 35	None
5	Weak Sedimented Gypsum - Fly Ash	108	Cohesion: 0.0 Friction angle: 27.5	Piezometric Line no. 1
6	Bedrock	125	Very Strong	Not Applicable



CPT Data Analysis Summary

The coefficient of consolidation in the horizontal direction (c_h) was determined using the chart proposed by Roberston et al. (1992). Robertson et al. (1992) had reviewed dissipation data from piezocone tests and compared these with values obtained in the lab or field. In order to use the chart proposed by Robertson et al. (1992) the time to reach U_{50} is determined in minutes. This value is a function of the pore pressure measured at the start of the dissipation test (Push pore pressure) and the location of the static groundwater table (hydrostatic pressure). In order to accurately predict the static groundwater level, piezometer data taken from the nearest piezometers was used. In some cases, the dissipation tests were run long enough for the to reach hydrostatic pressure. When this occurred, the depth to the groundwater table was determined using the hydrostatic pressure measured during the dissipation test. The time to reach U_{50} (t_{50}) was then determined and used to determine c_h. It should be noted that the chart provided by Robertson et al. (1992) provides an upper and lower bound for c_h values. As such the c_h value presented on the graph is the average of the upper and lower bound c_h values determined using the chart.

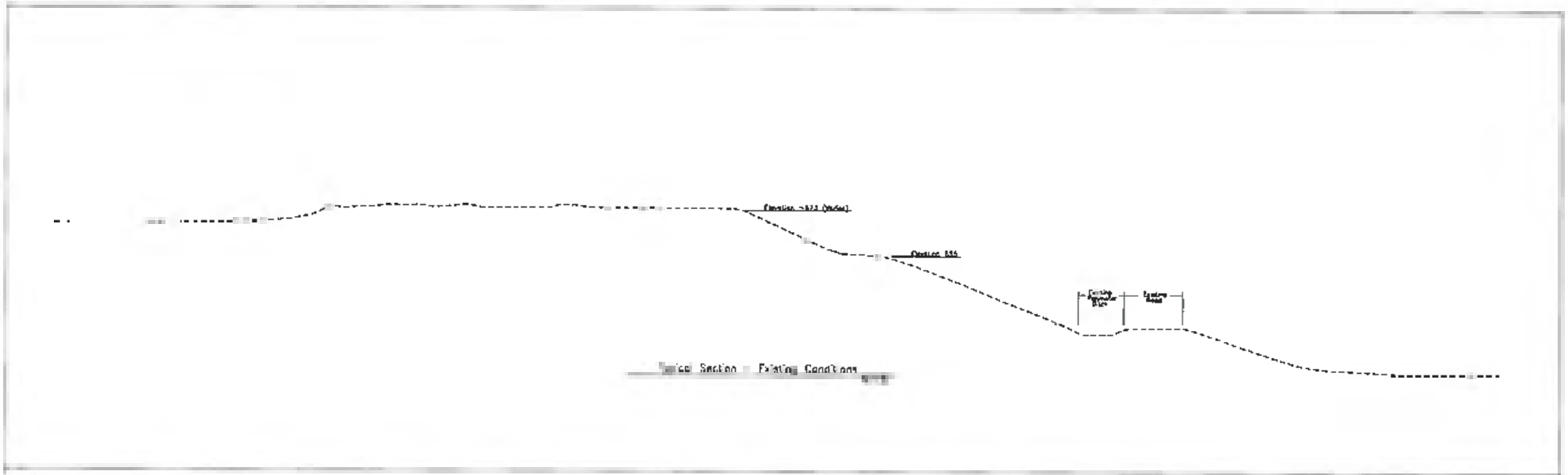
The hydraulic conductivity in the horizontal direction (k_h) is determined in a similar fashion using the chart proposed by Robertson et al. (1992). Once again, a lower and upper value of k_h is determined using t_{50}, and an average k_h value is presented. In this case, the relationship between k_h values in laboratory and field tests, and t50 is not as good as that observed between c_h and t_{50}. This is reflected in the difference in k_h estimates (which is approximately 1.5 orders of magnitude). As such it is typically recommended that the k_h values obtained from dissipation tests be only used as a guide.

Soil parameters (angle of friction, N60, and s_u) were determined using empirical correlations provided in the literature (and as presented by Lunne et al. 1997). These empirical relationships were determined using various natural soils. It should be noted that bottom ash and fly ash were not part of the data sets used to determine these relationships. As such these values should be used as a guide until enough empirical data can be used to very that these relationships apply for bottom ash and fly ash.

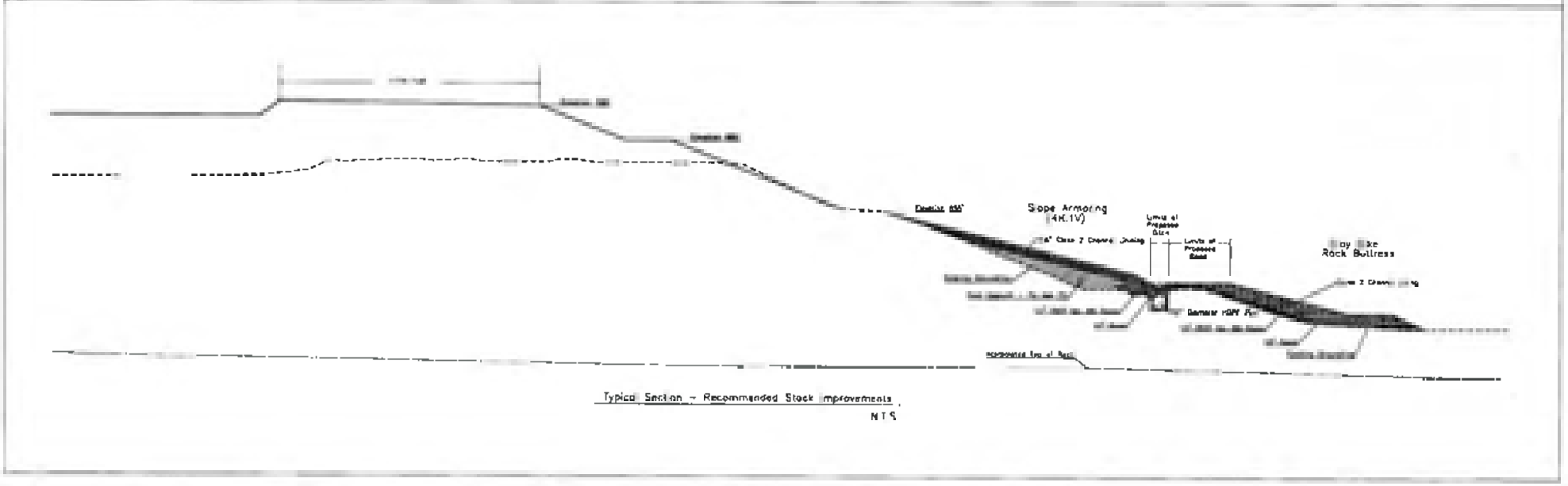
Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne. T., Powell, J.J.M., and Gillespie, D.G. (1992b) "Estimating coefficient of consolidation from piezocone tests" Canadian Geotechnical Journal, 29 (4), 551-557

Robertson, P.K., Woeller, D.J., and Finn, W.D.L. (1992) "Seismic cone penetration test for evaluating liquefaction potential under cyclic loading" Canadian Geotechnical Journal, 29 (4), 686-695

Lunne, T., Robertson, P.K., and Powell, J.J.M. (1997) "Cone Penetration Testing in Geotechnical Engineering", E&FN Spon, New York, NY

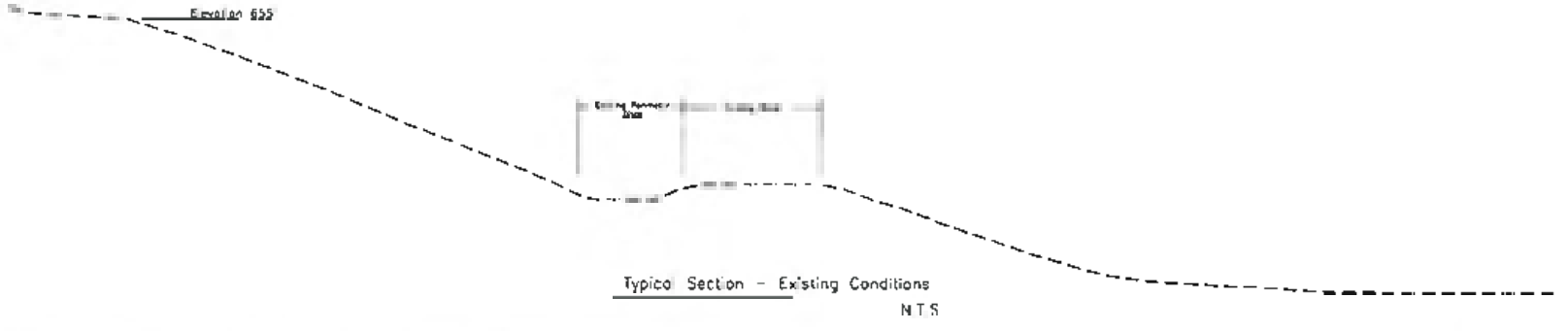


Typical Section - Existing Conditions

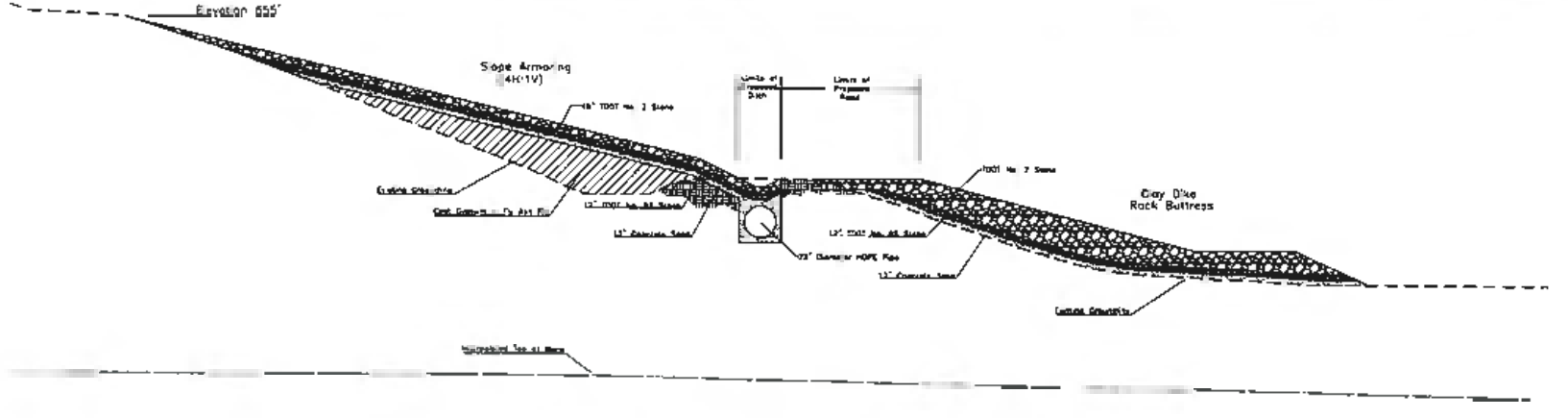


Typical Section - Recommended Stock Improvements

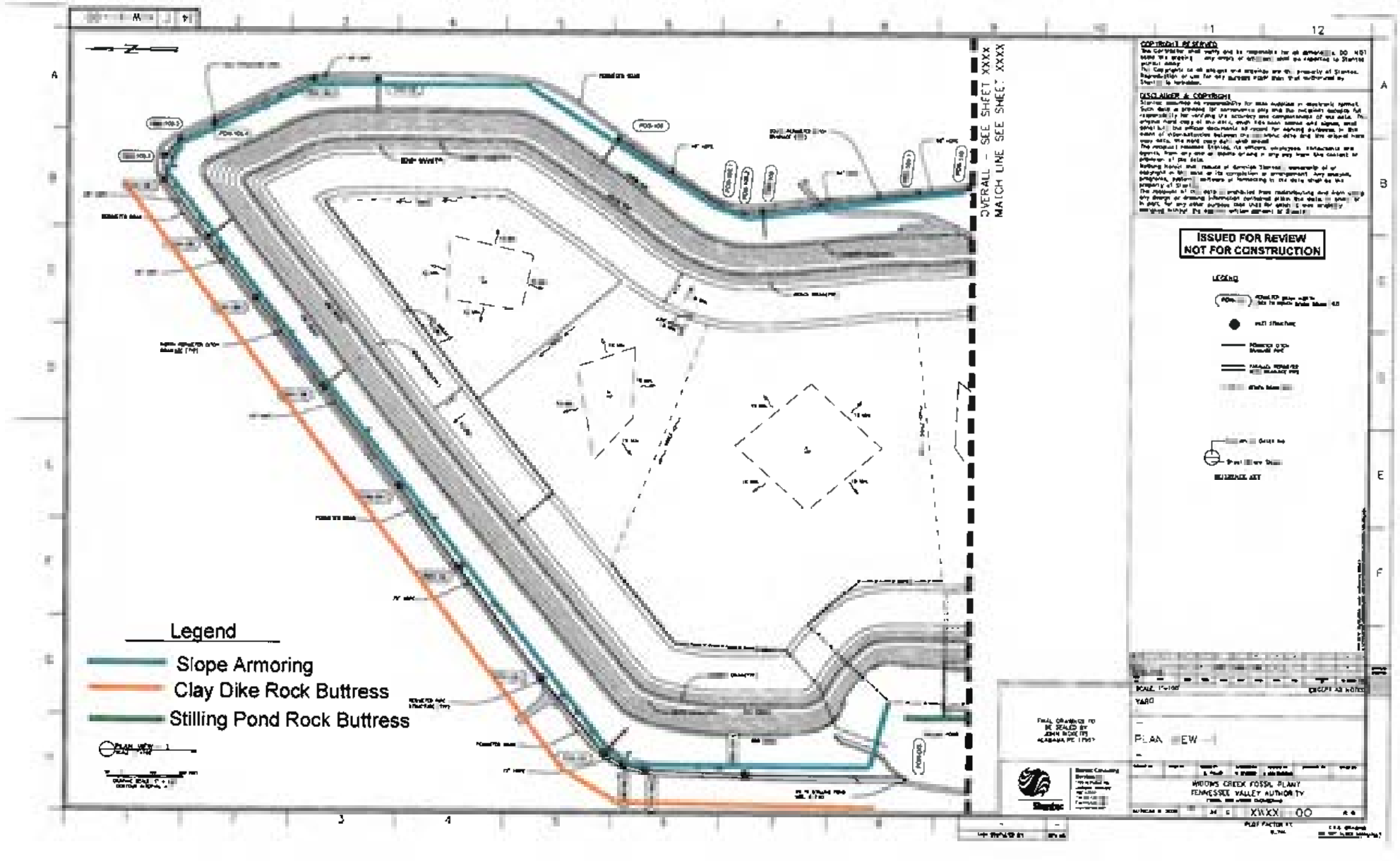
NTS

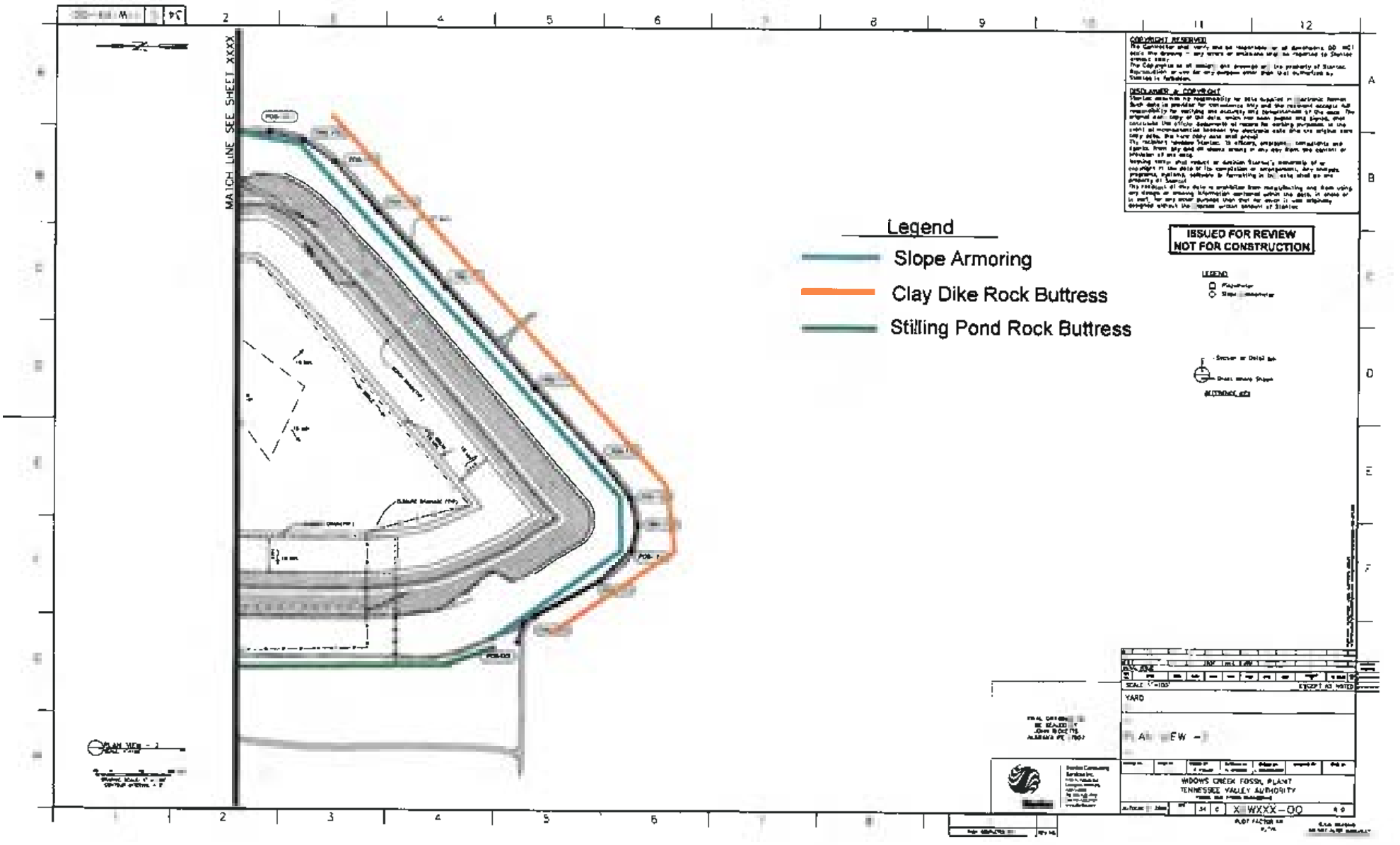


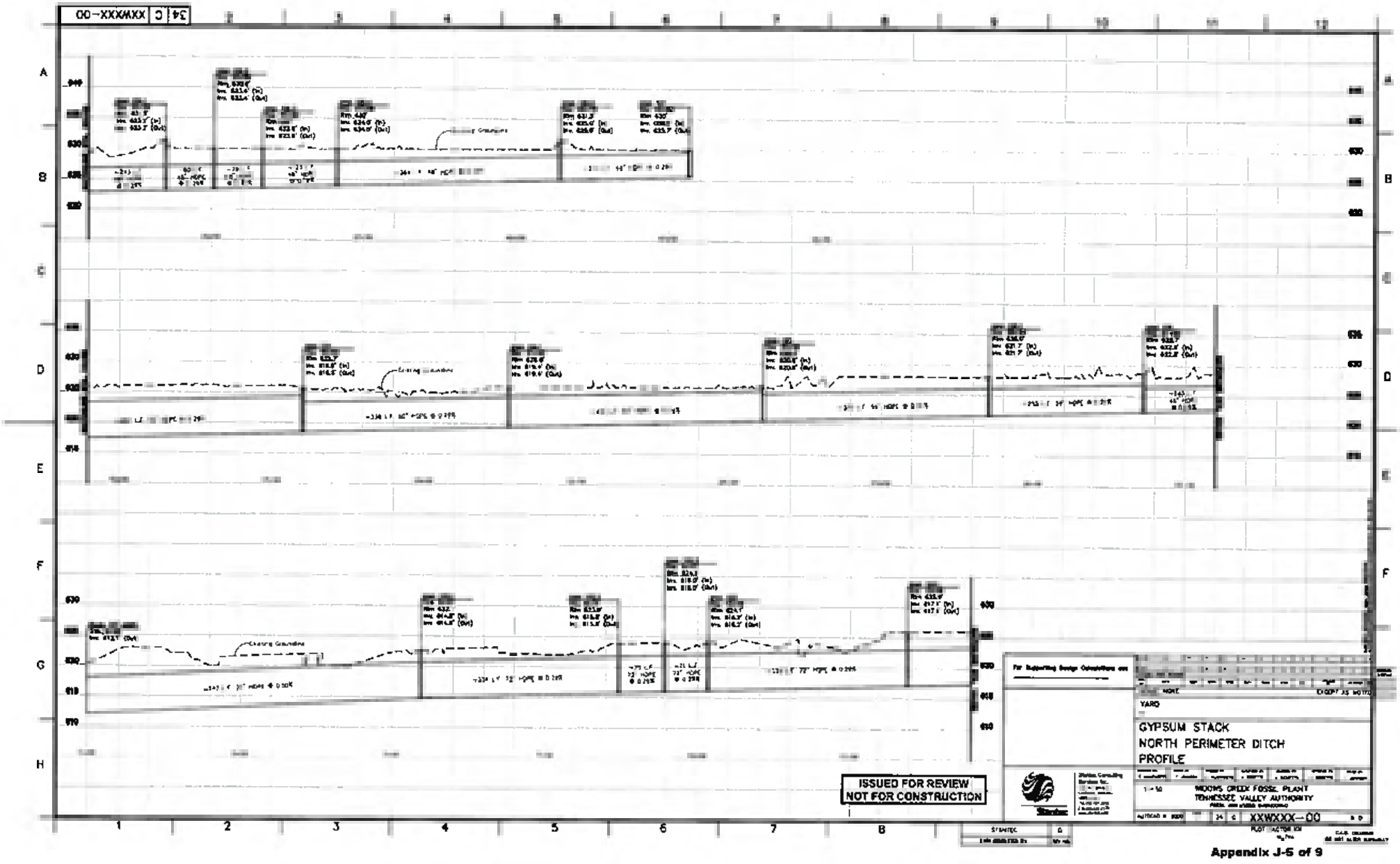
Typical Section – Existing Conditions
N.T.S

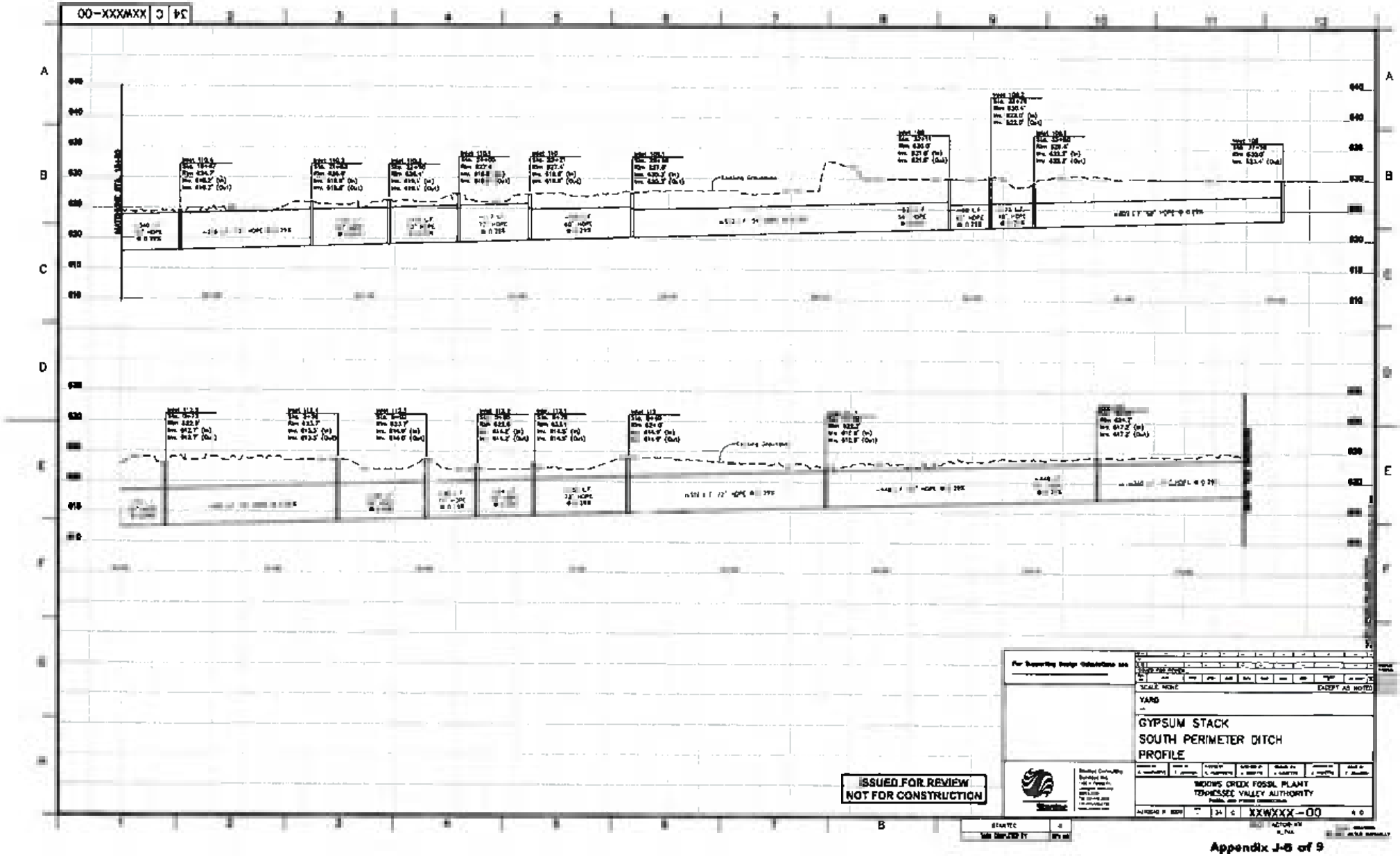


Typical Section – Recommended Stock Improvements
N.T.S


OVERALL - SEE SHEET XXXX
MATCH LINE SEE SHEET XXXX
ISSUED FOR REVIEW
NOT FOR CONSTRUCTION
Legend
Slope Armoring
Clay Dike Rock Buttress
Stilling Pond Rock Buttress
PLAN VIEW
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
XXXX 00


MATCH LINE SEE SHEET XXXX
Legend
Slope Armoring
Clay Dike Rock Buttress
Stilling Pond Rock Buttress
ISSUED FOR REVIEW
NOT FOR CONSTRUCTION
YARD
WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
X WXXX-00


ISSUED FOR REVIEW
NOT FOR CONSTRUCTION
GYPSUM STACK
NORTH PERIMETER DITCH
PROFILE
MOORE
EXCEPT AS NOTED
YARD
1" = 50'
MOORE CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
XXWXXX-00


ISSUED FOR REVIEW
NOT FOR CONSTRUCTION
YARD
GYPSUM STACK
SOUTH PERIMETER DITCH
PROFILE
WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
XXWXXX-00

Work Plan 14 - Closure Plan
Widows Creek Fossil Plant, Stevenson, Alabama
Stantec Project No. 175569039

Feasibility Design Phase - Construction Cost Estimate

Project Title: Gypsum Stack, Widows Creek Fossil Plant
Project Name: Closure Plan — Work Plan 14
Proj. Location: Stevenson, Alabama — 175569039 — **Prep. By:** ZCM
Estimate Date: 23-Oct-09 — Construction — **Currency:** USD

Item	Unit	Quant.	Unit Cost	Total Cost	Percentages	
Site Preparation						
(1) Mobilization/Demobilization	LS	1	$100,000.00	$100,000	0.6%	of TDC
(2) Clearing and Grubbing	AC	20	$1,000.00	$20,000	0.1%	of TDC
(3) BMP Silt Fence Installation	LF	32,000	$2.50	$80,000	0.5%	of TDC
Site Preparation Subtotal				***200,000***	***1.2%***	***of TDC***
Slope Stability						
Rock Buttress for Clay Dike						
(4) Sand	CY	21,000	$12.10	$254,100	1.5%	of TDC
(5) No. 89 Stone	CY	19,700	$38.00	$748,600	4.4%	of TDC
(6) Class 2 Channel Lining	CY	88,000	$38.00	$3,344,000	19.6%	of TDC
Slope Armoring for Stack						
(7) Sand	CY	27,700	$12.10	$335,170	2.0%	of TDC
(8) No. 89 Stone	CY	30,000	$38.00	$1,140,000	6.7%	of TDC
(9) Class 2 Channel Lining	CY	36,500	$38.00	$1,387,000	8.1%	of TDC
(10) Cast Gypsum Fill, w/ Foundation Benching on Slope	CY	107,600	$4.00	$430,400	2.5%	of TDC
(11) Sand for seepage collection in old perimeter ditch	CY	20,000	$12.10	$242,000	1.4%	of TDC
(12) 8" HDPE Perforated Pipe for seepage collection	LF	9,000	$5.63	$50,670	0.3%	of TDC
Stilling Pond Rock Buttress						
(13) Sand	CY	1,850	$12.10	$22,385	0.1%	of TDC
(14) No. 89 Stone	CY	1,670	$38.00	$63,460	0.4%	of TDC
(15) Class 2 Channel Lining	CY	6,300	$38.00	$239,400	1.4%	of TDC
(16) Temporary Water Diversion Pump System	LS	1	$50,000.00	$50,000	0.3%	of TDC
(17) 36" HDPE Pipe Extension (x2)	LF	70	$45.00	$3,150	0.0%	of TDC
(18) 30" HDPE Pipe Extension	LF	35	$37.00	$1,295	0.0%	of TDC
(19) 18" HDPE Pipe Extension (x2)	LF	70	$17.00	$1,190	0.0%	of TDC
(20) 8" HDPE Pipe Extension (x5)	LF	175	$5.63	$985	0.0%	of TDC
Slope Stability Subtotal				***$8,257,185***	***48.4%***	***of TDC***
Perimeter Ditch						
Earthwork for North & South Pipe Installation						
(21) Trench Excavation	CY	22,055	$2.50	$55,138	0.3%	of TDC
(22) Sand Backfill	CY	10,729	$12.10	$129,821	0.8%	of TDC
(23) Roadway, crushed 1.5" stone, compacted, 6" deep	SY	27,000	$10.00	$270,000	1.6%	of TDC
Earthwork for Inlet Installation						
(24) Excavation	CY	5,100	$2.50	$12,750	0.1%	of TDC
(25) Stone Base	CY	143	$38.00	$5,434	0.0%	of TDC
(26) Sand Backfill	CY	4,454	$12.10	$53,893	0.3%	of TDC
North Pipe Installation						
(27) 48" diameter	LF	1,102	$70.00	$77,140	0.5%	of TDC
(28) 54" diameter	LF	627	$104.00	$65,208	0.4%	of TDC
(29) 60" diameter	LF	754	$120.00	$90,480	0.5%	of TDC
(30) 72" diameter	LF	1,802	$150.00	$270,300	1.6%	of TDC
South Pipe Installation						
(31) 48" diameter	LF	549	$70.00	$38,430	0.2%	of TDC
(32) 54" diameter	LF	522	$104.00	$54,288	0.3%	of TDC
(33) 60" diameter	LF	168	$120.00	$20,160	0.1%	of TDC
(34) 72" diameter	LF	2,523	$150.00	$378,450	2.2%	of TDC
Surface Drop Inlets						
(35) North	each	16	$4,000.00	$64,000	0.4%	of TDC
(36) South	each	18	$4,000.00	$72,000	0.4%	of TDC
Connections to Seepage Collection System						
(37) Inspection Wells Connection	each	20	$3,500.00	$70,000	0.4%	of TDC
Perimeter Ditch Subtotal				***$1,727,492***	***10.1%***	***of TDC***

Work Plan 14 - Closure Plan
Widows Creek Fossil Plant, Stevenson, Alabama
Stantec Project No. 175569039

Feasibility Design Phase - Construction Cost Estimate

Project Title: Gypsum Stack, Widows Creek Fossil Plant
Project Name: Closure Plan — Work Plan 14
Proj. Location: Stevenson, Alabama — 175569039 — ***Prep. By:*** ZCM
Estimate Date: 23-Oct 09 — Construction — ***Currency:*** USD

Item	Unit	Quantity	Unit Cost	Total Cost	Percentages	
Slope Drains						
655 Bench Outlets 1,2,8,9						
(38) 8" HDPE Perforated Slope Drain Pipe	LF	2,820	$5.63	$15,877	0.1%	of TDC
(39) Slope Drain Trench Excavation	CY	39,750	$2.50	$99,375	0.6%	of TDC
(40) Slope Drain Sand Backfill	CY	3,720	$12.10	$45,012	0.3%	of TDC
(41) Slope Drain Backfill	CY	37,600	$12.00	$451,200	2.6%	of TDC
(42) 8" HDPE Solid Lateral Pipe	LF	600	$5.63	$3,378	0.0%	of TDC
(43) Lateral Outlet Trench Excavation	CY	2,860	$2.50	$7,150	0.0%	of TDC
(44) Lateral Outlet Sand Backfill	CY	140	$12.10	$1,694	0.0%	of TDC
(45) Lateral Outlet Backfill	CY	2,720	$12.00	$32,640	0.2%	of TDC
685 Bench Entire Perimeter						
(46) 8" HDPE Perforated Slope Drain Pipe	LF	8,775	$5.63	$49,403	0.3%	of TDC
(47) Slope Drain Trench Excavation	CY	41,600	$2.50	$104,000	0.6%	of TDC
(48) Slope Drain Sand Backfill	CY	6,500	$12.10	$78,650	0.5%	of TDC
(49) Slope Drain Backfill	CY	35,100	$12.00	$421,200	2.5%	of TDC
(50) 8" HDPE Solid Lateral Pipe	LF	1,500	$5.63	$8,445	0.0%	of TDC
(51) Lateral Outlet Trench Excavation	CY	7,200	$2.50	$18,000	0.1%	of TDC
(52) Lateral Outlet Sand Backfill	CY	340	$12.10	$4,114	0.0%	of TDC
(53) Lateral Outlet Backfill	CY	6,860	$12.00	$82,320	0.5%	of TDC
Slope Drains Subtotal				***$1,422,458***	*8.3%*	*of TDC*
Bench Drain System						
Earthwork for Surface Inlets						
(54) 18" Bench Drain	CY	3,410	$11.00	$37,510	0.2%	of TDC
(55) 30" Bench Drain	CY	888	$11.00	$9,768	0.1%	of TDC
(56) TDOT No. 89 Fill (per inlet)	CY	39	$38.00	$1,482	0.0%	of TDC
(57) Sand Backfill 18" Drain	CY	2,970	$12.10	$35,937	0.2%	of TDC
(58) Sand Backfill 30" Drain	CY	780	$12.10	$9,438	0.1%	of TDC
Surface Drop Inlets						
(59) 655 Bench Drain Inlet and Concrete Apron	each	13	$5,000.00	$65,000	0.4%	of TDC
(60) 685 Bench Drain Inlet and Concrete Apron	each	13	$5,000.00	$65,000	0.4%	of TDC
Earthwork Lateral Pipe						
(61) Excavation, Trench 15' deep	CY	15,273	$3.00	$45,819	0.3%	of TDC
(62) Sand Backfill	CY	913	$12.00	$10,956	0.1%	of TDC
(63) Cast Gypsum Fill	CY	14,207	$4.00	$56,828	0.3%	of TDC
Lateral Pipe						
(64) 18" diameter	LF	1,830	$17.00	$31,110	0.2%	of TDC
(65) 30" diameter	LF	910	$37.00	$33,670	0.2%	of TDC
Bench Regrading						
(66) Ditch Regrade	CY	30,800	$11.00	$338,800	2.0%	of TDC
Vegetative Soil Cover						
(67) Topsoil placement and grading, 12" depth	AC	62	$43,560.00	$2,700,720	15.8%	of TDC
(68) Seeding, Hydro or air with mulch and fertilizer	AC	62	$2,615.00	$162,130	1.0%	of TDC
Bench Drain System Subtotal				***$3,604,168***	*21.1%*	*of TDC*
Cap Drain System						
Earthwork						
(69) Excavation for Inlets	CY	2,886	$11.00	$31,746	0.2%	of TDC
(70) TDOT No. 89 Fill	CY	20	$38.00	$741	0.0%	of TDC
(71) Sand Backfill	CY	2,535	$12.10	$30,674	0.2%	of TDC
Surface Drop Inlets						
(72) Cap Drain Inlet with Concrete Apron	each	13	$5,000.00	$65,000	0.4%	of TDC
Earthwork Cap Drain Pipe						of TDC
(73) Excavation, 5 feet deep	CY	1,614	$3.00	$4,842	0.0%	of TDC
(74) Sand Backfill	CY	1,003	$12.10	$12,136	0.1%	of TDC
(75) Compacted Earth Backfill	CY	390	$12.00	$4,680	0.0%	of TDC
Cap Drain Pipe						
(76) 24" diameter	LF	1,250	$24.00	$30,000	0.2%	of TDC
(77) 30" diameter	LF	1,240	$37.00	$45,880	0.3%	of TDC
(78) 36" diameter	LF	1,650	$45.00	$74,250	0.4%	of TDC
Cap Drain Regrading						
(79) Regrade cap	CY	140,000	$11.00	$1,540,000	9.0%	of TDC
Vegetative Soil Cover						
(80) Topsoil placement and grading, 12" depth	AC	50	$43,560.00	$2,178,000	12.8%	of TDC
(81) Seeding, Hydro or air with mulch and fertilizer	AC	50	$2,615.00	$130,750	0.8%	of TDC
Cap Drain System Subtotal				***$1,839,949***	*10.8%*	*of TDC*

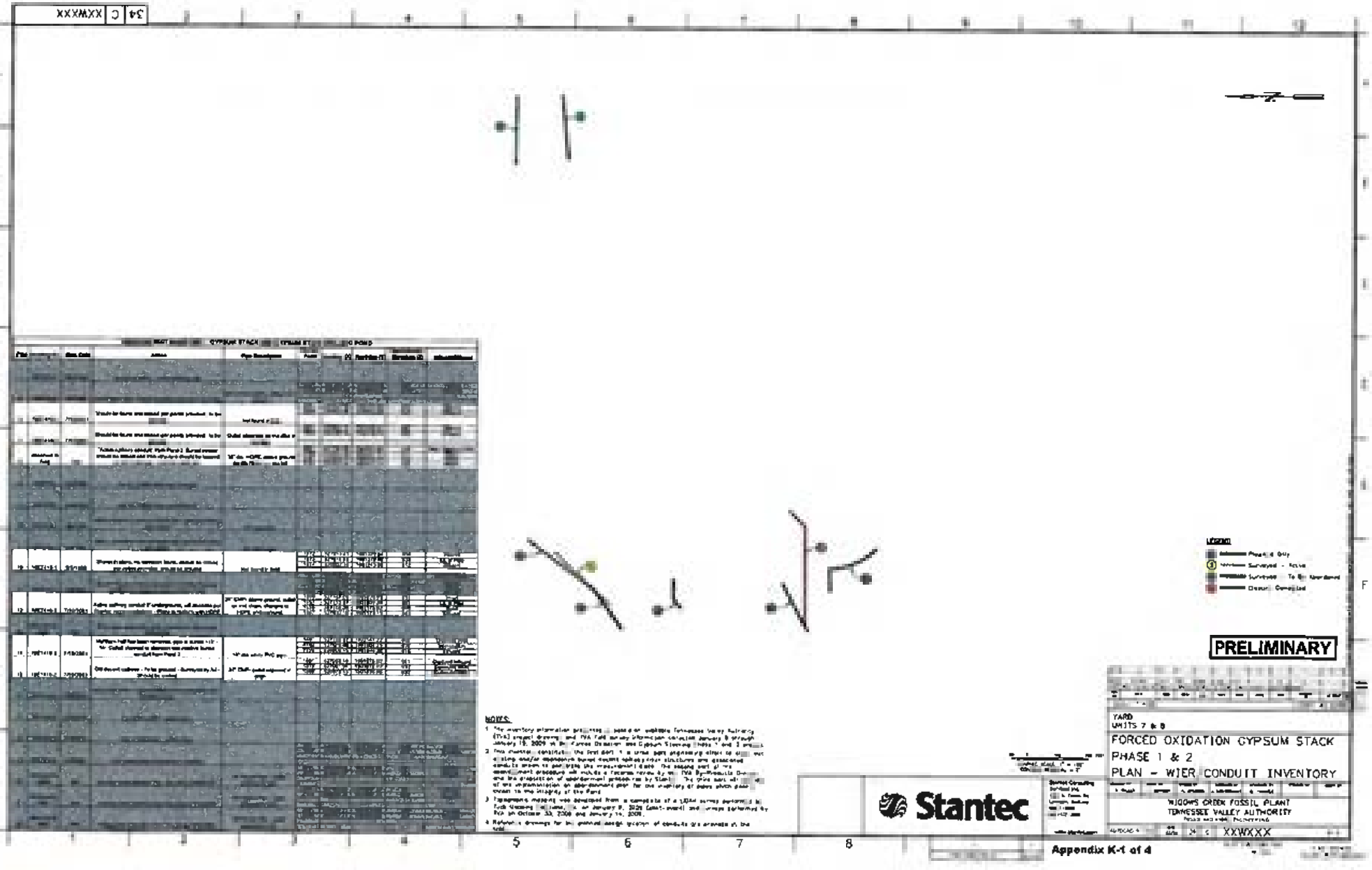
Work Plan 14 - Closure Plan
Widows Creek Fossil Plant, Stevenson, Alabama
Stantec Project No. 175569039

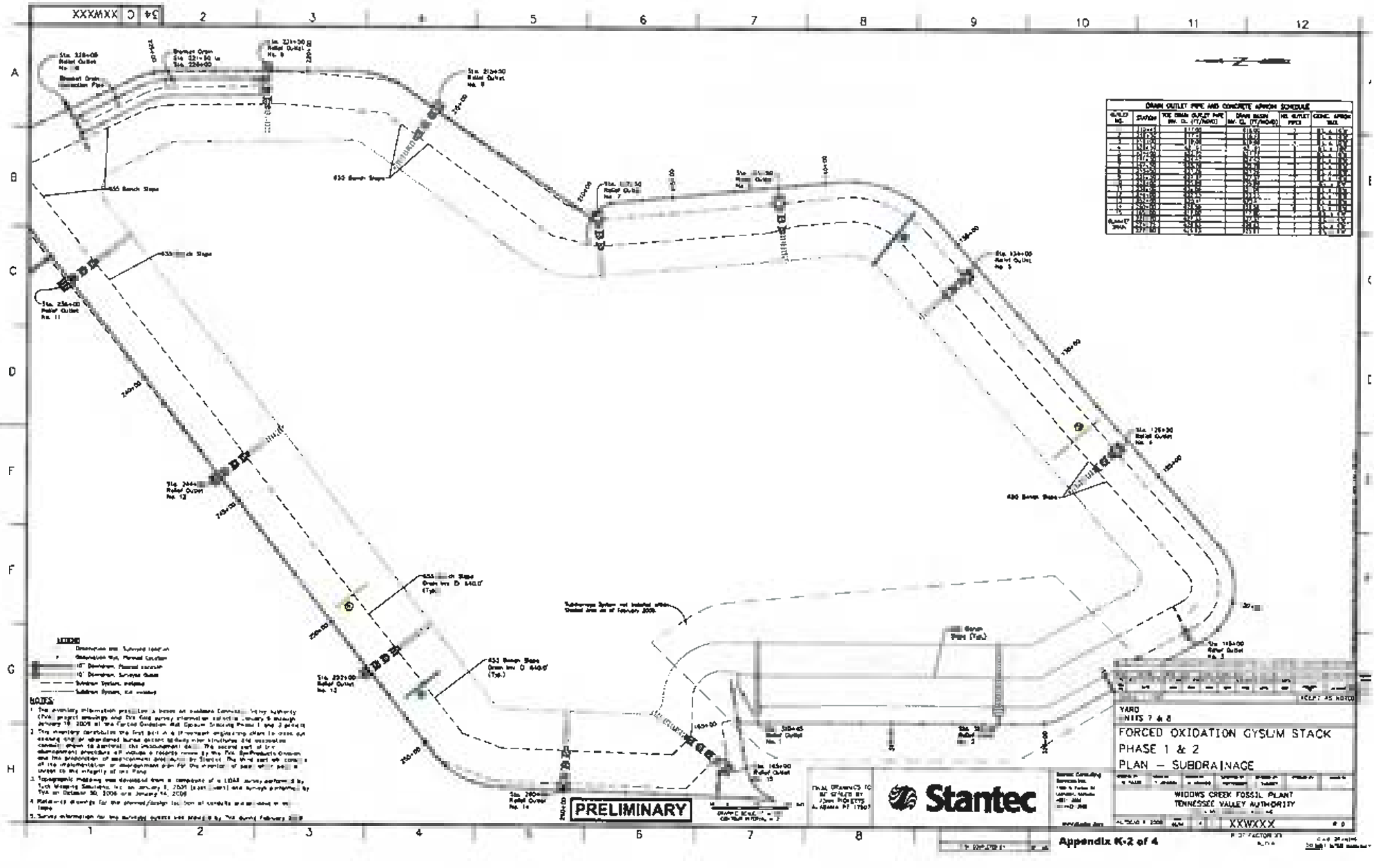
Feasibility Design Phase - Construction Cost Estimate

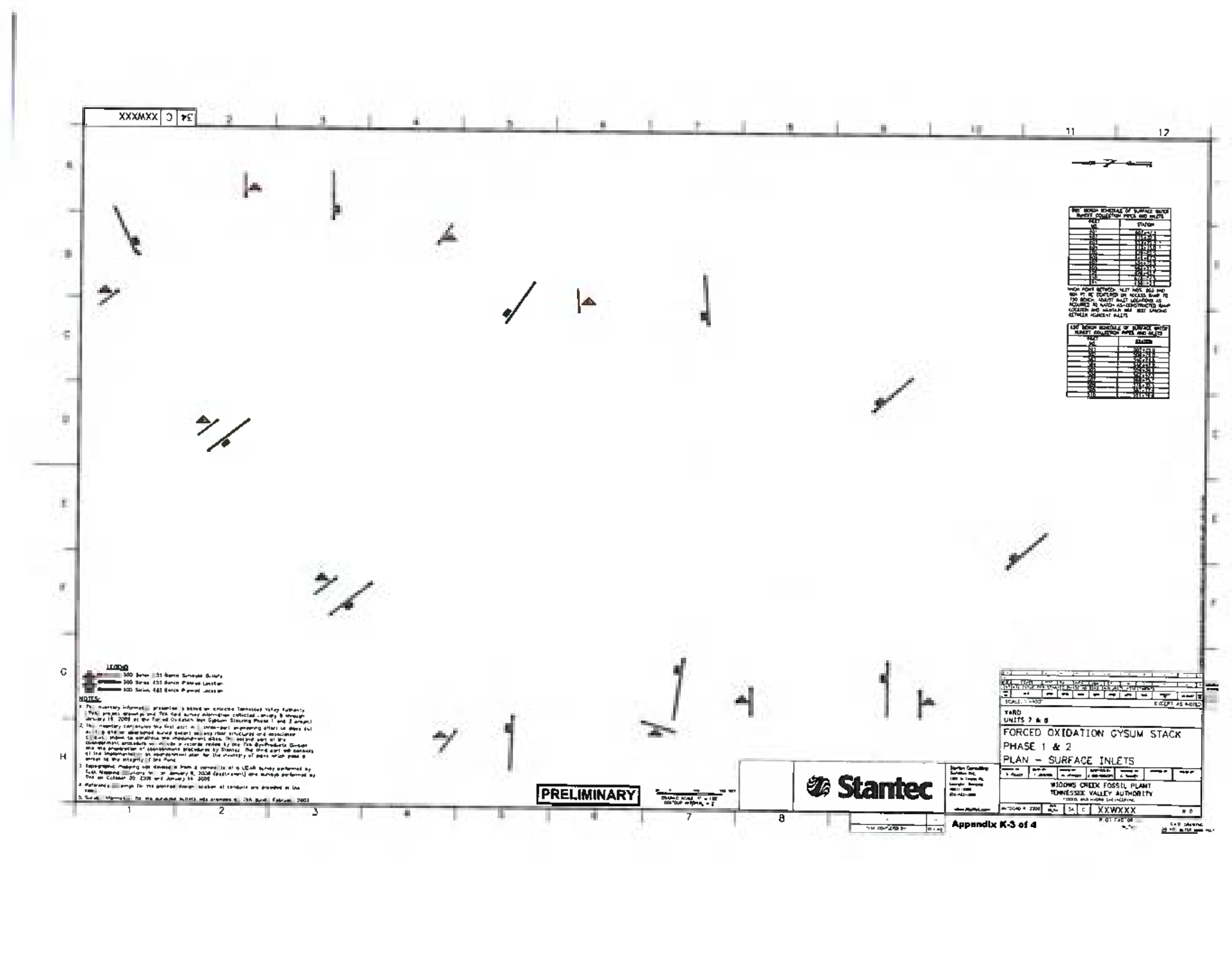
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Project Name:	Closure Plan		Work Plan 14		
Proj. Location:	Stevenson, Alabama		175569039	*Prep. By:*	ZCM
Estimate Date:	23-Oct-09		Construction	*Currency:*	USD

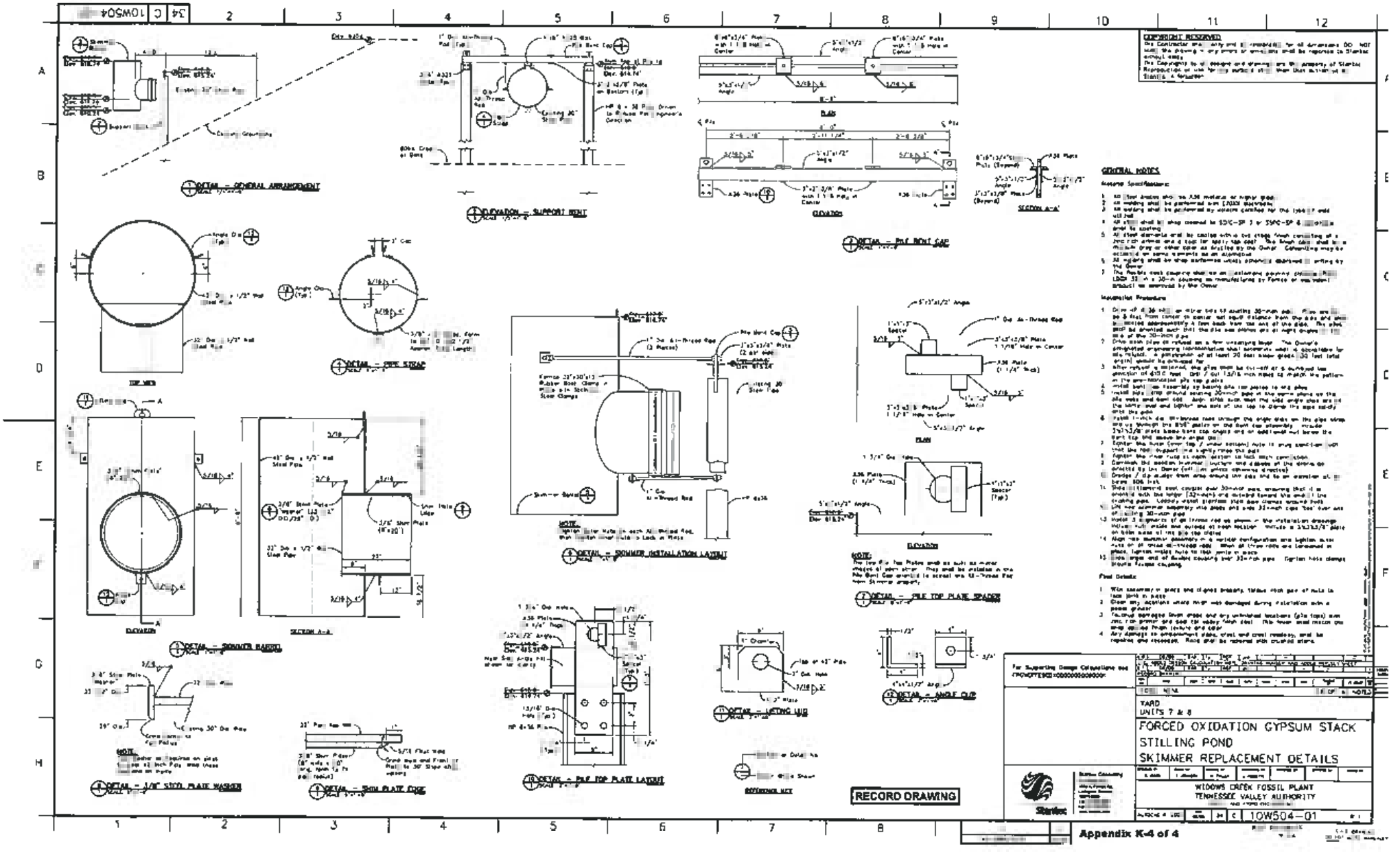
Item	Unit	Quantity	Unit Cost	Total Cost	Percentages	
Total Direct Field Costs				**$17,051,251**	100%	
				(TDC)		
Indirect Field Costs						
(82) Overtime Premium (5% TDL)				$170,513	5.0%	of TDC
(83) Miscellaneous Material (5% of TDM)				$511,538	5.0%	of TDM
(84) Freight (3% TDM)				$306,923	3.0%	of TDM
85 Sales Tax (6% TDM				$613,845	6.0%	of TDM
86 Special Equipment Rental (2% TDM				$204,615	2.0%	of TDE
(87) Material Escalation (8% TDM)				$818,460	8.0%	of TDM
(88) Labor Escalation				$102,308	3.0%	of TDL
Other Costs				**$2,728,200**	16.0%	of TDC
Total Field Costs				**19,779,452**	76.1%	of TIC
				(TFC)		
89 Engineering Design				1,500,000	5.8%	of TIC
(90) Engineering & Construction Management				800,000	3.1%	of TIC
Total Non-Field Costs				**2,300,000**	8.9%	of TIC
(91) 15% Contingency				**3,900,000**	15.0%	of TIC
Project Total Cost				**25,979,452**	152.4%	of TDC
						(TIC)

Assumptions
1) Unit costs include labor, material and equipment.
2) Excavated material is assumed to be non hazardous and doesn't require hazardous disposal.
3) Labor accounts for 20% of the total direct costs
4) Material accounts for 60% of the total direct costs
5) Equipment accounts for 20 % of the total direct costs
6) All quantities based on August 2009 conditions and proposed recommendations
7) Unit prices based on 2009 RSMeans Heavy Construction Cost Data

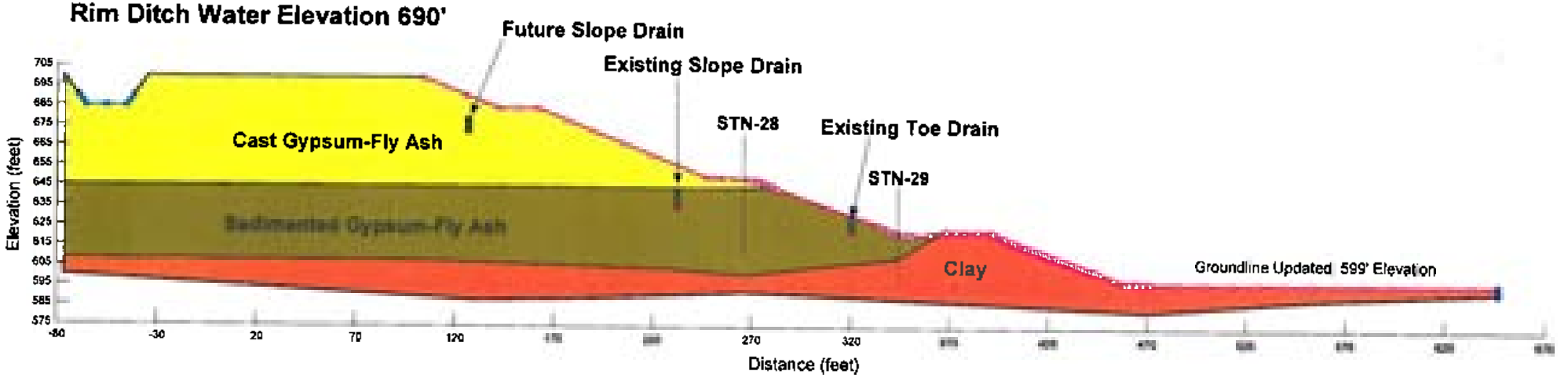

PRELIMINARY
YARD
UNITS 7 & 8
FORCED OXIDATION GYPSUM STACK
PHASE 1 & 2
PLAN - WIER CONDUIT INVENTORY
WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
Stantec
XXWXXX
Appendix K-1 of 4


DRAIN OUTLET PIPE AND CONCRETE APRON SCHEDULE
PRELIMINARY
Stantec
YARD
UNITS 7 & 8
FORCED OXIDATION GYSUM STACK
PHASE 1 & 2
PLAN – SUBDRAINAGE
WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
XXWXXX
NOTES:

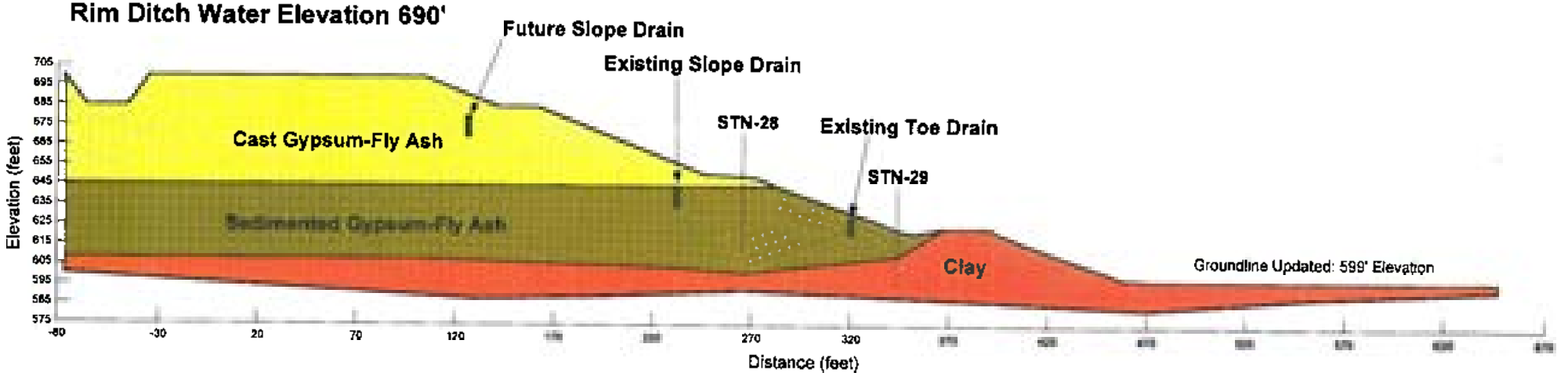

PRELIMINARY
Stantec
FORCED OXIDATION GYSUM STACK
PHASE 1 & 2
PLAN - SURFACE INLETS
WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
YARD
UNITS 7 & 8
XXWXXX
Appendix K-3 of 4


DETAIL – GENERAL ARRANGEMENT
ELEVATION – SUPPORT BENT
PLAN
ELEVATION
SECTION A-A
DETAIL – PILE BENT CAP
TOP VIEW
DETAIL – PIPE STRAP
DETAIL – SKIMMER INSTALLATION LAYOUT
PLAN
ELEVATION
DETAIL – PILE TOP PLATE SPACER
ELEVATION
SECTION A-A
DETAIL – SKIMMER BARREL
DETAIL – 3/8" STEEL PLATE WASHER
DETAIL – SKIM PLATE EDGE
DETAIL – PILE TOP PLATE LAYOUT
DETAIL – LIFTING LUG
REFERENCE KEY
DETAIL – ANGLE CLIP
GENERAL NOTES
RECORD DRAWING
YARD
UNITS 7 & 8
FORCED OXIDATION GYPSUM STACK
STILLING POND
SKIMMER REPLACEMENT DETAILS
WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
10W504-01
Appendix K-4 of 4

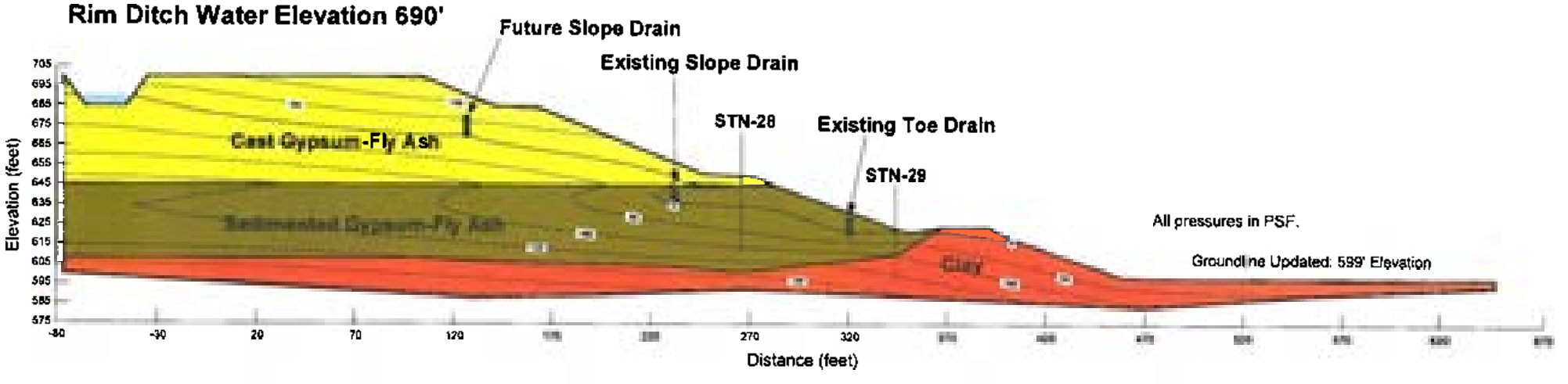
Widows Creek Fossil Plant
Gypsum Stack - Section A - 5 Year Buildout with Drains
Subsurface Profile and Boundary Conditions



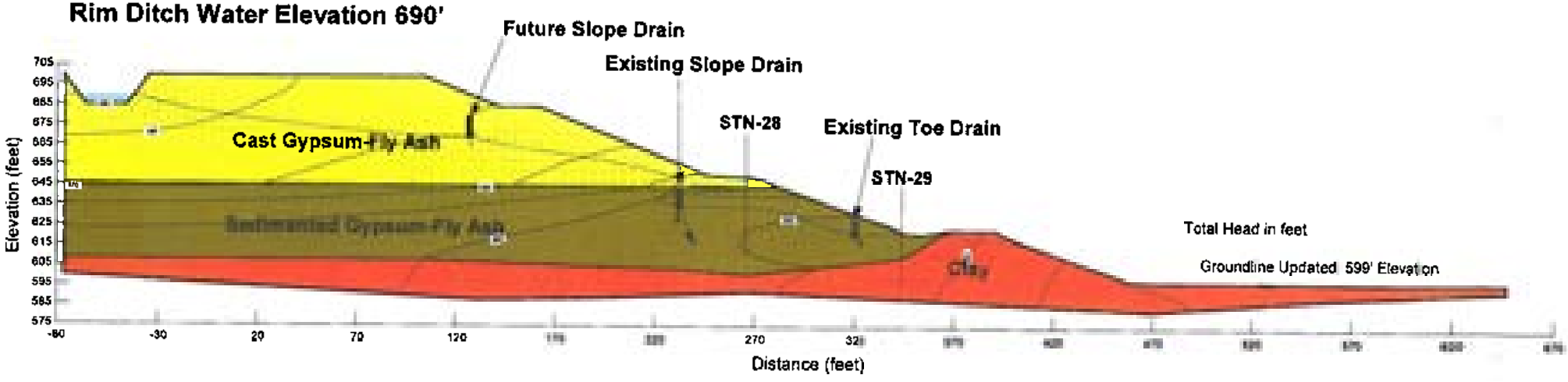
Widows Creek Fossil Plant
Gypsum Stack - Section A - 5 Year Buildout with Drains
Finite Element Mesh



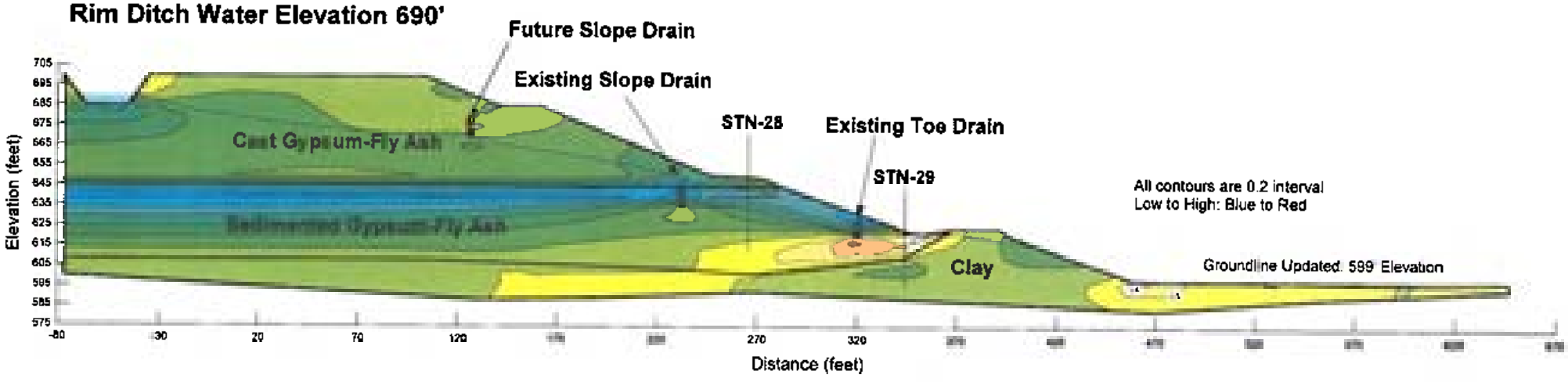
Widows Creek Fossil Plant
Gypsum Stack - Section A - 5 Year Buildout with Drains
Pore-Water Pressure Contours



Widows Creek Fossil Plant
Gypsum Stack - Section A - 5 Year Buildout with Drains
Total Head Contours and Flow Vectors

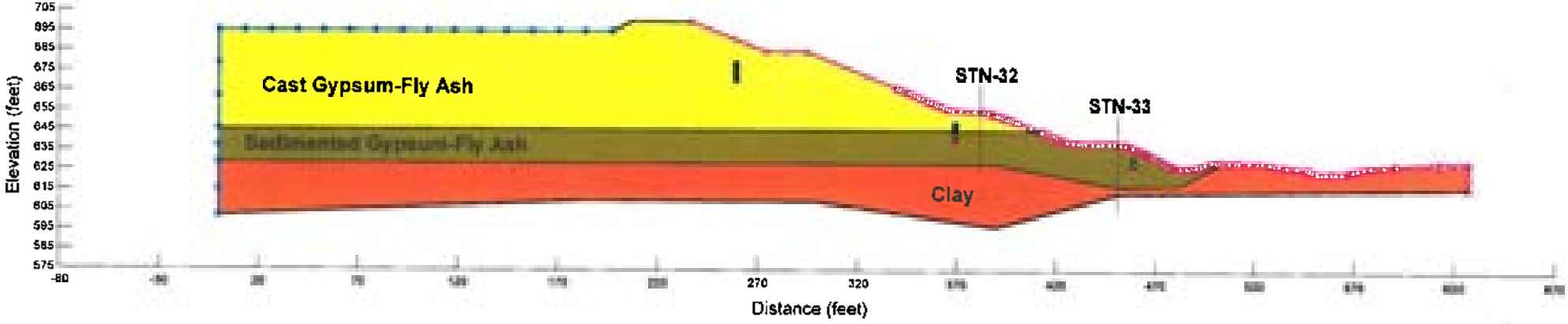


Widows Creek Fossil Plant
Gypsum Stack - Section A - 5 Year Buildout with Drains
Vertical Gradient Contours



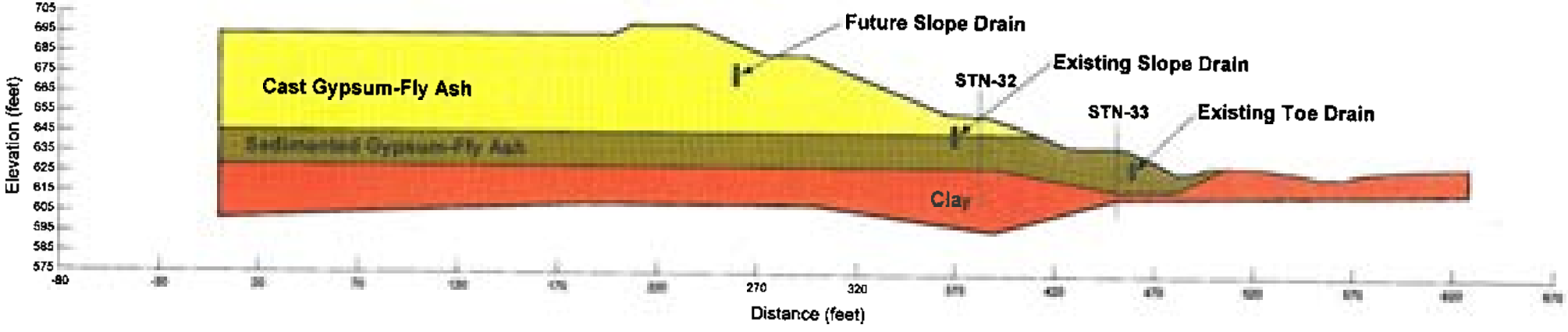
Widows Creeek Fossil Plant
Gypsum Stack - Section D - 5 Year Buildout with Drains
Subsurface Profile and Boundary Conditions

Pond 2A Normal Pool 695'



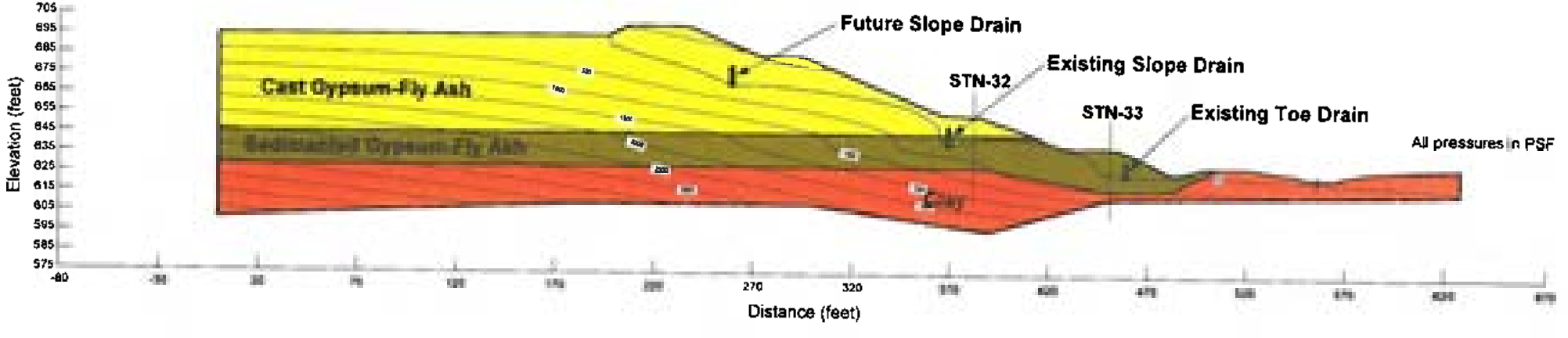
Widows Creeek Fossil Plant
Gypsum Stack - Section D - 5 Year Buildout with Drains
Finite Element Mesh

Pond 2A Normal Pool 695'



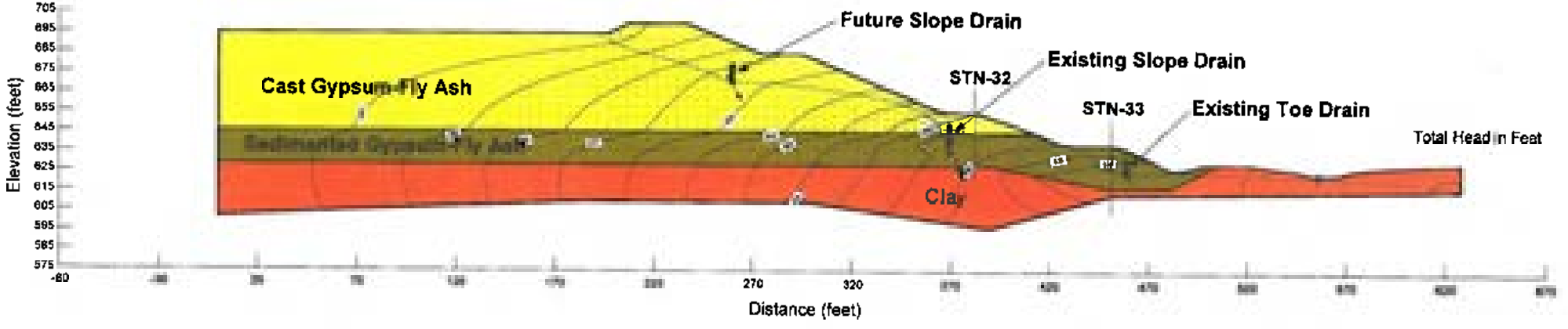
Widows Creeek Fossil Plant
Gypsum Stack - Section D - 5 Year Buildout with Drains
Pore-Water Pressure Contours

Pond 2A Normal Pool 695'



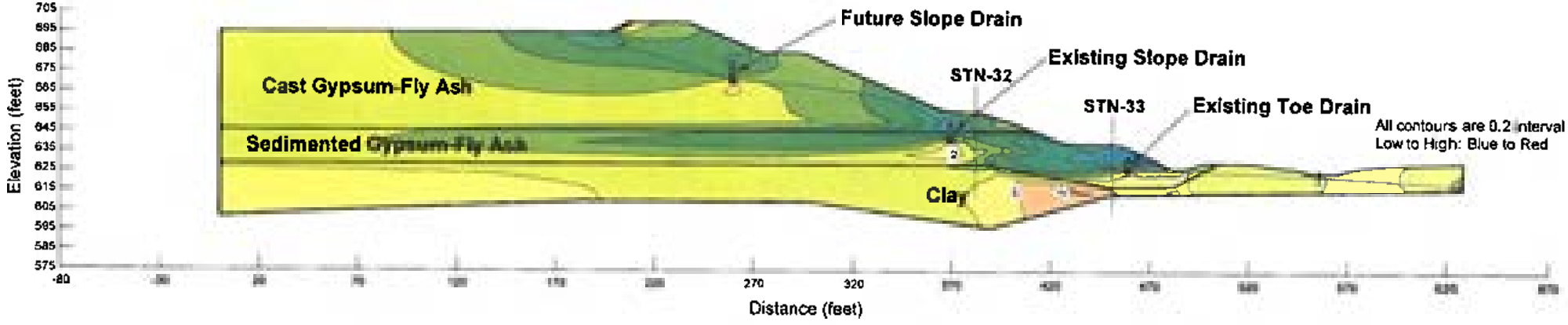
Widows Creeek Fossil Plant
Gypsum Stack - Section D - 5 Year Buildout with Drains
Total Head Contours and Flow Vectors

Pond 2A Normal Pool 695'

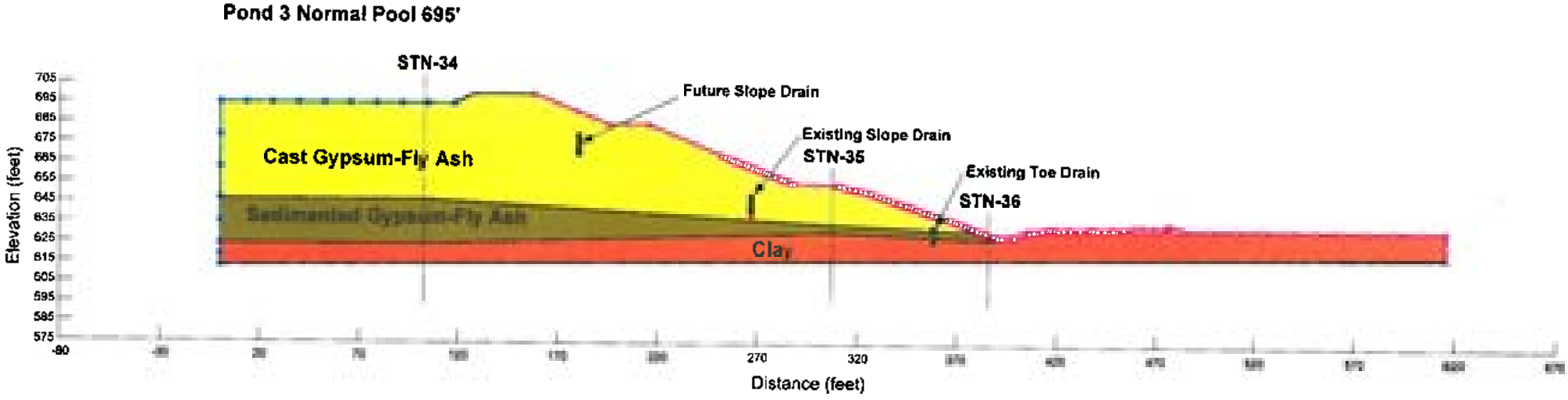


Widows Creeek Fossil Plant
Gypsum Stack - Section D - 5 Year Buildout with Drains
Vertical Gradient Contours

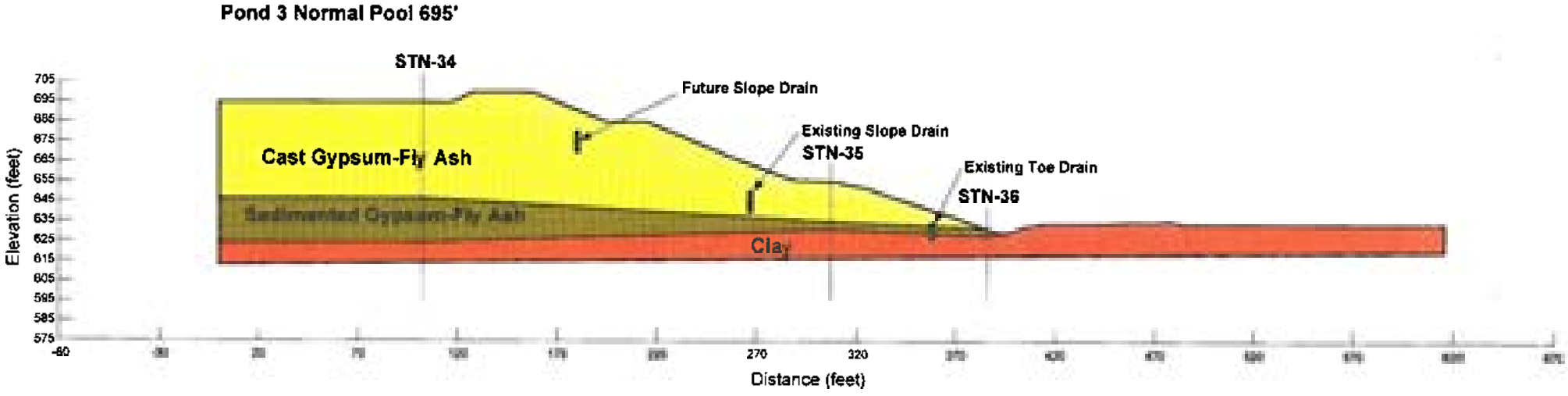
Pond 2A Normal Pool 695'



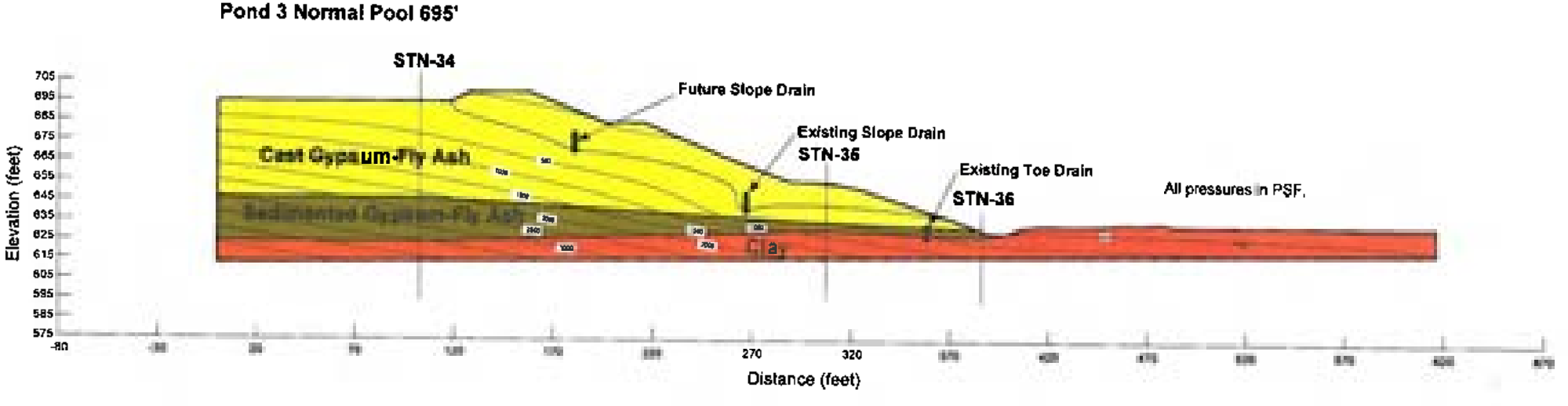
Widows Creeek Fossil Plant
Gypsum Stack - Section F - 5 Year Buildout with Drains
Subsurface Profile and Boundary Conditions



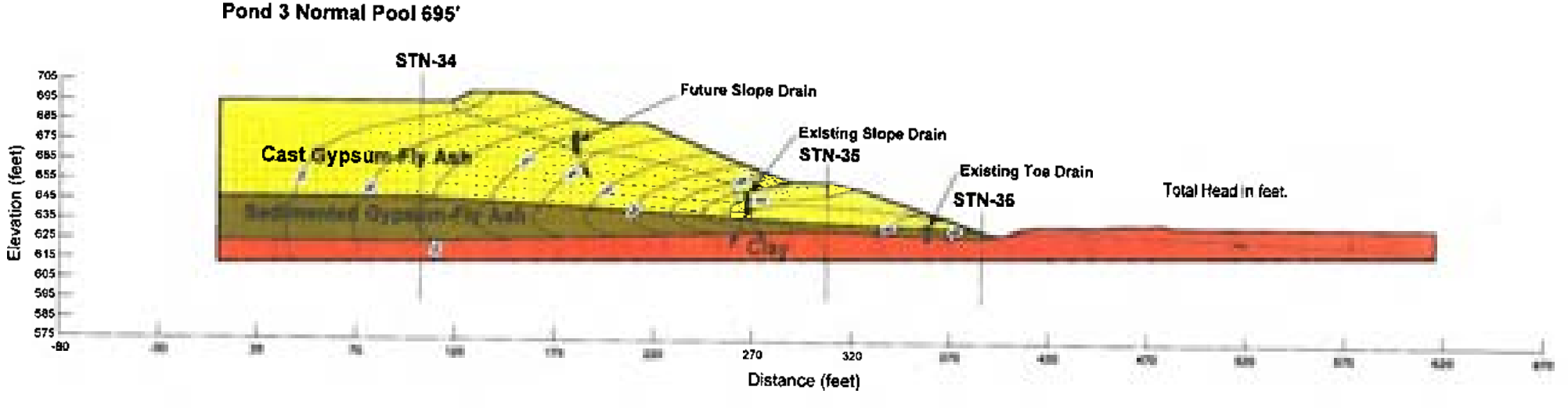
Widows Creeek Fossil Plant
Gypsum Stack - Section F - 5 Year Buildout with Drains
Finite Element Mesh



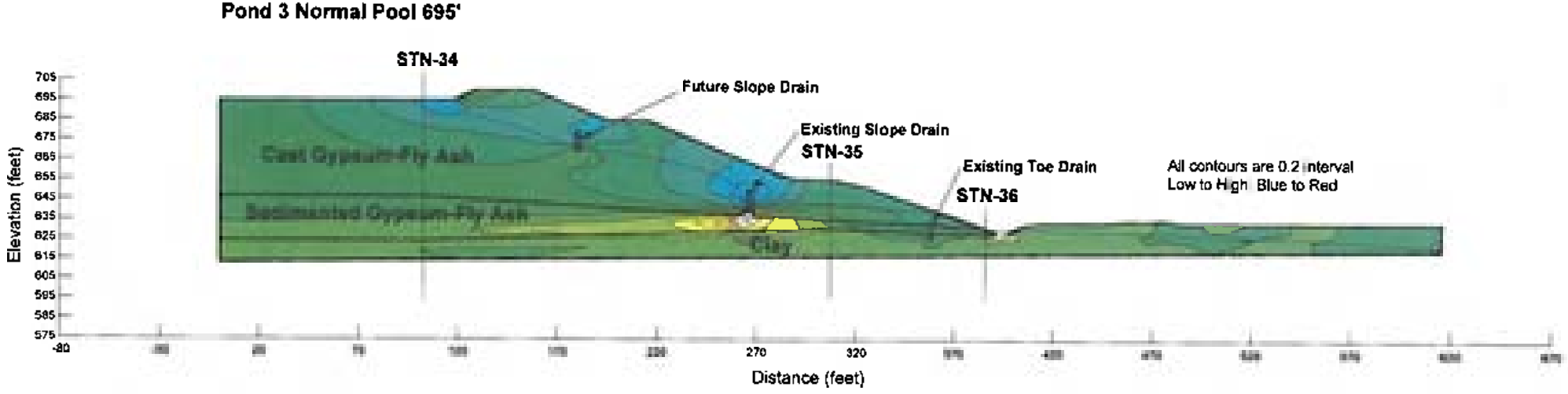
Widows Creeek Fossil Plant
Gypsum Stack - Section F - 5 Year Buildout with Drains
Pore-Water Pressure Contours



Widows Creeek Fossil Plant
Gypsum Stack - Section F - 5 Year Buildout with Drains
Total Head Contours and Flow Vectors



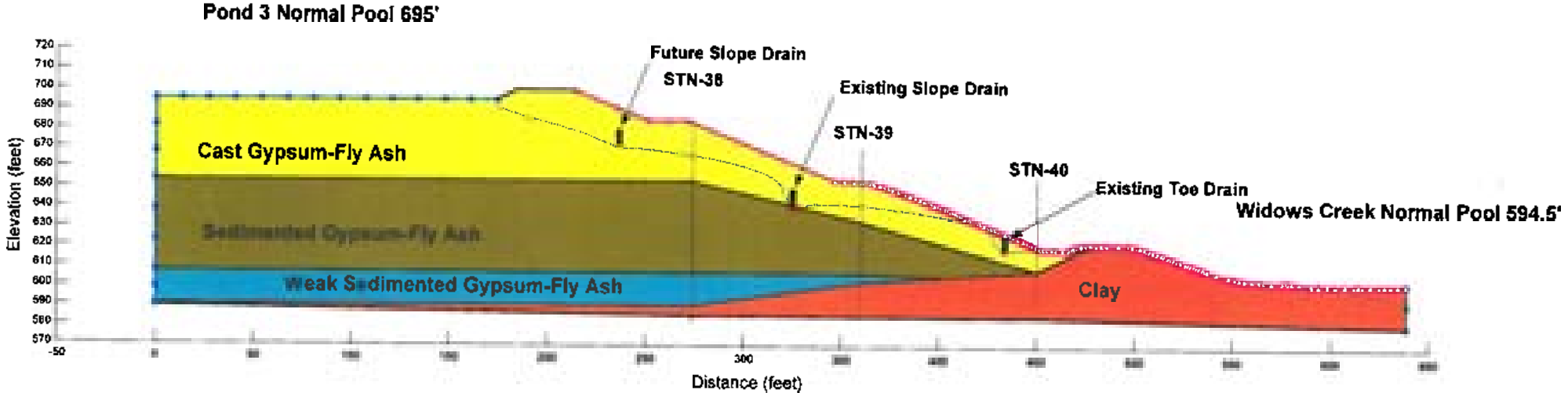
Widows Creeek Fossil Plant
Gypsum Stack - Section F - 5 Year Buildout with Drains
Vertical Gradient Contours



Widows Creeek Fossil Plant

Gypsum Stack - Section H - 5 Year Buildout with Drains

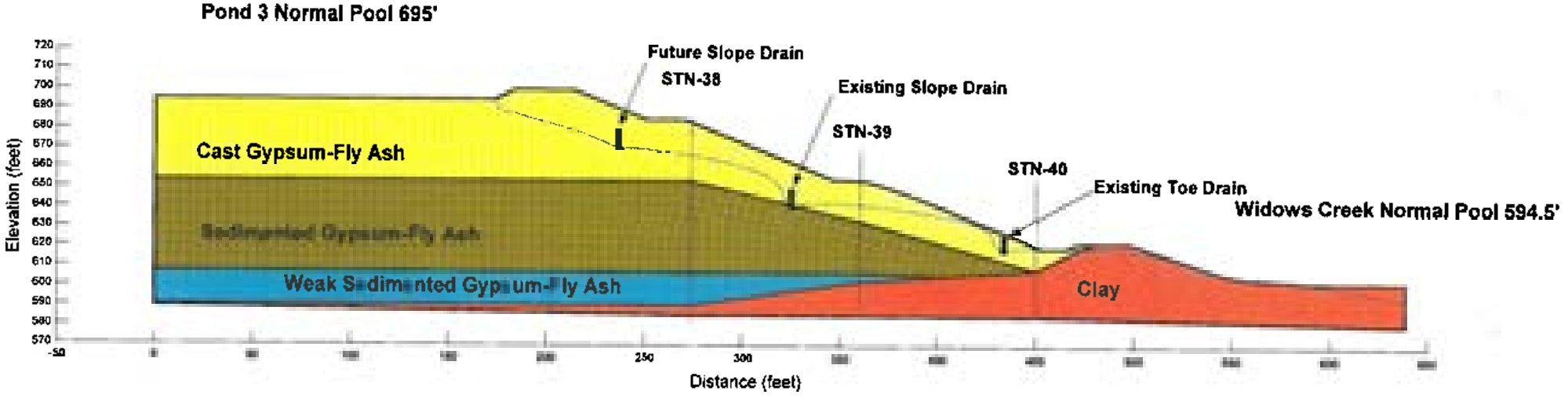
Subsurface Profile and Boundary Conditions



Widows Creeek Fossil Plant

Gypsum Stack - Section H - 5 Year Buildout with Drains

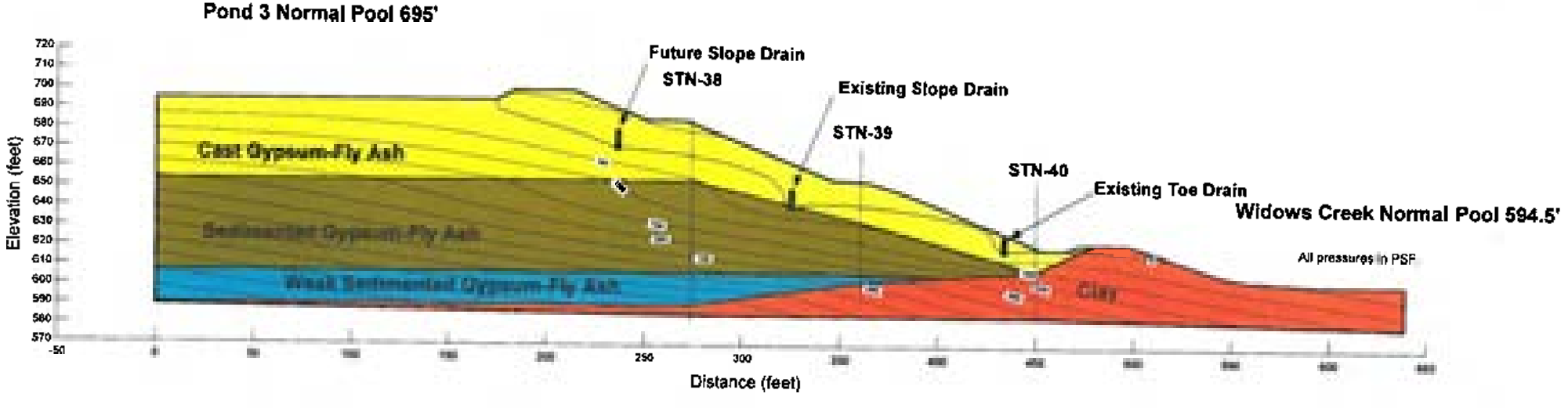
Finite Element Mesh



Widows Creeek Fossil Plant

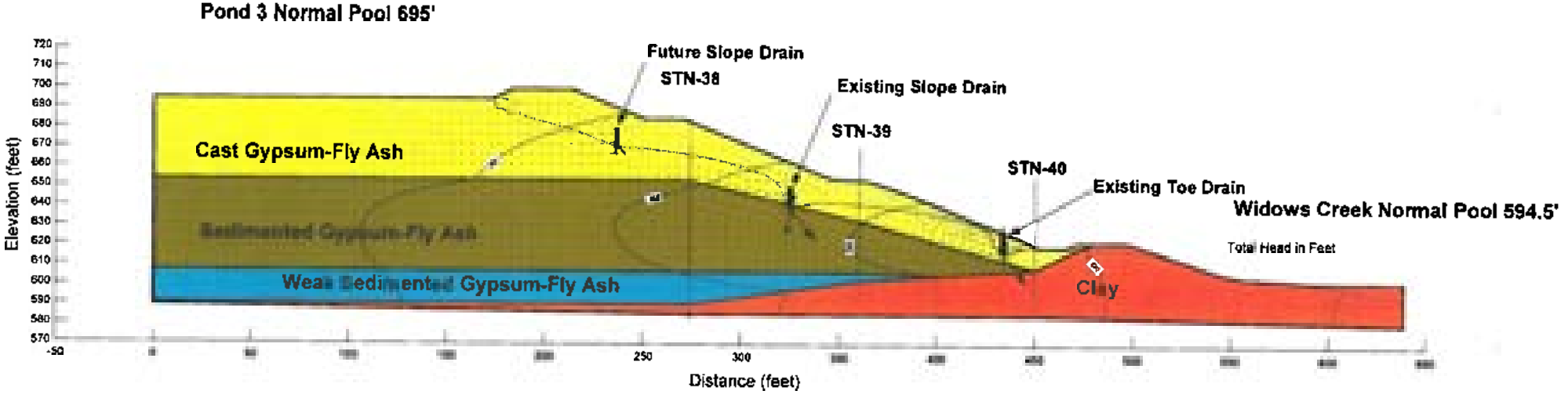
Gypsum Stack - Section H - 5 Year Buildout with Drains

Pore-Water Pressure Contours



Widows Creeek Fossil Plant

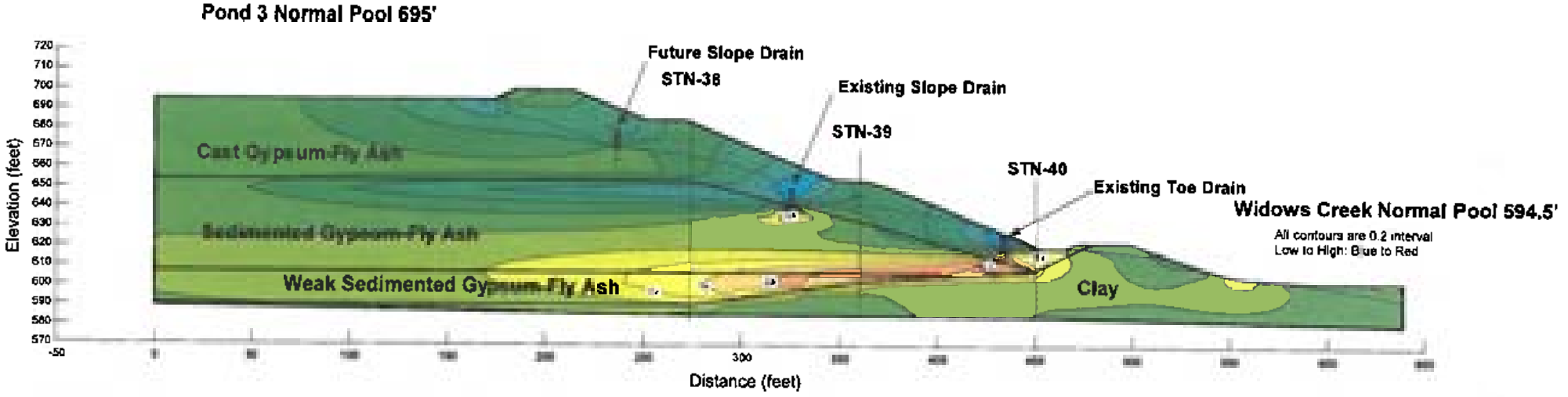
Gypsum Stack - Section H - 5 Year Buildout with Drains
Total Head Contours and Flow Vectors



Widows Creeek Fossil Plant

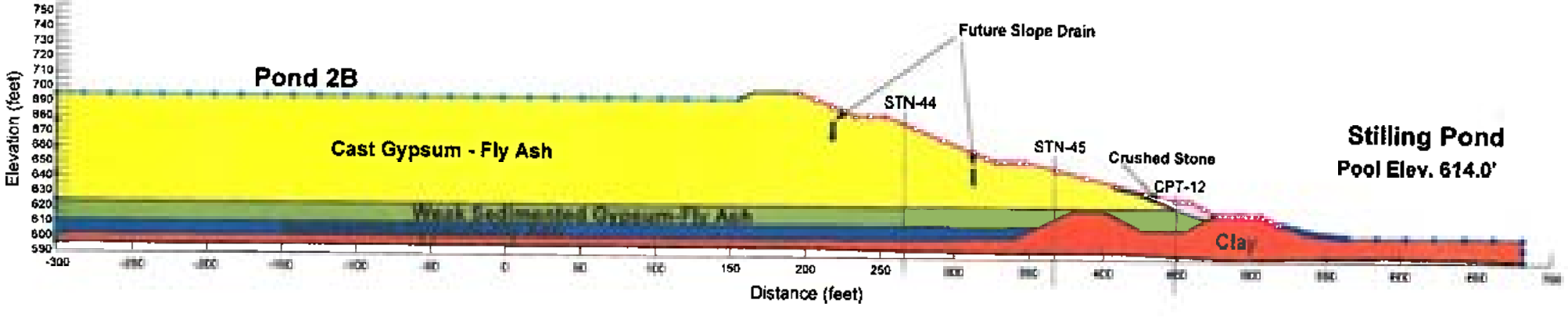
Gypsum Stack - Section H - 5 Year Buildout with Drains

Vertical Gradient Contours



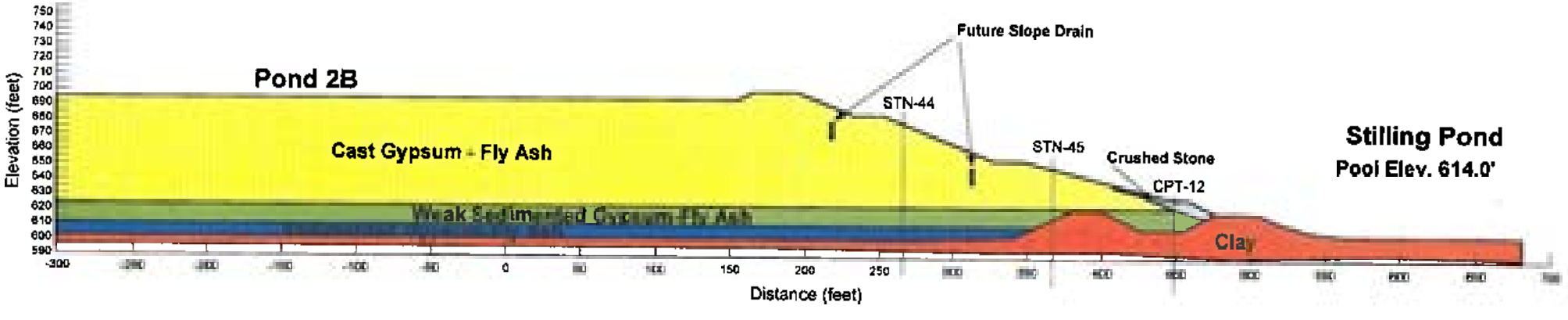
Gypsum Stack - Section K
5 Year Buildout with Drains
Subsurface Profile and Boundary Conditions

Normal Pool Operation 695'



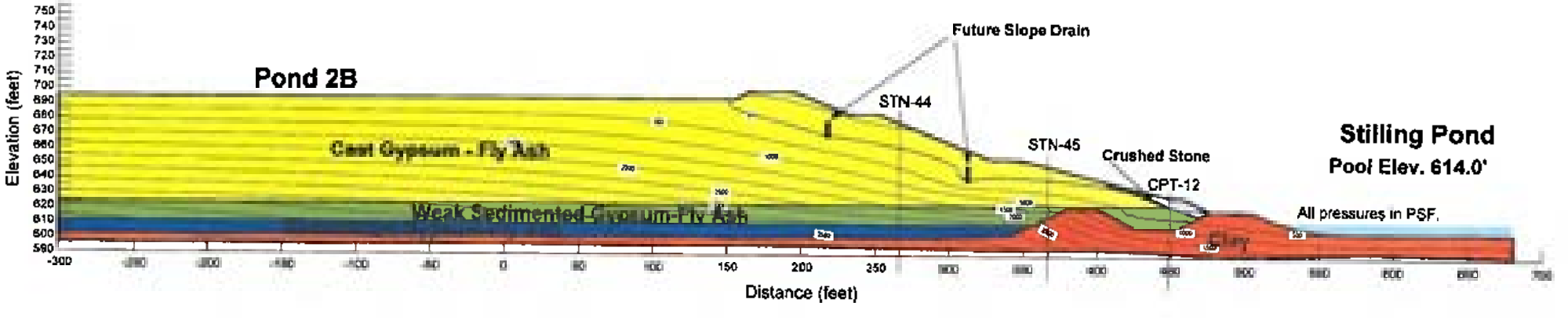
Gypsum Stack - Section K
5 Year Buildout with Drains
Finite Element Mesh

Normal Pool Operation 695'



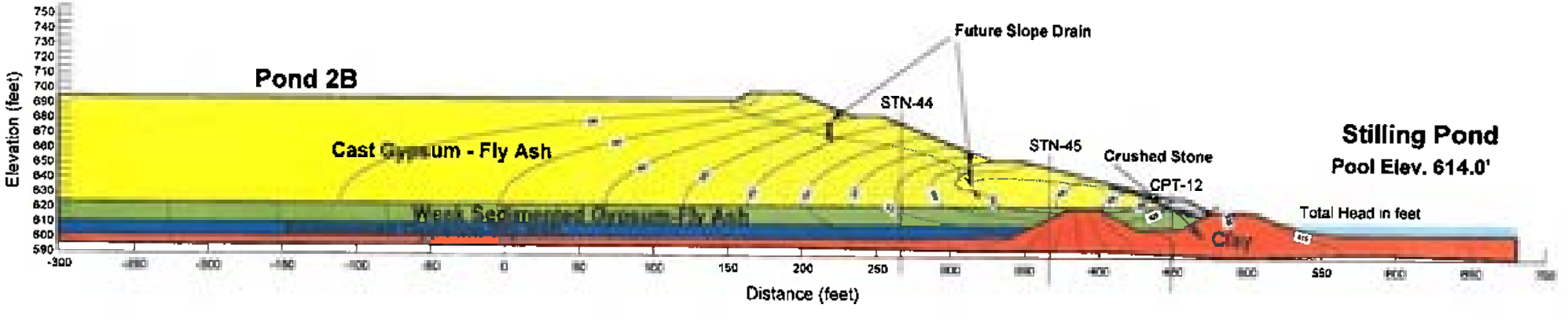
Gypsum Stack - Section K
5 Year Buildout with Drains
Pore-Water Pressure Contours

Normal Pool Operation 695'



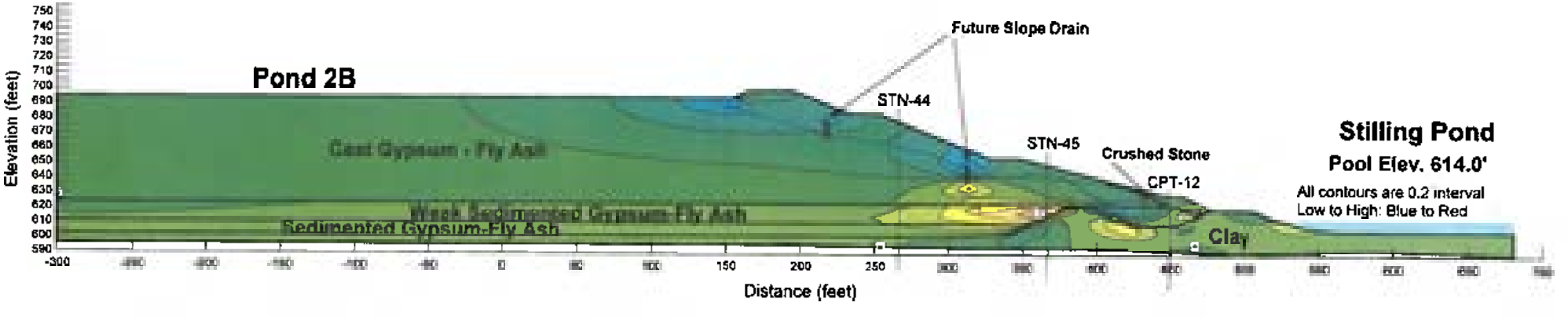
Gypsum Stack - Section K
5 Year Buildout with Drains
Total Head Contours and Flow Vectors

Normal Pool Operation 695'



Gypsum Stack - Section K
5 Year Buildout with Drains
Vertical Gradient Contours

Normal Pool Operation 695'



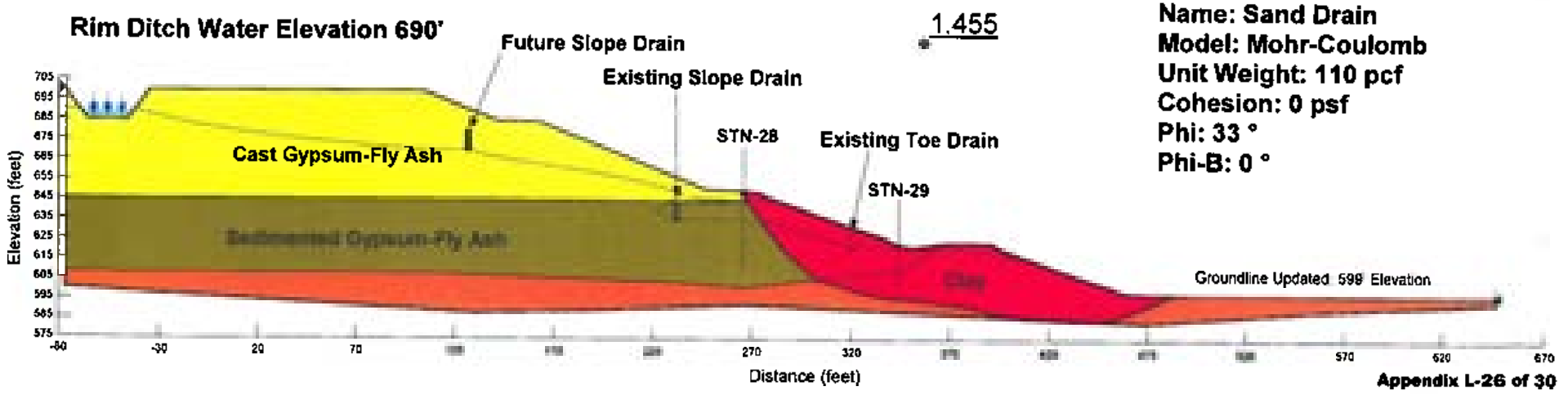
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Model: Mohr-Coulomb
Unit Weight: 113 pcf
Cohesion: 0 psf
Phi: 40 °
Phi-B: 0 °

Name: Sedimented Gypsum-Flyash
Model: Mohr-Coulomb
Unit Weight: 112 pcf
Cohesion: 0 psf
Phi: 41 °
Phi-B: 0 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion: 0 psf
Phi: 25 °
Phi-B: 0 °

Name: Sand Drain
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °

Widows Creek Fossil Plant
Gypsum Stack - Section A - 5 Year Buildout with Drains
Optimized Failure



Name: Cast Gypsum-Flyash
Model: Mohr-Coulomb
Unit Weight: 113 pcf
Cohesion: 0 psf
Phi: 40 °
Phi-B: 0 °

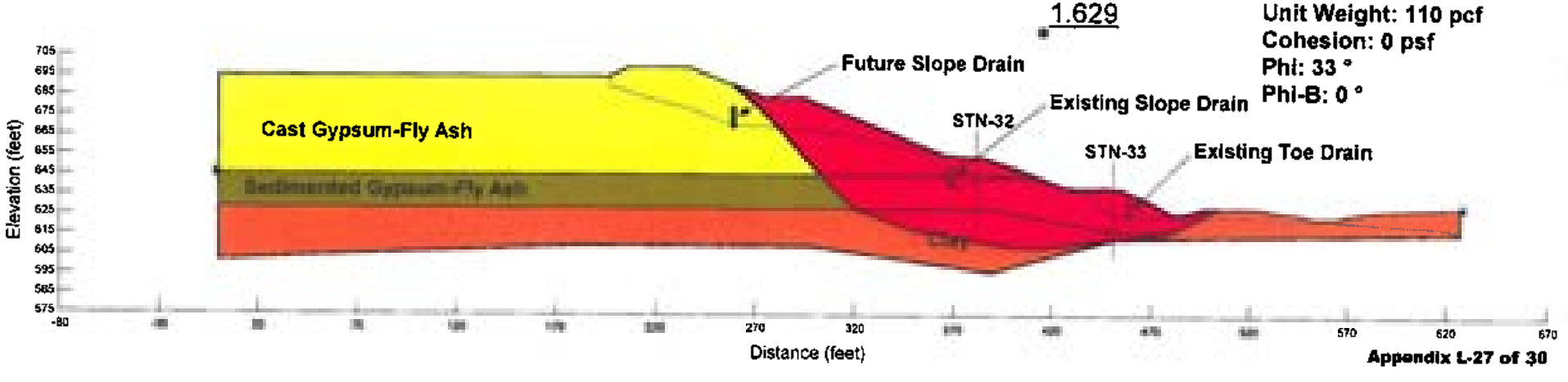
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Model: Mohr-Coulomb
Unit Weight: 112 pcf
Cohesion: 0 psf
Phi: 41 °
Phi-B: 0 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion: 0 psf
Phi: 25 °
Phi-B: 0 °

Name: Sand Drain
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °

Widows Creeek Fossil Plant
Gypsum Stack - Section D - 5 Year Buildout with Drains
Optimized Failure

Pond 2A Normal Pool 695'



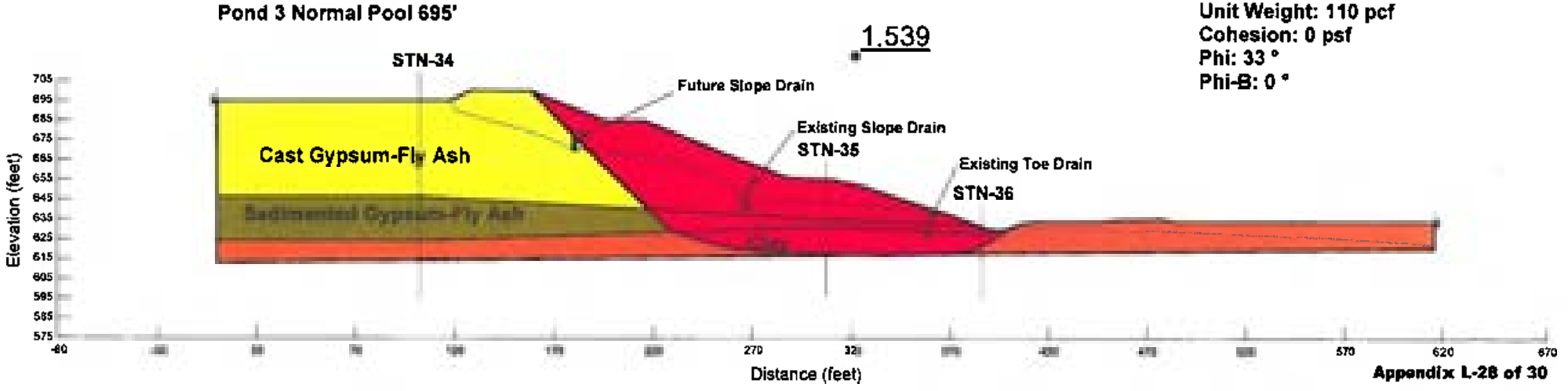
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Model: Mohr-Coulomb
Unit Weight: 113 pcf
Cohesion: 0 psf
Phi: 40 °
Phi-B: 0 °

Name: Sedimented Gypsum-Flyash
Model: Mohr-Coulomb
Unit Weight: 112 pcf
Cohesion: 0 psf
Phi: 41 °
Phi-B: 0 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion: 0 psf
Phi: 25 °
Phi-B: 0 °

Name: Sand Drain
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °

Widows Creeek Fossil Plant
Gypsum Stack - Section F - 5 Year Buildout with Drains
Optimized Failure



Name: Cast Gypsum-Flyash
Model: Mohr-Coulomb
Unit Weight: 113 pcf
Cohesion: 0 psf
Phi: 40 °
Phi-B: 0 °

Name: Sedimented Gypsum-Flyash
Model: Mohr-Coulomb
Unit Weight: 112 pcf
Cohesion: 0 psf
Phi: 41 °
Phi-B: 0 °

Name: Clay
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion: 0 psf
Phi: 25 °
Phi-B: 0 °

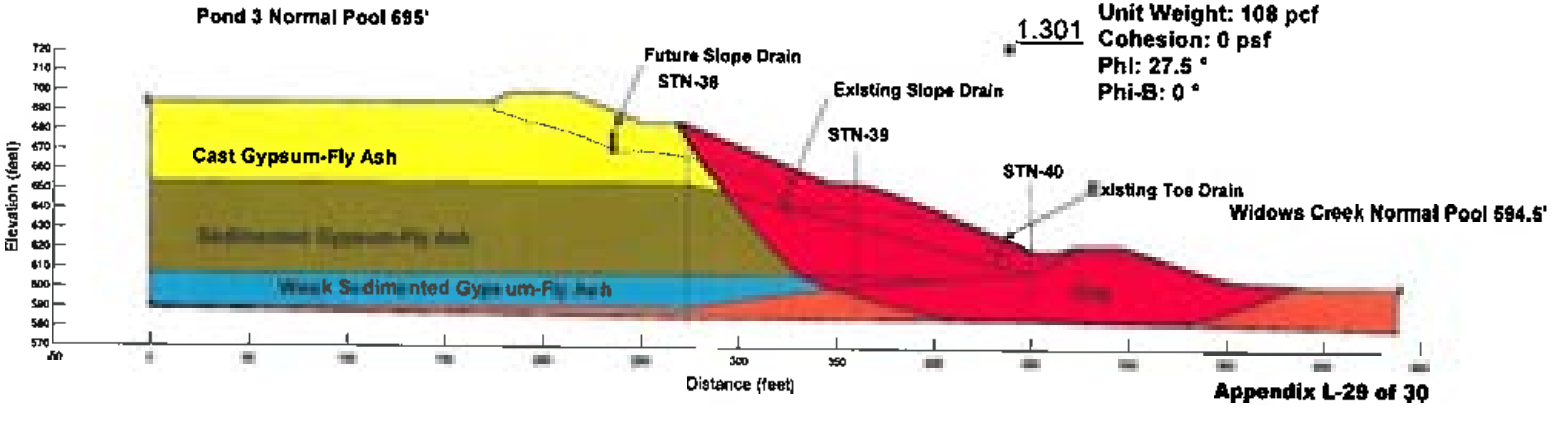
Name: Sand Drain
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °

Name: Weak Sedimented Gypsum-Flyash
Model: Mohr-Coulomb
Unit Weight: 108 pcf
Cohesion: 0 psf
Phi: 27.5 °
Phi-B: 0 °

Widows Creeek Fossil Plant

Gypsum Stack - Section H - 5 Year Buildout with Drains

Optimized Failure



Name: Cast Gypsum-Flyash
Model: Mohr-Coulomb
Unit Weight: 113 pcf
Cohesion: 0 psf
Phi: 40 °
Phi-B: 0 °

Name: Weak Sedimented Gypsum-Flyash
Model: Mohr-Coulomb
Unit Weight: 108 pcf
Cohesion: 0 psf
Phi: 27.5 °
Phi-B: 0 °

Name: Sedimented Gypsum-Flyash
Model: Mohr-Coulomb
Unit Weight: 112 pcf
Cohesion: 0 psf
Phi: 41 °
Phi-B: 0 °

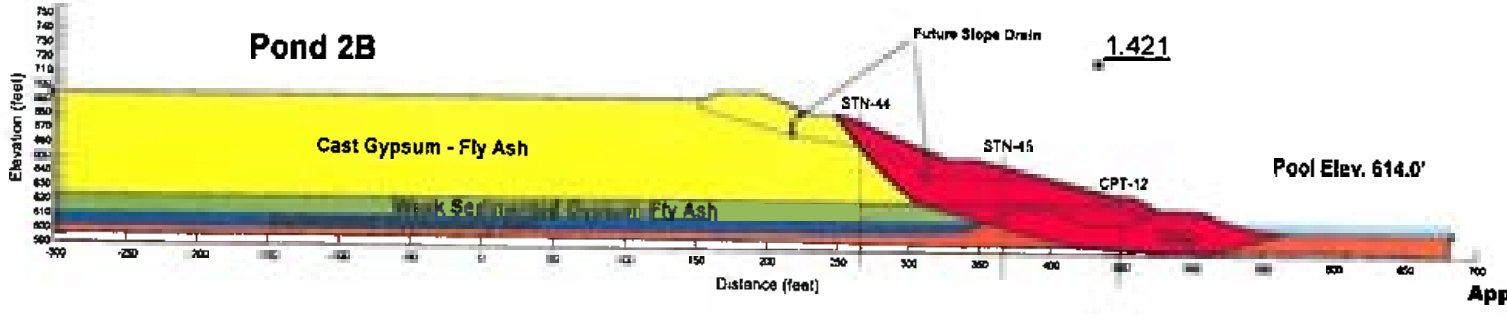
Name: Clay
Model: Mohr-Coulomb
Unit Weight: 123 pcf
Cohesion: 0 psf
Phi: 25 °
Phi-B: 0 °

Name: Class II Channel Lining
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 36 °
Phi-B: 0 °

Name: Sand Drain
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 0 psf
Phi: 33 °
Phi-B: 0 °

Gypsum Stack - Section K
5 Year Buildout with Drains
Optimized Failure

Normal Pool Operation 695'



US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 3

Breach Inundation Analysis, Widows Creek Fossil Plant, Jackson County, Alabama, Stantec Consulting Services, June, 2010

Stantec

Breach Inundation Analysis

Widows Creek Fossil Plant
Gypsum Stack
Jackson County, Alabama

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One Team. Infinite Solutions
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Cincinnati, Ohio 45241-20026
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Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

June, 2010

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Stantec

June 25, 2010
File: 175639024

Shawn K. Keef, PE
Program Manager - WCF
Tennessee Valley Authority – CCP Engineering
1101 Market Street, LP5E-C
Chattanooga, Tennessee 37402

Reference: Widows Creek Fossil Plant Breach Inundation Analysis

Dear Mr. Keef:

Enclosed is our report of the breach inundation analysis for the Widows Creek Facility gypsum stack. This report summarizes Stantec Consulting Services Inc.'s analysis methodologies and modeling results, including inundation mapping for the project area.

We appreciate the opportunity to assist Tennessee Valley Authority on this project. If you have any questions, please call our office.

Sincerely,

STANTEC CONSULTING SERVICES INC.

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Senior Project Engineer
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/lfb

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2.0 BREACH HYDROGRAPH DEVELOPMENT 2.2

3.0 HYDRAULIC MODEL DEVELOPMENT 3.5

4.0 RESULTS AND INUNDATION MAPPING 4.7

APPENDIX - BREACH INUNDATION DRAWINGS

1.0 Study Description

The Widows Creek Fossil Plant is located at the confluence of Widows Creek and the Tennessee River in Jackson County, Alabama. The existing gypsum stack has a footprint of approximately 120 acres and stands, from toe to top of dike, approximately 75 feet tall.

Stantec has performed breach analyses of the existing gypsum stack at the Widows Creek Fossil Plant using approximate methods. The results of this study were summarized in the report titled, "Preliminary Dam Breach Approximate Limits of Impact – Methodology" and submitted to the TVA on July 24, 2009.

Stantec has been requested to perform a more detailed analysis to determine the limit of impact caused by breaches of the gypsum stack dike. The following report summarizes the additional study of the breach impacts using FLO-2D, a two-dimensional flood routing software with the capability to model mudflows.

2.0 Breach Hydrograph Development

Stantec performed breach analyses of the gypsum stack using the U.S. Army Corps of Engineers (USACE) HEC-HMS computer modeling software, version 3.3. Breach analyses at four locations were performed for three failure scenarios:

1. "Sunny-day", Existing Stack Conditions: Assumes a failure of the existing embankment as determined by topographic information provided by TVA with no inflow to the stack or flooding in the surrounding area.

2. "Sunny-day", 18-Month Built-out Stack Conditions: Assumes a failure of the embankment following the placement of 18-months of additional ash and gypsum (2,866 Tons/day) with no additional inflow to the stack or flooding in the surrounding area.

3. Probable Maximum Precipitation (PMP) Event, 18-Month Built-out Stack Conditions: Assumes a failure of the 18-month built-out embankment during the Inflow Design Flood or PMP event (36.3 inches). Since the impoundment was modeled without an outflow, the entire rainfall volume was added to the impoundment volume and then failure initiated. The temporal distribution of rainfall had no impact on failure volumes or results.

For each of the scenarios, it was assumed that the embankment was fully saturated prior to the failure event. The dike breaches were simulated on the northwest, northeast, southeast and southwest sides of the stack. These locations are presented on Figures 1-6 (Appendix) with green circles. Piping failure at the toe of the embankment was assumed for the failure. For a piping failure, HEC-HMS simulates a rectangular breach that begins at the dike toe and has a gradually increasing breach orifice height and width, until the dam crest is reached. Using HEC-HMS, gypsum stack release hydrographs for each of the breach locations and failure scenarios were developed.

The Kingston failure of December 2008 provided a baseline scenario for a breach of a gypsum stack structure. The model parameters and assumptions used in this breach analysis describing the embankment failure were based on the Kingston failure, which experienced a sixty minute time-to-breach and 35% of total impoundment release volume. The release volume of approximately 35% of the total impoundment was also found to be a reasonable assumption in a study documented in "Floods from Tailings Dam Failures" by M. Rico, G. Benito and A. Diez-Herroro (2008). In this study, tailings' volume stored at the time of failure was shown to have a good correlation ($r^2 = 0.86$) with the tailings outflow volume. For the PMP event, built-out stack conditions failure scenario, it was assumed that 35% of the impounded gypsum material and the entirety of the stored precipitation was released through the dike breaches. Tables 1 to

3 summarize the breach scenario input assumptions and the results from the HEC-HMS modeling for all four modeled breach locations and for each failure scenario, respectively.

Table 1. Breach Parameters and HEC-HMS Results – "Sunny-Day", Existing Stack Conditions

Breach Location	Time to Breach (min)	Top of Dike (feet)	Bottom of Dike (feet)	Breach Width (feet)	Breach Side Slope (H:V)	Total Release Volume (CY)	Peak Discharge Rate (cfs)
Northwest	60	680	612	200	1:1	3.6 million	78,100
Northeast	60	680	633	200	1:1	2.6 million	45,000
Southeast	60	680	626	200	1:1	3.1 million	55,500
Southwest	60	680	605	200	1:1	3.6 million	90,000

Table 2. Breach Parameters and HEC-HMS Results – "Sunny-Day", 18-Month Built-Out Stack Conditions

Breach Location	Time to Breach (min)	Top of Dike (feet)	Bottom of Dike (feet)	Breach Width (feet)	Breach Side Slope (H:V)	Total Release Volume (CY)	Peak Discharge Rate (cfs)
Northwest	60	680	612	200	1:1	4.0 million	86,600
Northeast	60	680	633	200	1:1	3.0 million	53,800
Southeast	60	680	626	200	1:1	3.5 million	65,100
Southwest	60	680	605	200	1:1	4.0 million	98,900

Table 3. Breach Parameters and HEC-HMS Results – PMP Event, 18-Month Built-Out Stack Conditions

Breach Location	Time to Breach (min)	Top of Dike (feet)	Bottom of Dike (feet)	Breach Width (feet)	Breach Side Slope (H:V)	Total Release Volume (CY)	Peak Discharge Rate (cfs)
Northwest	60	680	612	200	1:1	4.5 million	95,100
Northeast	60	680	633	200	1:1	3.5 million	62,500
Southeast	60	680	626	200	1:1	4.0 million	72,800
Southwest	60	680	605	200	1:1	4.5 million	107,700

3.0 Hydraulic Model Development

For the flood routing analysis, the FLO-2D Flood Routing Model, version 2009.06, was selected. FLO-2D is a two-dimensional volume conservation model that routes inflow hydrographs through channels or unconfined floodplain surfaces in eight flow directions. In addition, the FLO-2D software is capable of routing the flood volume as a mudflow, thus taking into account the physical properties of the impounded gypsum.

3.1 MODEL STABILITY

The FLO-2D software has parameters that are used to control the duration and model stability for each simulation run. The duration of the breach simulations was set to 24 hours. To compensate for decreased roughness values in shallow flow conditions, the global Shallow Flow n-value parameter was set to the model default of 0.25. During the process of improving the model stability, the Maximum Floodplain Froude Number was set to 0.9 to prevent floodplain surging. The surface detention, percent change in flow depth and dynamic wave stability coefficients were set to values specified in the FLO-2D user's manual: 0.1, 0.2 and 1.0, respectively.

3.2 MODEL GRID DEVELOPMENT

The elevation grid required for input into the FLO-2D model was developed using the U.S. Geological Survey (USGS) National Elevation Dataset (NED) Digital Elevation Model (DEM). For this study, the floodplain grid cell size was 100 feet. The grid was supplemented with 1-foot contour interval topographic data of the Widows Creek facility provided by the Tennessee Valley Authority to represent the dike geometry around the ash pond and gypsum stack. The floodplain elevations and roughness factors (Manning's "n" values) for the grid system were developed using the FLO-2D Grid Developer System (GDS). No channel features were developed in the grid system. South of the gypsum stack, the model grid extent was terminated at the Tennessee River floodplain. At these limits, the grid cells were designated as outflow cells to record the volume of outflow from the model system and preserve model continuity. At the State Highway 277 bridge crossing over Widows Creek, the model grid was supplemented with topographic data from a field survey performed by TVA personnel on April 30, 2010.

Manning's "n" values were initially estimated using the USGS National Land Cover Database (NLCD) 2001 dataset. During the process of improving model stability, "n" values were modified based on the 2006 imagery obtained from the U.S. Department of Agriculture (USDA). A detailed field reconnaissance of the floodplain was not performed. The range of "n" values used in this modeling effort was based on Chow (1959).

To model the release of the impounded material during the breach simulations, the results from the HEC-HMS modeling were used as inflow hydrographs at one of four cells along the outside of the gypsum stack dike.

3.3 HYPERCONCENTRATED SEDIMENT FLOW

To simulate the release of the impounded volumes as gypsum material, the mudflow, or hyperconcentrated sediment flow, functions of the FLO-2D software were utilized. The hyperconcentrated sediment flow parameters used in this analysis were obtained from "Laboratory Analysis of Mudflow Properties" by Jim S. O'Brien and Pierre Y. Julien (1988), which documents laboratory measurements of the rheological properties of natural mudflow deposits in Colorado. The clay composition and particle distribution information from the Stantec laboratory testing of the gypsum stack material was compared to the properties of mudflow deposits documented in the literature. Stantec determined that the analyzed gypsum stack material has similar properties to the 'Glenwood 2' mudflow sample documented in O'Brien 1988. Based on the laboratory results and the values from the literature, the following hyperconcentrated sediment flow parameters were utilized in the flood routing:

- Percent solids by volume for saturated conditions = 50% for the "sunny-day" failure scenarios; = 45% for the PMP event scenario
- Sediment specific gravity = 2.68
- Viscosity vs. Sediment Concentration Coefficient = 0.648 and Exponent = 6.20
- Yield Stress vs. Sediment Concentration Coefficient = 0.0765 and Exponent = 16.90

Due to the roughness and irregular geometry of the floodplain, the overland laminar flow resistance value used in these simulations (K = 50,000) was set to the upper limit of the recommended range of values specified in the FLO-2D user's manual.

3.4 TENNESSEE RIVER

For the flood routing analysis on the Tennessee River, the USACE Hydrologic Engineering Center River Analysis System (HEC-RAS) software, version 4.0, was selected. An unsteady HEC-RAS model was developed for the Tennessee River reach adjacent to the Widows Creek facility. The baseflow for the reach was based on average annual flow values obtained from USGS gage data. The inflow to the Tennessee River reach during the simulated dike breaches was developed from the model grid outflow hydrographs. This inflow was input into the HEC-RAS model as a lateral inflow hydrograph at river mile 409.

4.0 Results and Inundation Mapping

Inundation mapping was developed for each of the breach scenarios. The inundation limits were delineated using the hydraulic model outputs and the imagery and topographic data described in Section 3.2. Figures 1, 3 and 5 present the maximum extent of the flood limits after combining the impact of the four independent breach simulations. Figures 2, 4 and 6 present the maximum inundation depths for the impacted area based on the results of the four model simulations.

The figures depict eight (8) structures that are located within the impact zone, as well as several roadways. Five of the "at risk" structures are located on properties acquired by the TVA and have been demolished. The inundation of State Highway 277 was reviewed based on field survey data collected at the bridge crossing of Widows Creek. The modeled flood elevation did not exceed the roadway embankment elevation.

Appendix

Breach Inundation Drawings

Dike Breach Impact Zone - Sunny-Day, Existing Stack

Gypsum Stack
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama

Figure 1

Dike Breach Inundation Depth - Sunny-Day, Existing Stack

Gypsum Stack
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama

Stantec

Stantec Consulting Services Inc.
1397 Lebanon Rd.
Cincinnati, Ohio
45241-2012
513-842-8300
www.stantec.com

Figure 2

Dike Breach Impact Zone - Sunny-Day, 18-Month Built-Out Stack

Gypsum Stack
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama

Figure 3

Stantec

Stantec Consulting Services Inc.
(3887 Lakeman Rd.
Cincinnati, Ohio
45241-3011
513-842-8200
www.stantec.com

Dike Breach Inundation Depth - Sunny-Day, 1% Annual Event Flood

Gypsum Stack
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama

Stantec

Figure 4

Dike Breach Impact Zone - PMP Event, 18-Month Build-Out Stack

Gypsum Stack
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama

Stantec

Stantec Consulting Services Inc.
11687 Lebanon Rd
Cincinnati, Ohio
(513) 842-8200
www.stantec.com

Figure 5

Legend
FLO-2D Model Maximum Depth (FT)
0 - 3
3 - 6
6 - 9
9 - 12
12 - 15
15 - 18
18 - 21
> 21
Contours
At-Risk Structures
Breach Locations
0
600
1,200 Feet
Notes
1. 2006 Aerial Photography obtained from USDA Farm Service Agency (FSA).
2. Bathymetric data interpolated from USGS Quadrangle: USGS for Bridgeport, Dixon Cove, Flat Head and Stevenson, Alabama dated February 1990. Hydraulic modeling was supplemented by topographic data provided by TVA.
3. Flood Depths and elevations and limits of impact based upon FLO-2D unsteady flow simulations that included modeling of dam breach event.
4. Model cells were adjusted to the Tennessee River normal pool elevation of 634 feet above mean sea level (msl) to approximate flood depths and elevations from aerial photography and available topographic mapping.
5. Actual areas within limits of impact will depend on actual failure or flooding conditions and may differ from areas shown on map.
Dike Breach Inundation Depth - PMP Event, 10-Month Built-Out Stack
Gypsum Stack
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
Figure 6
Stantec
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US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 4

TVA Spreadsheet "INCR_VOL_DATA_RHOM_WCF," Dated September 28, 2011

	POND	Last Survey Date	Survey Date	Average Top of Dike Elevation	Elevation Low Top of Dike	Free Standing Water Volume	Low Elevation at Time of Survey	Freeboard	Watersurface Elevation	Watersurface Area
BULL RUN FOSSIL PLANT	MAIN ASH	2/22/2011	6/24/2011	813.1	806.9	805.4	789.7	5.8	801.1	529983.6
	STILLING	10/27/2010	6/24/2011	809.2	806.1	805.4	787.6	5.6	800.5	325956.8
COLBERT FOSSIL PLANT	ASH POND 4	3/3/2011	7/20/2011	462.9	457.3	452.5	435.0	4.6	452.8	578268.4
	POND 4 STILLING	3/3/2011	7/20/2011	461.4	460.1	452.5	435.3	7.3	452.8	184460.4
	POND 5 STILLING	11/23/2010	7/20/2011	474.3	471.1	467.0	456.1	4.3	466.8	532437.5
CUMBERLAND FOSSIL PLANT	MAIN ASH POND	11/4/2010	7/28/2011	394.8	391.8	383.9	350.2	7.5	384.3	1227531.6
	STILLING POND	11/4/2010	7/28/2011	395.0	394.4	383.9	346.0	10.1	384.3	605293.6
GALLATIN FOSSIL PLANT	ACTIVE ASH POND A	1/13/2011	5/24/2011	474.9	470.4	469.4	448.7	0.5	469.9	4782642.8
	ASH POND E	10/20/2010	5/24/2011	474.6	469.8	463.0	445.0	6.5	463.4	5164572.3
	STILLING POND	8/31/2010	6/2/2011	462.9	457.2	455.9	444.6	0.3	456.9	2100557.6
JOHN SEVIER FOSSIL PLANT	MAIN ASH POND	10/21/2010	5/9/2011	1143.9	1138.5	1133.2	1117.9	4.7	1133.8	449919.5
	STILLING POND	10/28/2010	5/9/2011	1144.3	1142.6	1133.2	1110.3	9.4	1133.2	221332.1
	DRY FLY ASH STILLING	2/21/2007	2/6/2008	1099.5	1098.6	1088.2	1076.9	10.7	1087.9	105867.1
WIDOWS CREEK FOSSIL PLANT	MAIN ASH POND A	5/2/2011	8/10/2011	638.7	633.0	631.6	613.4	5.5	627.5	2943932.8
	RED WATER POND		6/17/2011	618.9	608.2	602.5	596.3	5.4	602.8	463285.0
	GYPSUM STILLING	5/2/2011	8/10/2011	621.4	618.5	614.6	602.7	4.1	614.4	283858.9

US EPA ARCHIVE DOCUMENT

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APPENDIX A

Document 5

Area Topographic Map

US EPA ARCHIVE DOCUMENT



US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 6

2010 Annual Inspection of CCP Facilities and Ponds, Widows Creek Fossil Plant, Stevenson, Alabama, Stantec Consulting Services, May 14, 2010

2010 Annual Inspection of CCP Facilities and Ponds

Widows Creek Fossil Plant
Stevenson, Alabama

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One Team. Infinite Solutions
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Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

May 14, 2010

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May 14, 2010

rpt_003_175550002

Mr. Michael S. Turnbow
Tennessee Valley Authority
1101 Market Street
LP 2G-C
Chattanooga, Tennessee 37402

Re: 2010 Annual Inspection of CCP Facilities and Ponds
Widows Creek Fossil Plant
Stevenson, Alabama

Dear Mr. Turnbow:

Stantec Consulting Services Inc. (Stantec) has completed the 2010 annual inspections for CCP facilities and ponds at the Widows Creek Fossil Plant. Facilities reviewed included:

- Main Ash Pond A
- Bottom Ash Stack
- Inactive Dredge Cell
- Upper/Lower Stilling Pond
- Pump Pond
- Chemical Ponds
- Red Water Pond
- Gypsum Stack
- Gypsum Stack Stilling Pond
- Limestone Drainage Basin
- Coal Yard Drainage Basin and Perimeter Ditch
- Abandoned Ash Disposal Area

The field work was executed on April 14 and 15, 2010. The results of the work along with facility-specific recommendations for maintenance or other activities are included on the enclosed documents. In addition, the following general plant-wide recommendations and comments area offered:

- It is recommended that vegetation maintenance continue, including mowing and clearing tall grass/cattail growth at regular intervals. If lack of vegetation is observed during these operations, re-seeding should be performed as soon as possible. If vegetation establishment difficulties continue in any areas, then TVA should consider refining existing procedures or developing site specific specifications which address topsoil, fertilizing, seed mixtures, etc.

- It is recommended that TVA catalogue, assign a responsible party and due date, and track the completion of the facility-specific recommendations provided herein.

- Please note that this scope did not include a review of the current Operations and Maintenance Manual (O&M) for WCF. Stantec understands that TVA plans to update the current O&M manual to include dam safety-related items. Stantec also understands that TVA plans to develop an Emergency Action Plan for WCF as well.

- It is recommended that TVA personnel continue dike inspections/monitoring to look for changes or conditions that might affect dike integrity. The frequency and procedures for inspections should be consistent with TVA's newly implemented inspection program. Particular emphasis should be placed on reviewing and monitoring the seepage areas for changed or worsened conditions, and identifying and repairing other maintenance items such as animal burrows, erosion, and lack of vegetation.

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

Sincerely,

STANTEC CONSULTING SERVICES INC.

Chris Jones, EIT
Project Engineer

Robert D. Fuller, PE
Associate

/rws

Enclosures:

2010 Annual Inspection of CCP Facilities and Ponds

Widows Creek Fossil Plant
Stevenson, Alabama

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

May 14, 2010

Enclosure A

Main Ash Pond A

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	160 acres	**TVA Hazard Classification:**	Significant
Maximum Height (toe to top of dike):	53 feet	**Dike Length:**	11,590 feet
Plant Discharge to Facility:	77.4 cfs	**Current Pool Elevation:**	632.4 feet

2. Site Visit Information

Stantec Inspection Team: Chris Jones, EIT, Robert Fuller, PE, Craig Millhollin, and Greg Budd

TVA Staff Present: None

Field Inspection Date: April 15, 2010

Weather/Site Conditions: Sunny, 84 degrees, dry conditions

3. History/Current and Future Operations

History: The area is first shown on TVA Drawing 10N7400-R5 dated 1959, labeled as New Ash Disposal Area. The perimeter dikes were constructed in two dike raisings. The initial dike was constructed of rolled earth fill to an elevation of 626 feet. A second rolled earth dike was constructed above the initial dike sometime before 1973, bringing the top of the perimeter dike to its current elevation of 636 feet.

Current Operations: The Main Ash Pond receives both fly ash and bottom ash. The fly ash and bottom ash from Units 1-6 are sluiced to one ditch, while the bottom ash from Units 7-8 are sluiced to a separate ditch. The outfall from the sluice lines is located in the Bottom Ash Stack. The ditches flow through the Bottom Ash Stack and converge into one ditch which flows into the northern corner of the Main Ash Pond A. In the southwest corner of the Main Ash Pond, five spillways decant water to the Upper Stilling Pond.

Future Planned Based on discussions with TVA, the pool elevation may be

Operational Changes:	lowered to an elevation of 622 feet in an effort to improve dike stability. This will require material to be dredged from the pond and placed in the Bottom Ash Stack and Dredge Cell areas. A new spillway system for the Main Ash Pond is also under development which may coincide with the plan to dredge material and lower the pool elevation. TVA plans to close the Main Ash Pond in FY 2015-2016.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Sparse vegetation noted on the northwest and southeast slopes. Bottom ash covering interior slope where vegetation is lacking (Photos 1-2).
Trees:	None observed.
Wave Wash Protection:	None observed.
Erosion:	Small erosion rills on the southeast interior slopes between the Dredge Cell and Main Ash Pond A and on the northwest slope where vegetation is lacking (Photos 2-4).
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	4 feet
Encroachments:	Material has been placed along the northwest side of the pond (Photos 4-5). Sediment is built up where the combined fly ash/bottom ash ditch enters the pond in the northern corner of the pond (Photo 6).
Slope:	2H:1V

4.2. Crest

Crest Cover and Slope:	Crushed stone coverage, relatively level along the northwest and northeast crests (Photo 7). Crushed stone/bottom ash on the southeast crest of the divider dike between the Dredge Cell and Main Ash Pond A

	(Photo 8).
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	20 feet.

4.3. Exterior Slopes

Vegetation:	Dense grass vegetation along most exterior slopes. Some areas of sparse coverage on the northwest slope between the Red Water Pond and Main Ash Pond A and on the northeast slope where woody growth was present prior to mowing (Photos 9-10). Sparse vegetation where rutting is evident (Photo 11) and at the toe of the northeast exterior slope (Photo 12).
Trees:	Large trees present at the toe of the northwest slope north of the Red Water Pond and at the toe of the northeast exterior slope along the edge of Widows Creek (Photos 10-12).
Erosion:	Erosion rills approximately twenty feet in length noted on the northwest exterior slope (Photo 13). Approximately sixty feet long erosion rill on the northeast slope reported in the March 2010 TVA quarterly report is still evident (Photo 14). There is also rutting evident on the 626 bench and at the toe of the northwest slope (Photos 11, 12, and 15).
Instabilities:	None observed.
Uniform Appearance:	The Main Ash Pond A is adjacent to the Dredge Cell along the southeast side and the Bottom Ash Stack along the southwest side, with no exterior slopes on these segments. There is a generally uniform appearance on the northeast and northwest slopes.
Seepage:	Seepage evident on the northeast slope (Photo 16). Seepage location noted on the March 2010 TVA quarterly report (Point No. 2024) was not evident at the time of the yearly inspection (Photo 17).

Benches:	Approximately ten feet wide bench along the northwest and northeast exterior slopes at elevation 626 feet. The bench varies in width in some locations, with little evidence of a bench on the northern portion of the northeast slope.
Foundation Drains, and Seepage Collection Systems:	No foundation drains or seepage collection systems.
Instrumentation:	Fifteen piezometers were installed by Stantec in 2009 and 2010 as part of the dike slope stability and seepage evaluation. Stantec is currently assisting TVA with the initial phase of an instrumentation monitoring program across all fossil plants. Piezometer readings are being taken and evaluated on a monthly basis.
Animal Burrows:	Seven animal burrows were noted along the northeast exterior slope. Three animal burrows were noted along the northwest exterior slope (Photo 18). One animal burrow was noted along the southeast exterior slope between the Main Ash Pond A spillways and the Upper/Lower Stilling Pond.
Slope:	2H: 1V
Height:	51 feet

4.4. Spillway Weirs/Riser Inlets

Number:	Five (5)
Size, Type and Material:	4 foot diameter, reinforced concrete manholes with 10-foot CMP skimmers (Photos 19-20)
Height of Riser Inlets:	35 feet
Access:	Access available to the middle riser via a steel walkway (Photo 21).
Joints:	Unable to be observed. Reported open joints during inspection performed in 2007.
Mis-Alignment:	Unable to be observed. Reported damage during dredging operation.

Closed/Abandoned Conduits:	Four wooden skimmers are located on the southeast side of the Main Ash Pond A. These appear to be abandoned spillways (Photo 22). For a complete inventory of existing conduits, reference is made to the WCF Pipe Inventory Report completed in April, 2010.

4.5. Outlet Pipes

Number:	Five (5)
Size, Type and Material:	36 inch diameter RCP
Headwall:	None.
Joint Separations:	Unable to be observed.
Mis-Alignment:	Unable to be observed.
Closed/Abandoned Conduits:	None reported or observed. Stantec conducted a pipe inventory of the stack to locate existing inactivate spillways.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Main Ash Pond A since the last annual inspection.

- The exterior slopes have been mowed.
- Woody vegetation and tree growth have been removed from the northwest exterior slope near the Red Water Pond Dike in accordance with a letter of recommendations dated November 17, 2009 (Photo 10).
- Crushed stone has been placed on all perimeter dike roads (Photo 8).
- The combined fly ash/bottom ash rim ditch on the northwest side of the pond has been moved inward towards the pond. The old rim ditch is used for storm water control (Photos 4-5).
- The crest has been widened along the northwest and northeast sides of the pond (Photo 7).
- Stacking of ash above the top of the clay dike level has begun in accordance with a 3-year stacking plan proposed by the Contractor.

6. Recommendations

URS Corporation is moving forward with design of pond modifications to reduce the hazard classification, improve operational conditions, and to ultimately achieve closure in FY 2015-2016. These design efforts will address slope stability, seepage, and spillway issues previously identified in Phase I and Phase II reports. The capital projects under design include rock toe buttressing, dredging and lower of pool in the Main Ash Pond A, spillway replacements in conjunction with closure of the dredge cell.

Therefore, the recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design and operation issues currently being addressed by URS. TVA Dam Safety priority codes are included in parentheses and described in Enclosure M.

- Continue to mow the exterior slopes as needed. (Priority 3)
- A lack of vegetation was noted at the toe of the northwest slope where rutting is evident. There is also rutting evident on the 626 bench. All ruts should be filled with compacted clay and seeded to re-establish vegetation in accordance with soil test recommendations from Auburn University which are kept onsite. (Priority 4)
- During the inspection, Stantec observed numerous animal burrows along the northeast and northwest dikes. It is recommended that these burrows be repaired in accordance with the guidelines shown in Enclosure N. (Priority 4)
- Future TVA inspections should continue to monitor the activity of burrowing animals, and repairs may need to include filling the burrow/tunnel with flowable fill. Last TVA should consider consulting a wildlife management expert to better control the burrowing problem since activity appears to be relatively high. (Priority 4)
- A few areas of erosion on the northeast and northwest exterior slopes were observed during the annual inspection. It is recommended that erosion rills/ruts from surface water runoff be filled and compacted with clay, and seeded to re-establish vegetation. (Priority 4)
- Continue inspections and monitor locations of known seepage problems. Refer to the Widows Creek Seepage Action Plan which is maintained in the Plant's SOS office for more information. (Priority 3)

Enclosure B

Bottom Ash Stack

Stantec

TVA 2010 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Bottom Ash Stack

1. General Facility Information

Facility Status: Active

Surface Area: 40 Acres

Maximum Height (toe to top of stack): 45 feet

2. Site Visit Information

Stantec Inspection Team: Chris Jones, EIT and Robert Fuller, PE

TVA Staff Present: None

Field Inspection Date: April 15, 2010

Weather/Site Conditions: Sunny, 84 degrees, dry conditions

3. History/Current and Future Operations

History: Commissioned pre-1967 to provide a stacking area for Bottom Ash.

Current Operations: The Bottom Ash Stack is currently being operated following the TransAsh three-year stacking plan. This plan consists of stacking ash and maintaining the fly ash ditch and bottom ash ditch which flow through the stack to the Main Ash Pond.

Future Planned Operational Changes: None at this time.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Exterior Slopes and Benches

Vegetation: Dense grass vegetation is located along the northwest slope (Photo 1). The south and southwest slopes have grass vegetation with adequate coverage in most locations. However, some locations have sparse/stressed vegetation (Photos 2-5). Vegetation lacking along the interior slopes of the perimeter ditch (Photo 6) and near the top of the exterior slope on the southwest slope (Photo 7). The 615 elevation bench on the northwest slope of the Bottom Ash Stack lacks vegetation (Photo 8). The 626 and 636 bench on the northwest slope are covered in bottom ash/crushed stone (Photo 9).

Trees:	Trees located on the northwest slope below the 626 bench near the Red Water Pond (Photo 10).
Erosion:	Some areas of localized erosion rills/gullies noted throughout, specifically where vegetation is lacking (Photo 11). Erosion rills/gullies present on the exterior slope below the 615 bench on the northwest side of the stack (Photo 12). Erosion was also noted due to pipe outfall where rip-rap is not present along the south slope (Photo 13).
Instabilities:	Sloughing along the perimeter road (Photo 26) continues to be a maintenance issue. The instability sheared slope inclinometer STN-86 in January, 2010 five feet below ground and continues to impact the roadside ditch. A slough is located on the northwest slope above the 636 perimeter road (Photo 14) where material has been placed since the previous annual inspection.
Uniform Appearance	The northwest slope appears to be uniform. The south slope varies (Photos 15-16).
Benches:	Benches are only located on the northwest slope. There is a 10 feet wide bench at elevation 615 feet, 45 feet wide bench at elevation 626 feet, and 20 feet wide bench at elevation 636 feet (Photos 8, 9, 17-19).
Slope:	On the northwest slope, 2H:1V below the 615 bench, 2.5H:1V to 3H:1V between the 615 and 626 bench, and 3H:1V between the 626 bench and 636 perimeter road and above the 636 perimeter road where material has been placed. The slope along the scrubber road on the south and southeast dikes is not uniform, with a slope ranging from 2H:1V where the sluice lines run up the slope to the bottom ash ditch to 4H:1 along the southeast slope. Slopes were measured from the 2009 TVA survey.
Height:	Approximately 45 feet (as measured in 2009 TVA survey), measured at the northwest side of the stack above the Red Water Pond.
Other:	Seepage has been a recurring issue along scrubber road at the toe of the Bottom Ash Stack. Red water seepage is still prevalent along this stretch of the exterior slope. The extent of seepage ranges from saturated slopes to flowing seepage at the southern corner of scrubber road. Based on the field observations, it appears the source of the seepage is from the Chemical Ponds located in the footprint of the stack and from

the bottom ash ditch and fly ash ditch which transport the slurry discharge through the Bottom Ash Stack to the Main Ash Pond (Photos 20-24).

Animal burrows were evident during the latest TVA quarterly inspection of the stack conducted in March, 2010. These burrows had not been repaired at the time of the yearly inspection. These burrows are located along the south and southwest slopes.

4.2. Perimeter Drainage Ditches and Down-Drains

Vegetation:	Vegetation is present in the bottom of the perimeter ditch in some locations where channel lining is lacking (Photo 25).
Rip-Rap Channel Lining:	Channel lining is present north of the Units 7/8 sluice lines to the south corner of scrubber road. The channel lining is placed above the ditch bottom on the slopes of the ditch (Photos 26-27).
Erosion:	None observed.
Sedimentation in Ditches:	No major issues observed.
Standing Water in Ditches or on Benches:	No standing water observed on the benches. Standing water is present in the ditches around the entire perimeter of the stack (Photos 28-30).
Silted/Impeded Drainage Pipes:	No issues observed.
Other:	No issues observed.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Bottom Ash Stack since the last annual inspection.

- The exterior slopes of the stack have been mowed.
- Material has been stacked according to the TransAsh three year stacking plan above the 636 perimeter road along the northwest side of the stack. A clay cover was placed over the bottom ash material on the exterior slope. Grass vegetation has now been established on this slope (Photos 9, 14, and 17).
- Eight piezometers and two slope inclinometers were installed around the perimeter of the stack. Four piezometers were installed on the northwest slope of the stack,

two on the 636 bench and two on the 626 bench. Four piezometers and two slope inclinometers were installed on the south slope: three piezometers along the crest road and one piezometer at the toe of the slope just above the perimeter ditch. Both slope inclinometers were installed at the base of the stack just above the perimeter ditch. The instruments were installed Summer, 2009 and have been read monthly since installation by Stantec.

6. Recommendations

URS Corporation is moving forward with a closure design for the Bottom Ash Stack. These design efforts will address slope stability and seepage issues previously identified. Therefore, the recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design issues currently being addressed by URS. Priority codes are included in parentheses and described in Enclosure M.

- Continue to mow the exterior slopes of the stack as needed. (Priority 3)
- During the inspection, animal burrows were noted along the south and southwest exterior slopes. It is recommended that these burrows be repaired in accordance with the guidelines provided in Enclosure N. Future inspections should continue to monitor for the activity of burrowing animals, and repairs should be made as needed. (Priority 4)
- Stantec developed a work plan to repair sloughing and control the seepage along the south and southwest slopes of the stack. This work plan was submitted to TVA for review. It is recommended that this plan be incorporated into URS mitigation design plan and implemented to address the red water seepage and associated slope instability along this perimeter segment of the stack. (Priority 4)

Enclosure C

Inactive Dredge Cell

1. General Facility Information

Facility Status:	Inactive	**NID Identification:**	N/A
Surface Area (inside dikes):	116 Acres	**TVA Hazard Classification:**	Significant
Maximum Height (toe to top of dike):	53 feet	**Dike Length:**	8,622 feet
Plant Discharge to Facility:	0 cfs	**Current Pool Elevation:**	N/A

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT, Robert Fuller, PE, Craig Millhollin, and Greg Budd
TVA Staff Present:	None
Field Inspection Date:	April 15, 2010
Weather/Site Conditions:	Sunny, 84 degrees, dry conditions

3. History/Current and Future Operations

History:

The Dredge Cell is located in the previous location of the Gypsum Scrubber Sludge Pond. The initial dike was constructed of rolled earth fill to an elevation of 626 feet. A second rolled earth dike was constructed above the initial dike sometime before 1973, bringing the top of dike elevation to 636 feet. The gypsum/scrubber sludge product was discharged into the southwest corner, south of the deflector dike running from west to east across the central part of the pond. Once TVA switched to a Forced Oxidation Gypsum Scrubber in the mid-1980's, a new Gypsum Stack was created. After the Gypsum Stack was in service beginning in 1986, the Scrubber Sludge Pond was used as a dredge cell for the Main Ash Pond. In the mid 2000's the perimeter dikes were raised to their current elevation of 646 feet using Bottom Ash fill. In 2007, plans were developed to again raise the dikes around this area, as the remaining capacity would be exhausted by 2009. This dike raising was not constructed.

Current Operations:	The Dredge Cell is currently inactive and being allowed to dewater.
Future Planned Operational Changes:	Based on discussions with TVA, URS Corporation is charged with designing a closure plan for the Dredge Cell.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	None observed.
Trees:	None observed.
Wave Wash Protection:	None observed.
Erosion:	The interior slopes of the Dredge Cell are constructed of bottom ash material. Small erosion rills from surface water runoff noted along the interior slopes of the entire perimeter (Photos 1-3).
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	N/A
Encroachments:	None observed.
Slope:	2H:1V

4.2. Crest

Crest Cover and Slope:	The crest of the Dredge Cell is covered in bottom ash/crushed stone (Photos 4-5). The crest appears relatively flat on the northeast, southeast, and southwest sides of the cell, with the northwest crest sloping away from the Dredge Cell towards the Main Ash Pond A.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.

Width:	30 feet

4.3. Exterior Slopes

Vegetation:	Dense grass vegetation is located on the exterior slopes below the bench at elevation 636 feet, with some locations of sparse/stressed vegetation, specifically near the top of the slope just below the 636 bench (Photos 6-8). On the southwest slope, there is no vegetation just below the 636 bench leaving exposed bottom ash (Photo 9). Vegetation is lacking on the exterior slopes between the 636 bench and the crest (Photos 10-13). Exposed bottom ash is located on the northwest slope and exposed bottom ash/clay cover is located on the southwest, northeast, and southeast slopes.
Trees:	None observed.
Erosion:	Erosion rills/gullies located on the exterior slopes above the 636 bench along the entire perimeter of the stack (Photos 10-11). Also, erosion rills located on the southwest exterior slope just below the 636 bench where vegetation is lacking (Photo 9). Localized erosion at the southern corner of the Dredge Cell on the 636 bench where the sluice lines run up the slope and are buried under the bench (Photo 14).
Instabilities:	None observed.
Uniform Appearance:	Yes
Seepage:	There were no apparent seepage issues observed for the Dredge Cell. The March, 2010 TVA quarterly inspection denoted a flowing (wet weather) seep on the northeast slope which was not evident during this inspection (Photo 15).
Benches:	There is one bench at approximate elevation 636 feet. This bench is located on the southwest, southeast, and northeast slopes. The bench is covered in crushed stone/bottom ash (Photo 16).
Foundation Drains, and Seepage Collection Systems:	None observed.

Instrumentation:	Twenty-three piezometers were installed in 2009 by Stantec as part of a Phase 2 dike slope stability and seepage evaluation. Stantec is currently assisting TVA with the early phases of an instrumentation monitoring program across all fossil plants. Piezometer readings are being taken and evaluated by Stantec on a monthly basis.
Animal Burrows:	Two animal burrows were noted along the northeast slope near the toe (Photos 17-18).
Slope:	2.5H:1V on the northeast exterior slope below the 636 bench, 3H:1V on the southeast and southwest slope below the 636 bench, and 4H:1V on all slopes above the 636 bench (as measured in 2009 TVA survey used for Stantec geotechnical evaluation).
Height:	Approximately 42 feet.

4.4. Spillway Weirs/Riser Inlets

Number:	2
Size, Type and Material:	3'L x 1'W x ~12'D, Aluminum frame, fiberglass stop log riser structures
Height of Riser Inlets:	Approximately 12 feet
Access:	Adequate access to both structures.
Joints:	In working order.
Mis-Alignment:	None observed.
Closed/Abandoned Conduits:	There are two weirs located on the northwest side of the Dredge Cell (Photos 1 and 5). These do not appear to be abandoned, but are no longer used as the Dredge Cell no longer receives process water from the plant.

4.5. Outlet Pipes

Number:	2
Size, Type and Material:	18 inch diameter corrugated HDPE
Headwall:	None

Joint Separations:	Buried, no evidence of joint separation.
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None observed or reported.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Dredge Cell since the last annual inspection.

- The exterior slopes have been mowed.
- New sluice lines that run along the southwest of the Dredge Cell on the 636 bench were buried. It appears the ditch that runs along this bench was cleaned (Photos 10-12).
- A pipe was installed under the crest of the 636 bench on the northeast slope during the dredging of Widows Creek after the January, 2009 event. The pipe was used to run the hose from the creek dredging operation under the road on this bench so that access around the Dredge Cell would not be blocked (Photo 19).
- Crushed stone was placed on the northwest crest between the Dredge Cell and Main Ash Pond A (Photo 5).

6. Recommendations

URS Corporation is moving forward with a closure design for the Dredge Cell. These design efforts will address slope stability and seepage issues previously identified. Therefore, the recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design issues currently being addressed by URS. Priority codes are included in parentheses and described in Enclosure M.

- Continue to mow the exterior slopes as needed to allow for future inspections of the slopes/dikes. (Priority 3)
- Standing water is present in the ditch on the southwest side at the 636 bench elevation. The sluice lines in this area were buried and work was completed on this ditch, but it appears that the ditch does not have adequate slope to freely drain (Photos 10, 12, and 16). (Priority 4)
- The culvert running under the road from the Main Ash Pond dike to the Dredge Cell on the southwest side is filled with sediment and nearly buried. This pipe should be replaced to promote flow in the ditch around the Dredge Cell (Photos 20-21).

(Priority 4)

- During the inspection, Stantec observed two animal burrows along the north dike and at the northeast corner of the stack (Photos 17-18). It is recommended that these burrows be repaired in accordance with the guidelines provided in Enclosure N. Future TVA inspections should continue to monitor the activity of burrowing animals, and repairs may need to include filling the burrow/tunnel with flowable fill. Last TVA should consider consulting a wildlife management expert to better control the burrowing problem since activity appears to be relatively high. (Priority 4)
- The pipe installed during the dredging operation of Widows Creek under the 636 bench on the northeast side of the Dredge Cell should be removed (Photo 19). It is currently acting as a discharge pipe for the perimeter ditch, which is not the intended purpose. (Priority 4)

Enclosure D

Upper/Lower Stilling Pond

Stantec

TVA 2010 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Upper/Lower Stilling Pond

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	8 acres	**TVA Hazard Classification:**	Significant
Maximum Height (toe to top of dike):	21 feet	**Dike Length:**	2,980 feet
Plant Discharge to Facility:	77.4 cfs	**Current Pool Elevation:**	611.5 feet

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT and Robert Fuller, PE
TVA Staff Present:	None
Field Inspection Date:	April 15, 2010
Weather/Site Conditions:	Sunny, 84 degrees, dry

3. History/Current and Future Operations

History:	Commissioned in 1986 as the third of four ponds in a series. The upland watershed includes Bottom Ash Stack, Main Ash Pond, and Dredge Cell.
Current Operations:	Effluent from the Main Ash Pond enters the Stilling Ponds from a system of five spillways. The stilling pond discharges into the pump pond via five spillways, maintaining a pool elevation of approximately 611.5 feet. The flow of water is directed around an approximately 100-foot divider dike separating the Upper Stilling Pond from the Lower Stilling Pond where carbon dioxide is added to balance pH.
Future Planned Operational Changes:	The existing Main Ash Pond and Upper/Lower Stilling Pond spillways will be replaced and abandoned in place. New spillways will be designed and installed similar to the new spillway at Johnsonville. The work will include a dewatering siphon system and emergency overflow spillway in one or both ponds. The design effort is currently ongoing.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Partially covered in grass, partially covered in stone for wave wash protection. Adequate coverage in most locations. Some locations have bare spots with no vegetation (Photos 1-5). Also erosion in the northern "groin" (Photos 6-9) and along the southwestern interior slope where wave wash protection is absent. Bare spots noted near the crest of the dike on the northeast and northwest slope (Photos 1, 3, and 10).
Trees:	None observed.
Wave Wash Protection:	Stone wave wash protection located on the southeastern side between the Stilling Pond and Pump Pond, the southwestern side, and the interior divider dike (Photos 11-15).
Erosion:	Small erosion rills noted on the southwest interior slope where wave wash protection is absent (Photo 16). Surface erosion rills also noted on the northeast interior slope just below the 626 bench (Photo 10).
Instabilities:	None observed.
Animal Burrows:	Animal burrows located on the northeast and northwest dikes at the toe of the dike near the waterline (Photos 17-18).
Freeboard:	2.5 feet measured at southeast dike.
Encroachments:	None observed.
Slope:	2H:1V southeast and southwest 3H:1V northeast and northwest

4.2. Crest

Crest Cover and Slope:	Crushed stone surface on the southwest and northwest sides, ash surface on the northeast and southeast sides. Crest slopes toward pond on northeast, northwest, and southwest dikes, and appears to slope towards the Pump

	Pond on the southeast dike.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	45 feet measured on the northwest dike, 18 feet measured on the southwest dike, 25 feet measured on the northeast dike, and 30 feet measured on the southeast dike.

4.3. Exterior Slopes

Vegetation:	The Stilling Pond is located adjacent to the Main Ash Pond and Dredge Cell. There is only an exterior slope on the southwest and southeast dikes. The southwest side has adequate grass coverage. The southeast dike is lacking vegetation and is covered in crushed stone/ash.
Trees:	None observed.
Erosion:	Substantial erosion noted at two locations on the southeast slope. The first location is a localized rill from surface runoff. The second location is the eastern portion of the dike where a slide appears to have occurred. Erosion rills are located along this portion of the dike. See the Pump Pond report for photos.
Instabilities:	A slide approximately 60 feet in length occurred on the eastern portion of the southeast dike (Photo19). See the Pump Pond report for additional description and photos.
Uniform Appearance:	Yes, the exterior slopes appear uniform.
Seepage:	Yes, seepage noted on both the southeast and southwest slopes (Photo 20).
Benches:	None observed.
Foundation Drains, and Seepage Collection Systems:	None reported or observed.
Instrumentation:	Two piezometers installed in February, 2010 on the

	northwest dike between the Stilling Pond and the Main Ash Pond. Two piezometers installed in May, 2009 on the northeast dike between the Stilling Pond and the Dredge Cell.
Animal Burrows:	None observed.
Slope:	2H:1V southwest side 1.5H:1V southeast side
Height:	11 feet measured on the southeast side

4.4. Spillway Weirs/Riser Inlets

Number:	Five (5)
Size, Type and Material:	48-inch diameter push together, 4-feet barrel section man hole-type concrete riser structure with 10-foot steel skimmer (Photos 21-22)
Height of Riser Inlets:	19.5 feet
Access:	Access to one riser structure via a steel walkway (Photo 23)
Joints:	Unable to be observed.
Mis-Alignment:	Last inspection reported joint leakage.
Closed/Abandoned Conduits:	None reported or observed.

4.5. Outlet Pipes

Number:	Five (5)
Size, Type and Material:	36-inch diameter RCP
Headwall:	None, outlet under at least six feet of water in the middle of the Pump Pond.
Joint Separations:	Unable to observe.
Mis-Alignment:	Unable to observe.
Closed/Abandoned Conduits:	None reported or observed.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance activities and improvements have been performed for the Stilling Pond since the last annual inspection.

- The interior slopes have been mowed (Photos 24-26).
- The floating ash/cenospheres in the Stilling Pond have been removed from the surface of the pond.

6. Recommendations

URS Corporation is moving forward with a closure design for the Upper and Lower Stilling Pond. These design efforts will address slope stability and seepage issues previously identified. The mitigation project should include the construction of compacted clay dikes on the southeast and southwest dikes. Existing bottom ash should be removed from dikes and replaced with compacted clay. The alignment of the southwest dike should be straightened (Photo 27). This work should be completed in a 6-month time frame. The recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design issues currently being addressed by URS. The following maintenance recommendations are offered for the Upper/Lower Stilling Pond. Priority codes are included in parentheses and described in Enclosure M.

- Continue to mow the interior slopes as needed. (Priority 3)
- Continue to maintain the floating booms (turbidity curtains) across the pond to collect the floating cenospheres (Photos 24-26). Clean these cenospheres from the pond as needed. (Priority 4)
- Repair animal burrows near the waterline on the northeast and northwest interior slopes in accordance with the guidelines provided in Enclosure N (Photos 17-18). (Priority 4)
- Repair erosion in the northern groin of the interior slopes (Photos 6-9). It is recommended that the erosion rills be filled/graded with compacted clay and a minimum of 30 inches of rip-rap be placed to form a uniform lined ditch from top to bottom. Replace the straw bales currently used for storm water control with a silt fence until ditch lining is complete. (Priority 4)
- Seed bare spots on the northeast and northwest interior slopes near the crest of the dike to re-establish vegetation. A new seed/fertilizer mixture may need to be used to enable vegetative growth. (Priority 4)
- Repair erosion rills along the southwest interior slope. Place wave wash protection

where it is currently missing (Photo 16). (Priority 4)

- Repair the broken handrail on the northern platform of the interior divider dike (Photo 28). (Priority 4)

Enclosure E

Pump Pond

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	0.4 acres	**TVA Hazard Classification:**	Significant
Maximum Height (toe to top of dike):	16 feet	**Dike Length:**	425 feet
Plant Discharge to Facility:	77.4 cfs Normal Operating conditions (recycle 100% of inflow back to plant)	**Current Pool Elevation:**	602 feet

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT and Robert Fuller, PE
TVA Staff Present:	None
Field Inspection Date:	April 14, 2010
Weather/Site Conditions:	Sunny, 83 degrees, dry conditions

3. History/Current and Future Operations

History:

The Pump Pond was commissioned in 1986 to recirculate process water from the Ash Pond Complex back to the power plant intake.

Current Operations:

The Pump Pond receives process and storm water from the Lower Stilling Pond via five spillway outlet pipes. The pond operates using a two-pump system. The large pump in the middle of the pond (U7/U8 Intake Pump Station) recirculates the water back into the sluice system. The pump located in the southern corner (Ash Pond Recirculation Pump Station) pumps water to the ball mill as makeup water. If the pumps are disables/non-functioning, the effluent and storm water flows through three spillway conduits to Outfall 001. See Photo 1 for an overview of the Pump Pond.

Future Planned Operational Changes:

TVA plans to close the Pump Pond by filling the impoundment with controlled fill and creating a permanent Lower Stilling Pond outfall system with a spillway outfall into the cove

leading to the Tennessee River. This is a conceptual plan, and the design effort has yet to commence by URS.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Dense vegetation on the northeast and southeast interior slopes. Sparse vegetation located on the northwest and southwest slopes.
Trees:	Small cluster of small trees (2" diameter) noted in the eastern corner of the pond (Photo 2).
Wave Wash Protection:	Wave wash protection located along the entire southwest interior slope (Photo 3) and along the base of the northwest interior slope (Photo 4). Wave wash protection also located along the southeast slope around the three outlet pipes (Photos 5-9).
Erosion:	Substantial erosion noted at two locations on the northwest slope. The first erosion area is a localized rill from surface runoff (Photos 10-11). The second location is the eastern portion of the dike where a slough appears to have occurred (Photo 11). Erosion rills are located along this portion of the dike (Photos 12-14).
Instabilities:	A slough occurred on the eastern portion of the northwest dike (see above). The entire interior slope of the northwest dike is relatively steep and subject to erosion due to surface runoff, seepage from the stilling pond (Photo 15), and cyclic rapid draw down conditions from the pump station operation .
Animal Burrows:	None observed.
Freeboard:	9 feet
Encroachments:	None observed.
Slope:	1.5H:1V

4.2. Crest

Crest Cover and Slope:	Crest covered in crushed stone with adequate coverage. The northwest, northeast, and southwest crests slope toward the interior of the pond, while the southeast crest slopes away from the pond toward a cove to the river.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	30 feet wide on northwest dike, 25 feet wide on northwest dike in location of slough, and 100 feet wide on southeast dike.

4.3. Exterior Slopes

Vegetation:	The Pump Pond is surrounded by the Old Scrubber Sludge Pond (Dredge Cell) to the northeast, the Upper/Lower Stilling Pond to the northwest and a constructed wetlands to the southwest. The only exterior slope is on the southeast dike, between the Pump Pond and a cove leading to the Tennessee River. This downstream slope has dense woody vegetation on either side of an existing discharge channel (Photo 16).
Trees:	Yes, numerous trees noted.
Erosion:	None observed.
Instabilities:	None observed.
Uniform Appearance:	No, the slope does not appear to be uniform.
Seepage:	None observed.
Benches:	None observed.
Foundation Drains, and Seepage Collection Systems:	No provisions for seepage collection.
Instrumentation:	None.

Animal Burrows:	None observed.
Slope:	3H:1V
Height:	16 feet

4.4. Spillway Weirs/Riser Inlets

Number:	The Pump Pond does not have spillway weirs nor skimmers under normal operating conditions. The outlet system is via two pumps. The pump in the middle of the pond (Photo 17) is the U7/U8 Intake Pump Station which recirculates water back to the power plant to reuse in the wet-sluice ash disposal system. The pump in the southern corner (Photo 18) is the Ash Pond Recirculation Pump Station (Ball Mill Pump Station) which circulates water to the limestone ball mill.
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None reported or observed.

4.5. Outlet Pipes

Number:	If the pumps are disabled/non-functioning, three (3) spillway conduits penetrating the southeast dike serve as the outfall for the Pump Pond (Photos 6-9).
Size, Type and Material:	36 inch x 60 inch elliptical corrugated metal pipe (CMP).
Headwall:	None.
Joint Separations:	None observed.
Mis-Alignment:	None observed.
Closed/Abandoned Conduits:	None reported or observed.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Pump Pond since the last annual inspection.

- The interior slopes of the pond have been mowed.
- The inactive sluice lines along the southeast dike were removed. The in-service sluice lines are in place along the crest of the dike.
- The two pump stations that service the Pump Pond were taken out of service for an extended period of time which forced the three outlet pipes to convey the process and storm water flows. A segment of the furthest southwest spillway conduit was unable to handle this flow due to a bad joint connection in the conduit which caused the first twenty-foot section of the conduit to become displaced. This segment has been removed. Class I rip-rap was placed around all three conduits as specified by Stantec. The work was documented in a letter on December 16, 2009 under Work Plan 8 - Pump Pond Emergency Response (Photos 6-9).

6. Recommendations

Based on discussions with TVA, it is understood that URS is currently designing a new spillway system in the Lower Stilling Pond to discharge process and storm water flows to through Outfall 001 and removing the existing pump stations. Therefore, the recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design issues currently being addressed by URS. Priority codes are included in parentheses and described in Enclosure M.

- Continue to mow the interior slopes as needed. (Priority 3)
- Remove the cluster of trees and root systems in the eastern corner of the pond in accordance with the guidelines provided in Enclosure O (Photo 2). (Priority 4)
- Two areas of erosion were noted during the inspection on the interior slope of the northwest dike between the Lower Stilling Pond and Pump Pond (Photos 10-14). It is recommended that these rills be filled and graded with compacted clay. All unprotected interior slopes should be improved with wave wash protection consisting of ALDOT Class I Rip-Rap. (Priority 4)
- The apparent slough located on the eastern portion of the northwest dike should be repaired to reconstruct the slopes with compacted clay to a final grade of 3:1 or flatter. It is recommended that this dike be evaluated and a design developed to mitigate the risks against future slides/instabilities. (Priority 4)

- The culvert installed from the engineered wetlands to the Pump Pond to direct the flow of water from the low-lying wetlands into the pond is not functioning properly. The invert of the culvert was installed at a higher elevation than the water surface of the pool that develops in the engineered wetlands after a rainfall event. A board has been placed across the outlet end of this culvert into the Pump Pond (Photo 19). It is recommended that this pipe be either removed or reset at a lower elevation to fulfill the intended purpose of draining the engineered wetlands. (Priority 5)
- It is apparent that the 12-inch steel pipe was abandoned in place by placing a blind flange (cover) over the inlet end (Photo 20). It is recommended that this abandoned pipe (Photo 21) be identified, excavated, and removed as part of the Pump Pond mitigation project currently undertaken by URS. (Priority 5)

Enclosure F

Chemical Ponds

TVA 2010 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Chemical Ponds

1. General Facility Information

Facility Status:	Inactive	**NID Identification:**	N/A
Surface Area (inside dikes):	Iron Pond: 2 acres Copper Pond: 1.5 acres	**TVA Hazard Classification:**	N/A
Maximum Height (toe to top of dike):	N/A	**Dike Length:**	Iron Pond: 1,300 feet Copper Pond: 1,000 feet
Plant Discharge to Facility:	N/A	**Current Pool Elevation:**	624 feet

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT and Robert Fuller, PE
TVA Staff Present:	None
Field Inspection Date:	April 14, 2010
Weather/Site Conditions:	Sunny, 84 degrees, dry conditions

3. History/Current and Future Operations

History:	The Iron Pond and Copper Pond were first shown on TVA-Drawings 10W213-R5 in 1976. The ponds are located within the interior of the Ash Pond Complex perimeter dikes in the south of the Bottom Ash Stack. This area was previously the Ash Disposal Area for Units 7-8.
Current Operations:	The chemical ponds are currently not receiving any discharge from the plant.
Future Planned Operational Changes:	None reported at this time.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	The Iron Pond is lined with wave wash protection with no vegetation. The Copper Pond has dense grass vegetation with adequate coverage on all interior slopes. There are some locations with woody growth (Photos 1-2).
Trees:	None observed.
Wave Wash Protection:	Wave wash protection is present along all interior slopes of the Iron Pond (Photos 3-6). Some wave wash protection noted on the southeast interior slope of the Copper Pond (Photo 2)
Erosion:	None observed.
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	5 feet, measured at the southeast dike of the Iron Pond. It should be noted that these ponds are located within the interior of the Ash Pond Complex perimeter dike, and the freeboard was measured for the immediate dike surrounding the ponds. Should dike overtop, the overflow would drain into a basin still within the limits of the main perimeter dike.
Encroachments:	None observed.
Slope:	Iron Pond - 2H:1V Copper Pond - 2H:1V

4.2. Crest

Crest Cover and Slope:	The crest of both ponds is covered with bottom ash and appears relatively flat (Photo 3).
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	30 feet

4.3. Exterior Slopes

Vegetation:	The Chemical Ponds are located within the perimeter dikes of the Ash Pond Complex. The exterior slopes associated with these dikes will be discussed within the Bottom Ash Stack report.
Trees:	N/A
Erosion:	N/A
Instabilities:	N/A
Uniform Appearance:	N/A
Seepage:	N/A
Benches:	N/A
Foundation Drains, and Seepage Collection Systems:	N/A
Instrumentation:	N/A
Animal Burrows:	N/A
Slope:	N/A
Height:	N/A

4.4. Spillway Weirs/Riser Inlets

Number:	The Chemical Ponds does not have spillway weirs nor skimmers. The outlet system is via a pump station located in each pond. The pumps are located in the center of the southeast dike of each pond (Photos 6-8). Water is pumped from the pond to a ditch traversing through the Bottom Ash Stack. The ponds no longer receive discharge from the plant, and currently maintain the existing pool elevation which is controlled by manually operating the dewatering pumps.
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A

Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None observed or reported.

4.5. Outlet Pipes

Number:	See section 4.4 for description of the pump outlet system.
Size, Type and Material:	N/A
Headwall:	N/A
Joint Separations:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None observed or reported.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Chemical Ponds since the last annual inspection.

- The interior slopes of the ponds and drainage basin have been mowed.
- It appears that the two drainage pipes noted in the previous inspection from the drainage basin through the Ash Pond Complex perimeter dike have been cleared and unclogged. A silt fence was placed around the outfall of these two pipes, but the fence is in need of repair (Photos 9-10).

6. Recommendations

Based on the yearly inspection, the following maintenance recommendations are offered for the Chemical Ponds. Priority codes are included in parentheses and described in Enclosure M.

- Continue to mow the interior slopes and vegetation in the drainage basin as needed. (Priority 3)
- If the Chemical Ponds will not be used in the future, a closure plan should be developed to dewater and close the ponds. The pond footprint could be used for the stacking of dry ash. (Priority 5)

- Repair the silt fence around the two outlet pipes from the drainage basin. (Priority 4)

Enclosure G

Red Water Pond

Stantec

TVA 2010 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Red Water Pond

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	16 acres	**TVA Hazard Classification:**	N/A
Maximum Height (toe to top of dike):	12 feet	**Dike Length:**	340 feet
Plant Discharge to Facility:	N/A	**Current Pool Elevation:**	604 feet

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT and Robert Fuller, PE
TVA Staff Present:	None
Field Inspection Date:	April 15, 2010
Weather/Site Conditions:	Sunny, 84 degrees, dry conditions

3. History/Current and Future Operations

History:	The Red Water Pond Dike was design in 1987 (TVA Drawing 107466-1 and 107466-2).
Current Operations:	The Red Water Pond collects surface runoff directly from the northwest slopes of the Main Ash Pond and indirectly from the northwest, west, and south slopes of the Bottom Ash Stack and Chemical Ponds via a perimeter ditch along Scrubber Road. The pond is controlled by a pump station, operating at a normal pool elevation between 600 and 604 feet.
Future Planned Operational Changes:	None reported at this time.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Eastern interior slope between the Main Ash Pond and Red Water Pond is covered in dense grass vegetation, with occasional bare spots where woody growth was present prior to mowing and from the tracks on the mower (Photos 1-4). The west slope between the railroad tracks and Red Water Pond has sparse vegetation with exposed soil/crushed stone where vegetation is lacking (Photos 5-7). The north slope has vegetation where wave wash protection is not present.
Trees:	Cluster of trees at the north end of the west dike (Photo 8).
Wave Wash Protection:	Wave wash protection present on the interior slope of the north dike, referred to herein as the Red Water Pond Dike (Photo 9).
Erosion:	Erosion observed on the south end of the east interior slope below the 620 feet bench elevation (Photos 10-12).
Instabilities:	None observed.
Animal Burrows:	TVA's March, 2010 quarterly inspection report identified animal burrows on the east dike. These animal burrows located along the toe of the east dike have not been repaired and are currently marked with pin flags. Additional burrows were also observed (Photos 13-14).
Freeboard:	5 feet, measured at the emergency overflow spillway on the north dike
Encroachments:	None observed.
Slope:	2H:1V north dike, east dike below the pump station access ramp and 620 bench located at the south end of the pond (Photos 15-16). 3:H1V west dike and east dike north of the access ramp to the pump station (Photos 5 and 9).

4.2. Crest

Crest Cover and Slope:	West dike crest is covered in railroad ballast along the railroad track and asphalt along the paved road. The west dike crest slopes toward the pond. The east dike is

	covered in crushed stone and appears crowned, with half the crest sloping towards the pond and half sloping towards the perimeter ditch of the ash pond. The north dike is covered in grass vegetation and rip-rap (Photos 17-19) .
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	20 feet: west dike (railroad) 50 feet: west dike (roadway) 20 feet: north dike 20 feet: east dike

4.3. Exterior Slopes

Vegetation:	The Red Water Pond is located immediately adjacent to the west side of the Main Ash Pond and the east side of the roadway/railroad tracks. The only exterior slope is located on the north dike. The exterior slope of this dike is covered in dense vegetation with thick mulch/wood chippings in one location (Photo 20). Channel Lining covers the west side of slope at the emergency overflow spillway notch and discharge channel.
Trees:	None observed.
Erosion:	None observed.
Instabilities:	None observed.
Uniform Appearance:	The slope of the dike appears uniform; however, the eastern portion of the slope is covered in vegetation while the western portion of the dike is covered in Channel Lining.
Seepage:	Yes, seepage in the form of standing water observed at the exterior toe of the north dike at the east end adjacent to the Main Ash Pond dike (Photos 21-22).
Benches:	None observed.

Foundation Drains, and Seepage Collection Systems:	None observed.
Instrumentation:	None observed.
Animal Burrows:	None observed. Possible beaver trail leading from the crest of the dike to the spillway riser to Widows Creek (Photos 23-24).
Slope:	3H:1V
Height:	12 feet

4.4. Spillway Weirs/Riser Inlets

Number:	The Red Water Pond does not have spillway weirs nor skimmers. The outlet system is via a pump station under normal conditions. The pump is located on the eastern side in the middle of the pond (Photo 25). Water is pumped from the pond up the slope to the old rim ditch/current perimeter ditch for the Main Ash Pond (Photos 26-27). The Red Water Pond (north) dike has an emergency overflow spillway notch and discharge channel. The spillway elements are well protected with large channel lining stone (Photo 19).
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None observed or reported.

4.5. Outlet Pipes

Number:	See Section 4.4 for description of pump outlet system.
Size, Type and Material:	N/A

Headwall:	N/A
Joint Separations:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None observed or reported.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed at the Red Water Pond since the last annual inspection.

- The interior slopes of the pond have been mowed.
- The woody vegetation and tree growth have been removed from the north dike in accordance with a letter sent by Stantec to TVA on November 17, 2009. This letter was prepared in response to Action Item 92 from the RHO&M Action Item Matrix (Photo 19).
- Ruts in the 626 bench which serves as the access road to the pump station and access ramp have been repaired. An approximately ten feet wide access road was created using bottom ash to cover the clay/vegetation access road and fill in the ruts. Ruts are still present on the bench/slope outside the new access road (Photo 28).
- An access ramp was cleared of trees from the crest north of the north dike to the base of the north dike near the access stairway (Photo 29).

6. Recommendations

Based on the yearly inspection, the following maintenance recommendations are offered for the Red Water Pond. Priority codes are included in parentheses and described in Enclosure M.

- Continue to mow the interior slopes as needed. (Priority 3)
- During the inspection, animal burrows were noted along the waterline of the east interior slope. It is recommended that these burrows be repaired in accordance with the guidelines provided in Enclosure N. Future inspections should continue to monitor for the activity of burrowing animals, and repairs should be made as needed. (Priority 4)
- Remove the cluster of trees and root systems at the north end of the west interior slope in accordance with the guidelines provided in Enclosure O (Photo 8). (Priority 4)

- Repair erosion on the interior slope of the east dike below the 620 bench (Photos 10-12). it is recommended that this erosion rill be filled with compacted clay. (Priority 4)
- During the inspection, areas lacking vegetation were noticed on the east interior slope. There appears to be limbs from woody growth that was mowed since the last inspection in these areas. Stantec believes that vegetation was unable to survive below the piles of limbs that was prevalent on the slopes in these locations. It is recommended that these locations be seeded to re-establish grassy vegetation. (Priority 4)
- Monitor the seepage located at the toe of the north dike (Photos 20-22). Stantec is currently conducting a geotechnical exploration of this dike and installing two piezometers to monitor pore water pressure through this dike in an attempt to better characterize this seepage issue. (Priority 3)
- In addition to the geotechnical exploration, remove the mulch covering the east portion of the north dike and sow grass as needed to allow better observation of the seepage issue (Photo 20). The ground conditions cannot be observed with the current coverage. (Priority 3)
- The outlet pipeline from the Red Water Pond pump station should be extended approximately 200 feet into the Main Ash Pond where the discharge flows directly into the recently relocated fly ash ditch. (Priority 4)

Enclosure H

Gypsum Stack

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	95 Acres	**TVA Hazard Classification:**	High
Maximum Height (toe to top of dike):	80 feet	**Dike Length:**	10,350 feet
Plant Discharge to Facility:	5,000 gpm (Unit 7/8 Combined)	**Current Pool Elevation:**	Pond 3 - 670 feet

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT, Robert Fuller, PE, Craig Millhollin, and Greg Budd
TVA Staff Present:	None
Field Inspection Date:	April 15, 2010
Weather/Site Conditions:	Sunny, 84 degrees, dry conditions

3. History/Current and Future Operations

History:	Commissioned in 1986. Contains sluiced material from Units 7 and 8. A failure of an abandoned decant weir occurred on January 9, 2009 releasing gypsum - fly ash material into the Stilling Pond and Widows Creek. Material placed around a newly constructed spillway outlet pipe in Pond 2B washed out, causing gypsum - fly ash material to spill down the slope into the Stilling Pond on January 4, 2010. Since January, 2010, Pond 2A/2B has been dewatered and will remain dry with the use of pumps. Pond 3 will continue to operate as the only clarification pond on top of the stack.
Current Operations:	The Gypsum Stack is currently being operated using wet handling methods in the east rim ditch. Material is excavated from the rim ditch and rolled into windrows. This material, once dried, is then transported to the interior of Ponds 1 and 2A/2B where it is mechanically placed and compacted.
Future Planned Operational Changes:	A 12-month operations plan for the Gypsum Stack is under development with the long term goal of closing the stack and

disposing of the CCP materials at an offsite landfill. The scheduled closure for the Gypsum Stack is in FY 2015. The interim 12-month plan will be to continue operating using wet handling methods. The overall goal is not to increase the pool level in Pond 3 until a dry press facilities is put in service within the next 12 months (May 1, 2011). A dry stacking plan is under development for operation of the stack between FY 2011 and FY 2015.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	No vegetation present on any interior slopes (Photos 1-2).
Trees:	None observed.
Wave Wash Protection:	None observed.
Erosion:	Yes, small erosion rills located on all interior slopes of Pond 3, Pond 2A/B, and the rim ditch (Photo 3).
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	Five feet
Encroachments:	Material is currently being mechanically placed inside the limits of Ponds 1 and 2A/B.
Slope:	2H:1V

4.2. Crest

Crest Cover and Slope:	Material is actively being placed at the Gypsum Stack. Because of this, there is no vegetation/stone coverage on the crest of the stack. The crest is exposed gypsum - fly ash material (Photos 4-6).
Erosion:	None observed.
Alignment:	No issues observed.

Settlement/Cracking: None observed.

Bare Spots/Rutting: None observed.

Width: Width ranges from 25 feet wide to over 100 feet wide.

4.3. Exterior Slopes

Vegetation: The exterior slopes of the stack generally have adequate coverage of grassy vegetation. A few areas have sparse grassy vegetation (Photo 7). Also, some locations on the clay starter dike below the perimeter road have grass coverage but the vegetation appears stressed (Photo 8). Areas of erosion due to surface water runoff noted in the March 2010 quarterly inspection have been repaired. These areas were filled with clay and seeded (Photos 9-10). Until 650/655 bench drain inlets are installed, the stormwater runoff erosion repairs will continue to be a significant maintenance task.

Trees: None observed.

Erosion: Many of the erosion rills/gullies that were present along the stack have been repaired since the March 2010 quarterly inspection (Photos 9-10). Smaller erosion rills which were not noted on the quarterly inspection are still present (Photos 11-13).

Instabilities: None observed.

Uniform Appearance: The exterior slopes are generally uniform around the perimeter of the stack.

Seepage: Seepage is evident around the perimeter of the stack. Two areas with clear flowing seepage were noted on the eastern exterior slope just above the perimeter ditch. One was near Outlet 9 (Photos 14-15) of the underdrain system and the other was between Outlet 9 and 10 (Photo 16). Both had rip-rap placed in the seepage area. A possible seepage area was observed on the western slope of Pond 3 in and just above the perimeter ditch (Photo 17)

Benches: From bottom to the top, the lowest bench around the perimeter of the stack is located at the top of the original clay starter dike at elevation 620 feet on the west side of

	the stack which linearly increases in elevation to approximately 628 at the northwest corner (Photo 18). The first tier bench around the perimeter of the stack is located at elevation 650/655 feet on the west, north, and east sides of the stack, transitioning to elevation 650 on the south side of the stack (Photo 19). The grading plan for the stack is provided in TVA Drawing Series 10W235. The benches are relatively flat with no provisions for handling stormwater runoff.
Foundation Drains, and Seepage Collection Systems:	The seepage collection system for the Gypsum Stack is depicted on TVA Drawing Series 10W235 showing a total of fifteen planned outlets. Two outlets of this system located along the west dike of Pond 2B between the Gypsum Stack and Stilling Pond have not been installed. The typical headwall for the outlet system has up to eight outlet pipes. The two outer-most pipe outlets (#1 and #8) are the laterals for the lowest toe drain. Moving inward, the next two pipes (#2 and #7) are the laterals for the first tier (650/655) bench seepage collection system. Based on the design drawings, the four inside outlet pipes are planned laterals at the 685 and 720 bench levels. See photos 21 to 33 for a picture and description of each outlet.
Instrumentation:	Piezometers are installed at 19 locations around the stack. One of these locations is a nested piezometer with two piezometers installed in different sensing zones. Also, three previously installed piezometers were damaged in the construction of Work Plan 5 (wcf-090602-wp-5) and were removed. Three slope inclinometers are installed. One is located near the midpoint of the north dike above the perimeter ditch, one is located on the west dike north of the stilling pond above the perimeter ditch, and one is located on the divider dike between Pond 2A/B and Pond 3.
Animal Burrows:	Animal burrows noted on the March 2010 TVA quarterly

	inspection were still present during the annual inspection. These were located on the north dike near the perimeter ditch and toe of the starter dike (Photos 35-36).
Slope:	The slope ranges from 2.4H:1V to 2.8H:1V below the 655 bench around the perimeter of the stack. On the west dike between Pond 2A/B and the Stilling Pond, the slope below the 655 bench is 4H:1V. The slope is 2.5H:1V around the perimeter of the stack above the 655 bench. The slope of the starter dike is 3H:1V.
Height:	The maximum height of the Gypsum Stack is approximately 80 feet measured on the south side of the stack.

4.4. Spillway Weirs/Riser Inlets

Number:	Two (2), one in Pond 2B and one in Pond 3 (Photos 37-38).
Size, Type and Material:	48 inch diameter concrete riser.
Height of Riser Inlets:	The approximate height of the riser in Pond 2B is 15 feet and the approximate height of the riser in Pond 3 is 6 feet.
Access:	No access to the riser inlets.
Joints:	Unable to be observed.
Mis-Alignment:	No issues observed.
Closed/Abandoned Conduits:	Work Plans 5 and 6 were completed in 2009. As part of these work plans, conduits which were found penetrating through the perimeter dike will grouted and abandoned in place. For a complete inventory of existing conduits, reference is made to the WCF Pipe Inventory Report completed in April, 2010.

4.5. Outlet Pipes

Number:	Two (Photo 38)
Size, Type and Material:	30 inch diameter solid HDPE pipe
Headwall:	None observed.

Joint Separations:	Unable to be observed.
Mis-Alignment:	No issues observed.
Closed/Abandoned Conduits:	See 4.4 for discussion on abandoned conduits.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Gypsum Stack since the last annual inspection.

- The exterior slopes of the stack have been mowed. Woody growth noted on the previous annual inspection report was no longer present.
- Erosion rills and ruts due to surface water runoff on the exterior slopes of the stack were repaired by filling in the rills/ruts with compacted clay.
- Vegetation coverage has been established on the exterior slopes above the 655 bench except along the west dike where material is actively being placed to an elevation of approximately 670. Vegetation is still sparse in some locations.
- On February 26, 2010, a seepage boil was discovered on the west side of Pond 3. A reverse graded filter was constructed to prevent piping of material by placing sand, TDOT No. 89 stone, and large rip-rap. Two 36 inch HDPE pipes were placed in the ditch to prevent the ditch from being blocked by the filter (Photo 39).
- The divider dike between Pond 2A/B and Pond 3 has been widened since the last annual inspection.
- The west dike of the Gypsum Stack (Photo 40) was modified during the summer of 2009 under Work Plan 5 (wcf-090602-wp-5). This work plan encompassed the following changes to this portion of the Gypsum Stack.
- A rock-toe buttress was constructed on the west dike. This buttress was constructed by placing C33 sand, TDOT No. 89's, and TDOT No. 2 stone on the slope. The buttress has a 2H:1V slope to a top elevation of 630 feet. The slope above the rock-toe buttress was cut back to 4H:1V. Stone was placed on the face of the slope to an elevation of 640 feet (Photo 40)
- The rock-toe buttress required that the perimeter ditch be filled in with sand and an 8 inch corrugated HDPE pipe to collect seepage at the toe of the slope. Five outlet pipes were installed from the corrugated pipe to discharge seepage collected in the drain into the Stilling Pond at various locations (Photo 38).
- Process and storm water from Pond 2A/B and Pond 3, which used to discharge onto the west dike exterior slope and flow into the perimeter ditch, currently flows directly into the Stilling Pond via solid HDPE pipes for each spillway structure (Photo 38).

- A new 36-inch diameter HDPE culvert system was installed from the existing perimeter ditch to the southeast corner of the Stilling Pond (Photos 41-42).

6. Recommendations

URS Corporation is moving forward with design of gypsum stack modifications to reduce the risk of slope stability failure, improve operational conditions, and to ultimately achieve closure. The initial design efforts will address slope stability and seepage issues previously identified primarily around the perimeter clay dike and lower portions of the stack below elevation 650 feet.

Stantec is currently designing an immediate repair plan (Work Plan 22) for the erosion that occurred in January, 2010 at the west dike of Pond 2B. The design and construction of the 650/655 bench surface drainage improvements is included in the 12-month operations plan. Therefore, the recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design and operation issues currently being addressed by URS and Stantec. TVA Dam Safety priority codes are included in parenthesis and described in Enclosure M.

- Continue to mow the exterior slopes as needed. (Priority 3)
- Soil cover thicknesses should be checked for conformance with the applicable TVA plans. (Priority 4)
- Bare spots and erosion rills noted on the March 2010 quarterly inspection and current annual inspection on the exterior slopes of the stack continue to be a maintenance issue. It is recommended the final grading, addition of topsoil, application of fertilizer/lime and hydroseeding be coordinated with the 655 bench drainage improvements planned this year. (Priority 4)
- During the inspection, Stantec observed three animal burrows along the north dike and at the northeast corner of the stack (Photo 34-35). It is recommended that these burrows be repaired in accordance with guidelines provided in Enclosure N. (Priority 4)
- Future TVA inspections should continue to monitor the activity of burrowing animals, and repairs may need to include filling the burrow/tunnel with flowable fill. Last TVA should consider consulting a wildlife management expert to better control the burrowing problem since activity appears to be relatively high. (Priority 4)
- Continue inspections and monitor locations of known seepage problems. Refer to the Widows Creek Seepage Action Plan for more information on monitoring potential seepage areas. (Priority 3)

Enclosure I

Gypsum Stack Stilling Pond

Stantec

TVA 2010 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Gypsum Stack Stilling Pond

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	7 acres	**TVA Hazard Classification:**	High
Maximum Height (toe to top of dike):	18 feet	**Dike Length:**	3,000 feet
Plant Discharge to Facility:	5,000 gpm	**Current Pool Elevation:**	614.0 feet

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT and Robert Fuller, PE
TVA Staff Present:	None
Field Inspection Date:	April 15, 2010
Weather/Site Conditions:	Sunny, 82 degrees, dry conditions

3. History/Current and Future Operations

History:	Commissioned in 1986 in conjunction with the Gypsum Stack. The spillways in Pond 2B and Pond 3 previously discharged directly onto the West Dike and ran down the exterior slope into the perimeter ditch, which discharged into the stilling pond. Work Plan 5 completed in 2009 routes the discharge of the spillways at the Gypsum Stack into a 30" diameter solid HDPE pipe discharging directly into the Stilling Pond.
Current Operations:	The Stilling Pond is currently able to receive discharge from two spillway riser structures located in Pond 3 and Pond 2B of the Gypsum Stack. At this time, Pond 2B is dry, with only the riser structure in Pond 3 discharging process and storm water into the Stilling Pond through the new spillway outlet system installed as part of Work Plan 5. The Stilling Pond also receives flow from the Gypsum Stack perimeter ditch from two directions. The Stilling Pond discharges into a steel spillway with a skimmer structure where it flows to the 008 Outfall.
Future Planned Operational Changes:	None at this time.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Dense grassy vegetation on the interior slope of the south and west dikes, and portions of the east dike (Photos 1-3). Vegetation lacking on the top half of the interior slope along the west dike (Photo 4). The north dike has sparse vegetation along the interior slope. Portions of the east dike are covered in crushed stone with some locations lacking coverage, resulting in exposed clay (Photos 5-6).
Trees:	None observed.
Wave Wash Protection:	Wave wash protection noted around the recently installed pipe outlets from Work Plan 5 on the east dike interior slopes (Photos 5-6).
Erosion:	Small erosion rills noted on the top half of the west dike interior slope where vegetation coverage is lacking (Photo 4)
Instabilities:	A small slough was noted in the March, 2010 TVA Quarterly Inspection on the interior slope of the south dike. This slough is still present (Photo 7).
Animal Burrows:	None observed.
Freeboard:	5 feet, measured at the emergency spillway on the northern portion of the west dike.
Encroachments:	None observed.
Slope:	2H:1V

4.2. Crest

Crest Cover and Slope:	The crest is covered in crushed stone on each dike (Photos 8-10). The crest appears to slope away from the pond on the south, west, and north dikes, and towards the pond on the east dike.
Erosion:	None observed.

Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	Shallow rutting on the west dike crest from tracked equipment (Photo 10).
Width:	25 feet on the east, north, and west dikes. 30 feet on the south dike.

4.3. Exterior Slopes

Vegetation:	Dense native grasses on the south dike exterior slope and portions of the west dike exterior slope. Lack of vegetation noted on top half at certain locations along the west dike (Photo 11).
Trees:	None observed.
Erosion:	None observed.
Instabilities:	None observed.
Uniform Appearance:	Yes.
Seepage:	Non-flowing seepage is evident on south and west exterior slopes (Photos 12-13).
Benches:	None observed.
Foundation Drains, and Seepage Collection Systems:	None installed.
Instrumentation:	None installed.
Animal Burrows:	None observed.
Slope:	3H:1V
Height:	15 feet

4.4. Spillway Weirs/Riser Inlets

Number:	There are no spillway weirs/riser inlets for the Gypsum Stack Stilling Pond. The outlet system for the pond consists of an outlet pipe with a skimmer barrel. This is discussed in 4.5 below.

Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None observed or reported. There is a planned emergency overflow notch located in the northwest end of the Stilling Pond (Photo 10). It has been recommended that the notch and downstream slope and channel be re-established using crushed stone lining. URS is developing these plans.

4.5. Outlet Pipes

Number:	One (1)
Size, Type and Material:	30 inch diameter steel pipe with a 42 inch diameter steel skimmer barrel (Photo 14).
Headwall:	N/A
Joint Separations:	None observed.
Mis-Alignment:	No issues observed.
Closed/Abandoned Conduits:	None observed or reported.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Gypsum Stack Stilling Pond since the last annual inspection.

- The wooden skimmer for the 30 inch diameter outlet pipe in the southwest corner of the pond was replaced with a steel skimmer barrel in May, 2009 (Photo 14). The new skimmer barrel was installed in accordance with Stantec Work Plan 3 (wcf-090225-wp-3).
- The interior slopes of the pond were mowed.
- Cenospheres floating on the pond surface are being captured with a series of floating booms. The removal of cenospheres continues on a periodic basis as

needed.

- The west dike of the Gypsum Stack was modified in the summer of 2009 under Stantec Work Plan 5 (wcf-090602-wp-5). This work plan encompassed the following changes to this portion of the Gypsum Stack and Stilling Pond.
- The perimeter ditch at the toe of the Gypsum Stack on the east side of the Stilling Pond was filled in with sand and an 8 inch corrugated HDPE pipe to collect seepage. The corrugated HDPE pipe has five outlet pipes extending into the Stilling Pond at various locations along the east dike (Photos 15-16).
- Process and storm water which used to flow into the perimeter ditch from the spillway system for the Gypsum Stack now flows directly into the Stilling Pond via solid HDPE pipes for each spillway structure (Photos 15-16).
- A new 36 inch diameter solid HDPE culvert pipe was installed from the existing perimeter ditch to the southeast corner of the Stilling Pond (Photos 17-18).

6. Recommendations

URS Corporation is moving forward with a closure design for the Gypsum Stack including the Gypsum Stack Stilling Pond. These design efforts will address slope stability and seepage issues previously identified. Therefore, the recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design issues currently being addressed by URS. Priority codes are included in parentheses and described in Enclosure M.

- Repair the slough located on the south interior slope by filling and compacting the area with clay. Seed the compacted clay to re-establish vegetation. (Priority 4)
- Seed the locations on the interior slope of the west dike to re-establish vegetation. A new fertilizer/seed mixture may need to be developed if vegetation is unable to be established in this area. (Priority 4)
- Continue to practice good cenosphere management by continuing to use floating booms (turbidity curtains) across the pond and cleaning the floating cenospheres from the pond surface as required. (Priority 4)
- The planned emergency overflow notch is located in the northwest end of the Stilling Pond (Photo 10). It is recommended that the notch and downstream slope and channel be re-established using crushed stone lining. URS is developing these plans. (Priority 5)

Enclosure J

Limestone Drainage Basin

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	LRP: 0.5 acres LRSP: 0.2 acres	**TVA Hazard Classification:**	N/A
Maximum Height (toe to top of dike):	LRP: N/A LRSP:10 feet	**Dike Length:**	LRP: 625 feet LRSP: 400 feet
Plant Discharge to Facility:	N/A	**Current Pool Elevation:**	605 feet

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT and Robert Fuller, PE
TVA Staff Present:	None
Field Inspection Date:	April 14, 2010
Weather/Site Conditions:	Sunny, 83 degrees, dry conditions

3. History/Current and Future Operations

History:

The Limestone Drainage Basin was first shown on plans in 1973 (TVA Drawing 10N8221), with the latest revision to add the Limestone Runoff Surge Pond (LRSP) in 1984 (TVA Drawing 10N8224-1).

Current Operations:

The Limestone Drainage Basin consists of two ponds, the Limestone Runoff Pond (LRP) and Limestone Runoff Surge Pond. The Limestone Runoff Pond collects surface runoff from the limestone stockpile area and an upland channel containing sluice lines. There is one 48 inch diameter inlet pipe from the slurry line ditch and an inlet trench from the limestone stockpile area. The Limestone Runoff Pond is hydraulically connected to the Limestone Runoff Surge Pond via two 30 inch diameter corrugated metal pipes and outlets to the engineered wetlands. The Limestone Runoff Surge Pond collects runoff from the Limestone Ball Mill via a ditch and a 24 inch diameter corrugated metal pipe. The outlet pipes from the Limestone Runoff Pond to the engineered wetlands are shown on TVA Drawing 10N8221 but were not found and

verified in the field.

Future Planned Operational Changes: It is Stantec's understanding that Worley Parsons has been tasked by TVA to evaluate the watershed of the Limestone Drainage Basin and to develop engineering recommendations to improve the hydraulics of the basin, eliminate permitted outfall(s), and repair the dike which has been leaking.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation: For the Limestone Runoff Pond, sparse to dense grass coverage on the interior slopes (Photos 1-3). The Limestone Runoff Surge Pond has dense grass/brush coverage on interior slopes.

Trees: None observed.

Wave Wash Protection: None observed.

Erosion: Substantial erosion rills noted at two locations on the western interior slope of the Limestone Runoff Pond (Photos 4-5). Smaller rills noted at various locations where there is a lack of vegetation on the interior slopes.

Instabilities: None observed.

Animal Burrows: None observed.

Freeboard: Limestone Runoff Pond: 4 feet
Limestone Runoff Surge Pond: 4 feet

Encroachments: Sediment buildup at the inlet ends of both ponds.

Slope: 1.5H:1V to 2H:1V - Limestone Runoff Pond
3H:1V - Limestone Runoff Surge Pond

4.2. Crest

Crest Cover and Slope: Crest covered in crushed limestone with adequate coverage, along the Limestone Runoff Pond, sloping toward pond interior. Limestone Runoff Surge Pond grass covered with adequate coverage, sloping toward pond

	interior.
Erosion:	Small rills along western crest of limestone drainage basin.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	25 feet

4.3. Exterior Slopes

Vegetation:	The drainage basin is situated in a depression. The only exterior slope is on the southeast dike of the Limestone Runoff Surge Pond. The slope is densely vegetated with mature trees and has large (5' to 10' diameter) boulders piled along the exterior toe.
Trees:	Densely vegetated with trees.
Erosion:	None observed.
Instabilities:	None observed.
Uniform Appearance:	There does not seem to be any method to the grading of the exterior slope, which cannot be observed due to the large pile of boulders scattered along the slope.
Seepage:	Yes, a seep was noted on the previous annual inspection which is still present (Photo 6).
Benches:	None
Foundation Drains, and Seepage Collection Systems:	None observed.
Instrumentation:	None observed.
Animal Burrows:	None observed.
Slope:	2H:1V
Height:	20 feet (Southeast Dike, LRSP)

4.4. Spillway Weirs/Riser Inlets

Number:	There are no spillway weirs/riser inlets for the Limestone Runoff Pond or Limestone Runoff Surge Pond.
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None reported or observed.

4.5. Outlet Pipes

Number:	LRP: Two (2) 18-inch diameter pipes, under water (Unable to be observed in field, information from TVA Drawing 10N8221 indicates invert elevation of 602 feet). LRSP: Two (2) 30-inch corrugated metal pipes through the northwest dike into the Limestone Drainage Basin (Photos 7-8).
Size, Type and Material:	LRSP: Two (2) 30-inch corrugated metal pipe LRP: two (2) 18-inch pipe (unknown material)
Headwall:	None.
Joint Separations:	None observed.
Mis-Alignment:	None observed.
Closed/Abandoned Conduits:	None reported or observed.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Limestone Drainage Basin since the last annual inspection.

- Vegetation on the interior slopes of the Limestone Runoff Pond has been mowed.
- Vegetation around the Limestone Runoff Surge Pond has been mowed and large

trees have been removed. Debris still remains in the pond (Photo 9).

- The trench running from the limestone stockpile to the Limestone Runoff Pond has been better defined (Photo 10).
- A dye test was performed for the Limestone Runoff Surge Pond to verify that the observed seep was from the pond. The test was conducted in November 2008 and dye surfaced from beneath the water approximately three minutes after the placement, and was seen in the visible seep approximately fifteen minutes after placement.

6. Recommendations

It is Stantec's understanding that Worley Parsons has been tasked to conduct an H&H analysis of the Limestone Drainage Basin and will provide engineering recommendations for improving the system. TVA should follow the recommendations obtained from that study. Based on the yearly inspection, the following maintenance recommendations are offered for the Limestone Drainage Basin. Priority codes are included in parentheses and described in Enclosure M.

- Repair the damaged 48-inch diameter corrugated metal pipe in the southern corner of the Limestone Runoff Pond. It appears that a hole developed in the top of the pipe, causing the material above the pipe to collapse into the pipe, blocking flow (Photos 11-14). Remove sedimentation blocking the inlet end of this pipe and protect inlet with rip-rap. (Priority 4)
- Two localized areas of erosion were noted during the inspection on the western dike of the Limestone Runoff Pond (Photos 2, 4, and 5). It is recommended that these be filled/graded. Also, smaller rills were noted along the entire interior slopes. These should also be filled/graded. (Priority 4)
- Regrade the interior slopes of the Limestone Runoff Pond to 3H:1V. (Priority 4)
- Clean and remove sediment from the Limestone Runoff Pond (Photos 3 and 11). (Priority 5)
- Remove debris from the Limestone Runoff Surge Pond (Photo 9). (Priority 5)
- Seepage from the southeast dike of the Limestone Runoff Surge Pond has been reported for several years. A remedial design should be developed to repair this dike (Photo 6). (Priority 5)

Enclosure K

Coal Yard Drainage Basin
and Perimeter Ditch

Stantec

TVA 2010 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Coal Yard Drainage Basin and Perimeter Ditch

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	2 acres	**TVA Hazard Classification:**	N/A
Maximum Height (toe to top of dike):	N/A	**Dike Length:**	1,500 feet
Plant Discharge to Facility:	N/A	**Current Pool Elevation:**	600 feet

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT and Robert Fuller, PE
TVA Staff Present:	None
Field Inspection Date:	April 14, 2010
Weather/Site Conditions:	Sunny, 85 degrees

3. History/Current and Future Operations

History:	The Coal Yard Drainage Basin was designed in 1977 to collect the runoff from the coal stockpile.
Current Operations:	Runoff from the coal stockpile flows into the coal yard perimeter ditch which flows into the coal yard sediment basin via three 18 inch diameter corrugated HDPE pipes. From the sediment basin, the runoff flows into the coal yard drainage basin via three 24 inch corrugated plastic pipes. The coal yard drainage basin is controlled by a pump station, which pumps the water to the main ash pond where it is discharged through Outfall 001.
Future Planned Operational Changes:	None at this time.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Grass-covered interior slopes with adequate coverage in most locations. The section on the east slope where the inlet pipes from the sediment basin enter the drainage basin is lacking vegetation and is covered in coal fines (Photos 1-2).
Trees:	None observed.
Wave Wash Protection:	None observed.
Erosion:	Erosion noted on the east slope where there is a lack of vegetation where the inlet pipes enter the pond (Photo 2).
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	5 feet
Encroachments:	None observed.
Slope:	1.5H:1V to 2H:1V

4.2. Crest

Crest Cover and Slope:	East crest covered in crushed stone and coal fines (Photo 3). South, North, and West crests are grass covered with adequate coverage (Photo 1). The crests slope towards the drainage basin.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	10 feet on the west dike, 15 feet on the east dike

4.3. Exterior Slopes

Vegetation:	N/A, drainage basin is dug out with no exterior slopes.
Trees:	N/A

Erosion:	N/A
Instabilities:	N/A
Uniform Appearance:	N/A
Seepage:	N/A
Benches:	N/A
Foundation Drains, and Seepage Collection Systems:	N/A
Instrumentation:	N/A
Animal Burrows:	N/A
Slope:	N/A
Height:	N/A

4.4. Spillway Weirs/Riser Inlets

Number:	This basin does not have spillway weirs. The outlet system is via a pump station which pumps the water to the main ash pond (Photo 4).
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None reported or observed.

4.5. Outlet Pipes

Number:	See 4.4 for description of pump outlet system.
Size, Type and Material:	N/A
Headwall:	N/A
Joint Separations:	N/A

Mis-Alignment: N/A

Closed/Abandoned Conduits: None reported or observed.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Coal Yard Drainage Basin since the last annual inspection.

- The vegetation on the crests and interior slopes of the drainage basin have been mowed. The bridge to access the pump station was overgrown, and is now accessible (Photos 1 and 4).
- Erosion noted on the east crest and slope appears to have been repaired by filling in erosion rills with coal ash. New erosion has been observed since the repairs were made (Photo 3).
- The inlet pipes from the sediment basin to the drainage basin were reset and covered with approximately two feet of coal ash to bring the east slope to a level grade (Photo 5).
- The sediment basin has been cleaned out and sediment removed from the inlet pipes to allow for water to flow from the sediment basin to the drainage basin (Photos 6-7).
- It appears portions of the coal yard perimeter ditch may have been cleaned out to promote water to drain into the drainage basin.

6. Recommendations

The following maintenance recommendations are offered for the Coal Yard Drainage Basin and Perimeter Ditch. Priority codes are included in parentheses and described in Enclosure M.

- Repair erosions on the interior slope of the east dike. It is recommended that this area be filled/graded with compacted clay and seeded to establish vegetation (Photo 2). (Priority 4)
- Continue to mow the interior slopes and crests of the dikes to maintain access to the pump station. (Priority 3)
- Continue to clean sediment out of the sediment basin to promote runoff to travel from the perimeter ditch to the drainage basin via the sediment basin (Photo 7). (Priority 4)
- Repair the corroded handrails on the pump station. The handrails have deteriorated

in multiple locations (Photo 8). (Priority 4)

- Clean and regrade the perimeter ditches to promote runoff to be channeled to the drainage basin with adequate freeboard during rainfall events. Some locations in the ditch have very little, if any, available freeboard (Photos 9-10). (Priority 4)
- There appears to be high spots in the perimeter ditch. Also, a delta has formed at the convergence of the perimeter ditch and interior ditch which should be cleaned out (Photos 11-13). A hydraulics analysis should be performed to determine ditch capacity and resize the ditch to increase slope in order to obtain adequate self-cleansing flow velocities. (Priority 5)

Enclosure L

Abandoned Ash Disposal Area

TVA 2010 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Abandoned Ash Disposal Area

1. General Facility Information

Facility Status:	Inactive	**NID Identification:**	N/A
Surface Area (inside dikes):	136 acres	**TVA Hazard Classification:**	N/A
Maximum Height (toe to top of dike):	N/A	**Dike Length:**	9,375 feet
Plant Discharge to Facility:	N/A	**Current Pool Elevation:**	N/A

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT and Robert Fuller, PE
TVA Staff Present:	None
Field Inspection Date:	April 15, 2010
Weather/Site Conditions:	Sunny, 84 degrees, dry conditions

3. History/Current and Future Operations

History:	Ash disposal area for Units 1-6 commissioned before 1958. It was decommissioned in June of 1969.
Current Operations:	Inactive
Future Planned Operational Changes:	None

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Dense grassy vegetation.
Trees:	Large trees present throughout the entire ash disposal area.
Wave Wash Protection:	N/A

Erosion:	None observed.
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	N/A
Encroachments:	N/A
Slope:	N/A

4.2. Crest

Crest Cover and Slope:	Dense grass vegetation.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	20 feet

4.3. Exterior Slopes

Vegetation:	Dense grass vegetation.
Trees:	Large trees present throughout entire ash disposal area.
Erosion:	None observed.
Instabilities:	None observed.
Uniform Appearance:	Yes
Seepage:	None observed.
Benches:	None
Foundation Drains, and Seepage Collection Systems:	None observed or reported.
Instrumentation:	None
Animal Burrows:	None observed.

Slope:	N/A
Height:	N/A

4.4. Spillway Weirs/Riser Inlets

Number:	None
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	There are two abandoned spillway weirs/risers noted on the previous yearly inspection report. These were not located during this year inspection due to large brush growth.

4.5. Outlet Pipes

Number:	None
Size, Type and Material:	N/A
Headwall:	N/A
Joint Separations:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	See section 4.4

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The Abandoned Ash Disposal Area has been closed since 1969. There have been no construction activities or repairs since the previous annual inspection.

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6. Recommendations

The following maintenance recommendations are offered for the Abandoned Ash Disposal Area. Priority codes are included in parentheses and described in Enclosure M.

- Continue to mow access roads as needed. (Priority 4)

Enclosure M

Dam Safety Priorities

Dam Safety Priorities

	Description
1	Urgent - Correct Immediately
2	Complete Within 1 Week of Inspection
3	Complete Within 1 Month (30 days) of Inspection
4	Complete Within 6 Months of Original Entry Date
5	Complete Within 1 Year of Original Entry Date
6	Complete Within 3 Years of Original Entry Date
7	Complete Within 5 Years of Original Entry Date
8	Work During Scheduled Outage - Blank Until Outage is Scheduled

Enclosure N

Animal Burrow Guidelines

General Guidelines for Repair of Animal Burrows at TVA Fossil Plants

Identification

Animal burrows are relatively common along slopes of dams and dikes. If left untreated, these burrows can result in the creation of seepage paths through the embankment. Additionally tunnels may eventually collapse resulting in surface irregularities in the embankment. General guidelines for repair of animal burrows are provided below. However, if the burrow extends more than three (3) feet below the embankment surface or extends across a dam, the repair of these features should be evaluated by a geotechnical engineer on a case-by-case basis so that appropriate recommendations can be made.

Guidelines for Burrow Repair

It is recommended that shallow animal burrows (up to 3 feet) shall be repaired with surface treatment methods as follows:

- Animals shall be captured and removed from the area. It is recommended that a local conservation representative be consulted prior to this action.
- The animal burrow shall be excavated and cleaned of excess soil along its pathway up to a depth of 3 feet. With this type of repair, an isolated excavated area of the embankment is exposed.
- The excavated area shall be backfilled with compacted cohesive material.
- If the burrow extends more than three feet into the embankment, a geotechnical engineer shall further evaluate the burrow depth and recommend a deep burrow treatment method or other exploratory methods.
- One possible method which may be recommended to treat a deep burrow can consist of a special grout (flowable fill) pumping system with a hose inserted into the burrow.

Ultimately, these repairs will not prevent rodents from creating new burrows within dam embankments. Accordingly, continual efforts must be made to discourage rodent activity. Mowing of vegetation on the slopes / crest of the embankment and trimming of water-side vegetation at regular intervals will tend to discourage rodents from re-establishing burrows along the dike and will allow timely observation of new activity if it occurs.

Enclosure O

Tree Removal Guidelines

General Guidelines for Tree Removal on Slopes at TVA Fossil Plants

Identification

Trees and heavy brush growth should be controlled on TVA dams and dikes. If left in place, trees can result in the creation of seepage paths within the embankment. Allowing vegetation to become overgrown restricts the level of inspection that can be performed on the structure. General guidelines for removal of trees and maintenance of vegetation are provided below. Evaluations other than those outlined below shall be made by a geotechnical engineer in consultation with facility representatives on a case-by-case basis.

Guidelines for Tree Removal and Maintenance of Vegetation

Tree Removal

At locations where it is not reasonable to remove trees by a mowing them with a bush hog or with similar mowing equipment:

- All trees shall be cut using a handsaw or chainsaw and the cut tree and branches discarded.
- Remove the remaining tree trunk, stump, and rootwad.
- Grub any remaining roots of the tree so that only 2 inches or smaller roots are left in place.
- The resulting cavity from removal of the rootwad shall be cleaned of loose soil and debris.
- The cavity shall then be backfilled with cohesive soil and compacted and the area seeded to re-establish vegetation. If the tree has been removed from along the upstream or downstream face of a slope, benches shall be cut into the slope face where the cavity is to be backfilled. This will allow for a proper bond between the existing dike and the backfill being used to reform the slope. If benches are needed, bench heights shall not exceed 4 to 5 feet in height.

Maintenance of Vegetation

- Mowing is recommended at regular intervals to allow for appropriate inspection of embankment slopes.
- If areas lacking vegetation are observed during mowing and clearing operations or subsequent inspections, the areas should be seeded to re-establish vegetation as soon as practicable.

US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 7

2011 Annual Inspection of CCP Facilities and Ponds, Widows Creek Fossil Plant, Stevenson, Alabama, Stantec Consulting Services, July 27, 2011

2011 Annual Inspection of CCP Facilities and Ponds

Widows Creek Fossil Plant
Stevenson, Alabama

Stantec Consulting Services Inc.
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July 27, 2011 rpt_001_175561035

Mr. Michael S. Turnbow
Tennessee Valley Authority
1101 Market Street
LP 2G-C
Chattanooga, Tennessee 37402

Re: 2011 Annual Inspection of CCP Facilities and Ponds
Widows Creek Fossil Plant
Stevenson, Alabama

Dear Mr. Turnbow:

Stantec Consulting Services Inc. (Stantec) has completed the 2011 annual inspections for CCP facilities and ponds at the Widows Creek Fossil Plant. Facilities reviewed included:

- Main Ash Pond A
- Bottom Ash Stack
- Old Scrubber Sludge Pond (Dredge Cell)
- Upper/Lower Stilling Pond
- Pump Pond
- Chemical Ponds
- Red Water Pond
- Gypsum Stack
- Gypsum Stack Stilling Pond
- Limestone Drainage Basin
- Coal Yard Drainage Basin and Perimeter Ditch
- Abandoned Ash Disposal Area

Beginning this year, field coordinate data is provided for each observed area of concern and an executive summary is added to the report.

The field work was executed on June 13 and 14 , 2011. The results of the work along with facility-specific recommendations for maintenance or other activities are included on the enclosed documents. In addition, the following general plant-wide recommendations and comments area offered:

- Since the last annual inspection, the grounds keeping has vastly improved and allowed inspectors to observe areas of concern to a greater detail than in years past. It is recommended that vegetation maintenance continue, including mowing and clearing tall grass/cattail growth at regular intervals. If lack of vegetation is observed during these operations, re-seeding should be performed as soon as possible. If vegetation establishment difficulties continue in any areas, then TVA should consider refining existing procedures or developing site specific specifications which address topsoil, fertilizing, seed mixtures, etc.

- It is recommended that TVA catalogue, assign a responsible party and due date, and track the completion of the facility-specific recommendations provided herein.

- Please note that this scope did not include a review of the current Operations and Maintenance Manual (O&M) for WCF. Stantec understands that TVA plans to update the current O&M manual to include dam safety-related items. Stantec also understands that TVA plans to develop an Emergency Action Plan for WCF as well.

- It is recommended that TVA personnel continue dike inspections/monitoring to look for changes or conditions that might affect dike integrity. The frequency and procedures for inspections should be consistent with TVA's inspection program. Particular emphasis should be placed on reviewing and monitoring the seepage areas for changed or worsened conditions, and identifying and repairing other maintenance items such as animal burrows, erosion, and lack of vegetation.

Stantec appreciates the opportunity to provide continued engineering services for the fossil plants. 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, PE
Senior Associate

Randy L. Roberts, PE
Principal

Enclosures

Executive Summary

Stantec Consulting Services Inc. (Stantec) in conjunction with TVA RHO&M and TVA Surveying has completed an Annual Inspection of twelve CCP facilities at Widows Creek Fossil Plant (WCF). These CCP containment facilities include: Main Ash Pond A, Bottom Ash Stack, Dredge Cell, Upper/Lower Stilling Pond, Pump Pond, Chemical Ponds, Red Water Pond, Gypsum Stack, Gypsum Stack Stilling Pond, Limestone Drainage Basin, Coal Yard Drainage Basin, and Abandoned Ash Disposal Area. This inspection was performed to evaluate the current conditions of the disposal facilities and to provide recommendations for improvement.

During the inspection and reporting process, each facility was inspected by a team of two to three individuals who walked the perimeter of each facility and recorded the locations of seeps, instabilities, erosion features and other inconsistencies that may affect the stability of the containment system. Once the inspection was complete, the notes, photographs and location coordinates of each item were compiled for each facility and are presented in this report.

The forms that were completed for each facility include a list of deficiencies that were discovered and recommendations for their mitigation. Reference is made to the list of item locations included in Enclosure O of the report. This item list includes the item number, description, point number, picture number, northing, easting, and recommendation. These item numbers are referenced throughout the report and recommendations to clearly identify the area of concern/interest.

In general, it is recommended to continue to mow the interior and exterior slopes and maintain vegetation on all CCP facilities to allow for future inspections and limit woody growth from establishing. Specific deficiencies and recommendations for each facility noted during the annual inspection were compiled and are summarized below.

Main Ash Pond A

- Erosion was noted on interior slopes where there is a lack of vegetation or wave wash protection (Items 19-24). The erosion should be repaired in accordance with the general guidelines provided in Enclosure N (Priority 4).

- There is rutting evident on the 626 bench. All ruts should be filled with compacted clay and seeded to re-establish vegetation (Items 27-28, 70; Priority 4).

- During the inspection, Stantec observed one animal burrow along the northeast exterior slope (Item 30). It is recommended that the burrow be repaired in accordance with the guidelines shown in Enclosure N (Priority 4).

  Future TVA inspections should continue to monitor the activity of burrowing animals, and repairs may need to include filling the burrow/tunnel with flowable fill. A wildlife management expert should be consulted to better control the burrowing problem.

- Continue inspections and monitor locations of known seepage problems. Refer to the Widows Creek Seepage Action Plan which is maintained by the RHO&M Construction Manager for more information.

Bottom Ash Stack

- The south and southeast exterior slopes should be seeded to re-establish vegetation where vegetation is lacking (Items 75-76). This has caused erosion rills to form and bottom ash has been exposed in some locations (Items 77-78). These should be repaired in accordance with the general repair guidelines included in Enclosure N (Priority 4).

- Riprap should be placed below the two outlet pipes on the southwest slope of the stack (Item 36).

- A grate should be placed over the inlet which is missing the grate cover on the west side of the stack (Item 35).

- The corroded steel pipes along the northeast slope of the Bottom Ash Stack should be removed in accordance with an engineered plan to properly mitigate the pipe penetrations through the perimeter dikes (Items 34, 71-74).

- Stantec developed a work plan to repair sloughing and control the seepage (Photos 23-26) along the south and southwest slopes of the stack in 2009. This work plan was submitted to TVA for review. It is recommended that this plan be finalized and implemented to address the red water seepage and associated slope instability along this perimeter segment of the stack (Priority 4).

Old Scrubber Sludge Pond (Dredge Cell)

- Repair the erosion in accordance with the repair guidelines included in Enclosure N (Items 16-18, 81-84; Priority 4).

- Areas lacking vegetation should be re-seeded/fertilized in order to establish vegetation (Items 81-83; Priority 4).

- The trees growing adjacent to discharge pipe from the Gypsum Stack Stilling Pond should be removed in accordance with the guidelines included in Enclosure N (Item 85; Priority 4).

- The pipe installed during the dredging operation of Widows Creek under the 636 bench on the northeast side of the Dredge Cell should be removed (Item 86). It is currently acting as a discharge pipe for the perimeter ditch, which is not the intended purpose (Priority 4).

- There is a dislodged concrete manhole cover on the northeast exterior slope that should be repaired (Item 84; Priority 4).

- Remove the saplings which are growing on top of the Dredge Cell. The excessive vegetation growing on top of the cell can cause issues during closure. These should be removed in accordance with the general guidelines included in Enclosure N (Priority 4).

Upper/Lower Stilling Pond

- Continue to maintain the floating booms (turbidity curtains) across the pond to collect the floating cenospheres. Clean these cenospheres from the pond as needed.

- Repair the wave wash erosion and erosion rills noted on the southwest and northwest interior slopes (Items 25 and 95). Install wave wash protection along interior slopes where necessary in accordance with the guidelines provided in Enclosure N.

- Repair animal burrows near the waterline on the northwest interior slopes in accordance with the guidelines provided in Enclosure N (Item 26; Priority 4).

- Remove the small trees growing along the northeast interior slope (Photos4, 8 and 25), (Items 92-93) and in the skimmer structure (Item 91) in accordance with the tree removal guidelines provided in Enclosure N (Priority 4).

- Seed bare spots on the northeast and northwest interior slopes near the crest of the dike to re-establish vegetation. A new seed/fertilizer mixture may need to be used to enable vegetative growth (Priority 4).

- Repair erosion rills along the northeast interior slope (Item 94; Priority 4).

Pump Pond

- Remove the cluster of trees and root systems in the eastern corner of the pond in accordance with the guidelines provided in Enclosure N (Item 98; Priority 4).

- Two areas of erosion were noted during the inspection. One area was on the interior slope of the northwest dike between the Lower Stilling Pond and Pump Pond (Item 99). The second area of erosion is on the southeast interior slope near the pump station (Item 100). It is recommended that these rills be repaired in accordance with the general guidelines provided in Enclosure N. All unprotected interior slopes should be flattened uniformly with structural fill overlaid by wave wash protection consisting of ALDOT Class I Rip-Rap (Priority 4).

- An apparent slough located on the eastern portion of the northwest dike (Item 101) which was noted in the 2010 annual inspection should be repaired to reconstruct the slopes with structural fill to a final grade of 3:1 or flatter. It is recommended that this dike be evaluated and a design developed to mitigate the risks against future slides/instabilities (Priority 4).

- A culvert installed to direct the flow of storm water from the adjacent low-lying wetlands area into the Pump Pond is not functioning properly. The invert of the culvert was installed at a higher elevation than the water surface of the pool that develops in the engineered wetlands after a rainfall event. A board has been placed across the outlet end of this culvert into the Pump Pond (Item 102). It is recommended that this pipe be redesigned and reconstructed at a proper elevation to fulfill the intended purpose of draining the adjacent area to the pond (Priority 5).

- It appears a 12-inch diameter steel pipe was abandoned in place by placing a blind flange (cover) over the inlet end. It is recommended that this abandoned pipe be identified, excavated, and removed, if found appropriate (Items 103-104; Priority 5).

Chemical Ponds

- A closure plan should be developed to dewater and close the ponds. The pond footprint could be used for the stacking of dry ash.

- Repair the animal burrows noted near the copper pond in accordance with the general repair guidelines provided in Enclosure N (Item 105; Priority 4).

- The hydraulics of the drainage basin south of the chemical ponds need to be better defined. It was unclear during the annual inspection how this area is intended to drain and what purpose it serves. A study should be conducted to determine the effectiveness and purpose of this drainage area (Item 106).

Red Water Pond

- The uprooted tree along the west interior dike should be removed and the depression filled with compacted clay (Item 37; Priority 4).

- The inlet pipes along the west interior slope below the railroad tracks should be cleared of vegetation and the erosion/scouring should be repaired (Items 38-39; Priority 4).

- Remove the small trees and root systems along the north dike interior slope in accordance with the guidelines provided in Enclosure N (Item 107; Priority 4).

- Corroded pipes penetrating the perimeter dikes located on the 620 bench were observed and have previously been recommended to be abandoned by URS as noted in the 2011 second quarter dike inspection report (Items 34, 71-74; Priority 4).

- Areas lacking vegetation were noticed on the east interior slope and the west end of the north dike crest. It is recommended that these locations be seeded to re-establish vegetation (Priority 4).

- Cracking was observed in the asphalt roadway and the guard rail is dipping along the crest of the west dike (Items 40-42). The interior slope below the road should be monitored for any potential sloughs that could be causing the loss of pavement support (Priority 4).

- Continue to monitor the previously identified seepage area (Seep 7) located at the toe of the north dike (Priority 3).

- The inlet pipes to the south end of the Red Water Pond from under the roadway should be inspected and cleaned of debris (Items 43-44; Priority 4).

- A fallen tree obstructing access to a piezometer along the shoulder of the 626 bench access road should be removed (Item 33; Priority 3).

Gypsum Stack

- It is recommended that all instrumentation be protected with bollards and steel casing. It was Stantec's understanding that this would occur during the instrumentation automation project. Depending on the status of that project, it may

be necessary to install protective bollards and casing prior to the automation similar to that shown in Photo 52 (Priority 4).

- Erosion should be repaired in accordance with the general repair guidelines included in Enclosure N (Items 1, 2, 5-8, 10 and 14). On the west dike, the silt fence should be reinstalled once erosion repairs are completed to prevent gypsum from washing down the exterior slope (Items 6-7; Priority 4).

- The ditch along the access road on the south side of the stack should be improved/maintained. Currently, there is erosion along the interior groin where the road and exterior slope are joined due to the lack of a defined roadside ditch (Item 112; Priority 4).

- The damaged observation well should be repaired in the same manner as other observation wells. The pipe should be extended and a new yellow cover should be placed on top of the well (Item 3; Priority 4).

- Seep 5 should be investigated in the field. There are similar locations around the stack at the same elevation of Seep 5 which has an 8" diameter HDPE outlet pipe with water flowing from the pipe into the perimeter ditch. The area around Seep 5 should be carefully excavated/probed to determine if a buried pipe is located in this area (Priority 3).

- The two new seeps identified during the 3rd quarter inspection (Photos 25-26; Items 9 and 13) should be monitored for any changes in flow or sediment transport. Previously identified seeps should also be monitored in accordance with the Seepage Action Plan.

Gypsum Stack Stilling Pond

- Seed the locations on the interior slope of the west dike to re-establish vegetation. A new fertilizer/seed mixture may need to be developed if vegetation is unable to be established in this area (Priority 4).

- Continue to practice good cenosphere management by continuing to use floating booms (turbidity curtains) across the pond and cleaning the floating cenospheres from the pond surface as required.

- Repair the erosion rills noted on the interior slopes (Items 11, 114-116; Priority 4).

- The planned emergency overflow notch is located in the northwest end of the Stilling Pond. It is recommended that the notch and downstream slope and channel be re-established using crushed stone lining (Priority 5).

Limestone Drainage Basin

- Remove the debris floating in the Limestone Runoff Surge Pond (Priority 5)

- Clean out the trench running from the limestone stockpile to the Limestone Runoff Pond which has filled in with sediment (Item 46; Priority 5).

- Repair the damaged 48-inch diameter corrugated metal pipe in the southern corner of the Limestone Runoff Pond which was noted in the 2010 annual inspection. It

appears that a hole developed in the top of the pipe, causing the material above the pipe to collapse into the pipe, blocking flow (Item 47). Remove sedimentation and vegetation blocking the inlet end of this pipe and protect inlet with rip-rap (Item 48; Priority 4).

- Two localized areas of erosion were noted during the inspection on the western dike of the Limestone Runoff Pond (Item 45). It is recommended that these be filled/graded. Also, smaller rills were noted along the entire interior slopes. These should also be filled and graded (Priority 4).

- Regrade the interior slopes of the Limestone Runoff Pond to 3H:1V (Priority 4).

- Clean and remove sediment from the Limestone Runoff Pond (Priority 5).

- Remove debris from the Limestone Runoff Surge Pond (Priority 5)

- Seepage from the southeast dike of the Limestone Runoff Surge Pond has been reported for several years. A remedial design should be developed and implemented to repair this dike (Priority 5).

- Conduct vegetation maintenance (mowing) activities along the interior slopes of the Limestone Runoff Pond and Limestone Runoff Surge Pond.

- A H&H engineering analysis of the Limestone Drainage Basin should be conducted to provide engineering recommendations for the improvement of the hydraulics of the basin. It was Stantec's understanding that Worley Parsons was tasked to complete this study as of the 2010 annual inspection, but the status of the study is unknown at this time.

Coal Yard Drainage Basin and Perimeter Ditch

- Repair erosion rills on the interior slope of the east dike. It is recommended that this area be filled and graded with compacted clay and seeded to establish vegetation (Items 50-51; Priority 4).

- Mow the interior slopes and crests of the dikes to maintain access to the pump station (Priority 3).

- Clean sediment out of the sediment basin and perimeter ditch to promote runoff to travel from the perimeter ditch to the drainage basin via the sediment basin (Priority 4).

- Repair the corroded handrails on the pump station as noted in the 2010 annual inspection report. The handrails have deteriorated in multiple locations (Priority 4).

- Clean and regrade the perimeter ditches to promote runoff to be directed to the drainage basin with adequate freeboard during rainfall events. Some locations in the ditch have very little, if any, available freeboard (Priority 4).

- There appears to be high spots in the perimeter ditch. Also, coal fines have sloughed into the perimeter ditch and formed a delta which should be cleaned out (Items 52). A hydraulics analysis should be performed to determine ditch capacity and resize the

ditch to increase slope in order to obtain adequate self-cleansing flow velocities (Priority 5).

Abandoned Ash Disposal Area

- No recommendations.

Enclosure F

Chemical Pond

Enclosure G

Red Water Pond

Enclosure H

Gypsum Stack

Enclosure I

Gypsum Stack Stilling Pond

Enclosure J

Limestone Drainage Basin

Enclosure K

Coal Yard

Enclosure L

Abandoned Ash Disposal

Enclosure M

Dam Safety Priorities

Enclosure N

General Repair Guidelines

Enclosure O

Items

Enclosure A

Main Ash Pond A

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	160 acres	**TVA Hazard Classification:**	Significant
Maximum Height (toe to top of dike):	53 feet	**Dike Length:**	11,590 feet
Plant Discharge to Facility:	77.4 cfs	**Current Pool Elevation:**	626.9 feet

2. Site Visit Information

Stantec Inspection Team: Chris Jones, EIT, Robert Fuller, PE, Briggs Evans, and Greg Budd

TVA Staff Present: James Mullins

Field Inspection Date: June 13, 2011

Weather/Site Conditions: Sunny, 85 degrees, dry conditions

3. History/Current and Future Operations

History: The area is first shown on TVA Drawing 10N7400-R5 dated 1959, labeled as New Ash Disposal Area. The perimeter dikes were constructed in two dike raisings. The initial dike was constructed of rolled earth fill to an elevation of 626 feet. A second rolled earth dike was constructed above the initial dike sometime before 1973, bringing the top of the perimeter dike to its current elevation of 636 feet.

Current Operations: The Main Ash Pond A receives both fly ash and bottom ash from two waste streams. The fly ash and bottom ash from Units 1-6 are sluiced to one (eastern) ditch, while the bottom ash from Units 7-8 are sluiced to a separate (western) ditch. The outfall from the sluice lines is located in the Bottom Ash Stack. The ditches flow through the Bottom Ash Stack and converge into one ditch which flows into the northern corner of the Main Ash Pond A. In the southwest corner of the Main Ash Pond A, six siphons transport water to the Upper Stilling Pond. The five existing spillway decant structures are still in

	place, but a new siphon system is currently used to control the pool elevation in the Main Ash Pond A, which is being maintained at about five feet below normal pool.
Future Planned Operational Changes:	URS Corporation is charged with designing the spillway replacements and closure plan for the Main Ash Pond A.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Sparse vegetation noted on the northeast and southeast slopes. Bottom ash covering interior slope where vegetation is lacking (Photos 1-3).
Trees:	None observed.
Wave Wash Protection:	None observed.
Erosion:	Erosion rills on the southeast interior slopes between the Dredge Cell/Main Ash Pond A Divider Dike where vegetation is lacking (Photos 2-8; Items 19-24). Erosion rills along northwest interior slopes of the rim ditch (Photos 9-11).
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	9 feet (pool currently controlled by siphon system)
Encroachments:	None observed.
Slope:	2H:1V

4.2. Crest

Crest Cover and Slope:	Crushed stone coverage, relatively level along the northwest and northeast crests (Photo 12). Crushed stone from the northwest and northeast crest corners extending down to the toe of the exterior slope for haul truck access during the rock buttress construction work (Photo 13).
Erosion:	None observed.

Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	20 feet.

4.3. Exterior Slopes

Vegetation:	Dense grass vegetation along most exterior slopes. Some areas of sparse coverage on the northwest slope between the Red Water Pond and Main Ash Pond A and on the northeast slope (Photos 14-15). Sparse vegetation at the toe of the northeast exterior slope (Photo 16).
Trees:	Large trees present at the toe of the northwest slope north of the Red Water Pond and at the toe of the northeast exterior slope along the edge of Widows Creek (Photos 16-17).
Erosion:	Erosion/possible slough due to a backhoe bucket was noted on the northwest exterior slope (Photos 18-19; Items 29 and 31). Cracking was noted along the north exterior slope (Photo 20; Items 27-28). There is also rutting evident on the 626 bench (Photo 21). Erosion was noted at the toe of the northwest slope (Photo 22; Item 32).
Instabilities:	None observed.
Uniform Appearance:	The Main Ash Pond A is adjacent to the Dredge Cell along the southeast side and the Bottom Ash Stack along the southwest side, with no exterior slopes on these segments. There is a generally uniform appearance on the northeast and northwest slopes.
Seepage:	A seepage area was observed on the northwest exterior slope during the 2010 Annual Inspection has been repaired and is identified as Seep 18 (Photo 23).
Benches:	Approximately ten feet wide bench is located along the northwest and northeast exterior slopes at elevation 626 feet. The bench varies in width in some locations, with little evidence of a bench on the northern portion of the

	northeast slope.
Foundation Drains, and Seepage Collection Systems:	None observed.
Instrumentation:	Fifteen piezometers were installed by Stantec in 2009 and 2010 as part of the dike slope stability and seepage evaluation. Piezometer readings are being taken on a monthly basis by URS.
Animal Burrows:	One new animal burrow was noted along the northeast exterior slope (Photo 24; Item 30). One animal burrow that has been repaired was noted along the northeast exterior slope (Photo 25).
Slope:	2H: 1V
Height:	51 feet

4.4. Spillway Weirs/Riser Inlets

Number:	The five 4-foot diameter concrete manholes with 10-foot CMP skimmers are no longer in service. The pool elevation of the Main Ash Pond A is controlled by the siphon system which has been installed since the 2010 annual inspection (Photos 26-27). The old spillways have not been abandoned at this time, but are no longer in use. The siphons are discussed in Section 4.5 and 5.
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	Four wooden skimmers are located on the southeast side of the Main Ash Pond A. These appear to be abandoned spillways (Photo 31). For a complete inventory of existing conduits, reference is made to the WCF Pipe Inventory Report completed in April, 2010.

4.5. Outlet Pipes

Number:	Six (6)
Size, Type and Material:	12 inch diameter steel siphons with floats (Photo 27)
Headwall:	None.
Joint Separations:	None observed.
Mis-Alignment:	None observed.
Closed/Abandoned Conduits:	None reported or observed. Stantec conducted a pipe inventory of the stack to locate existing inactivate spillways.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Main Ash Pond A since the last annual inspection.

- The exterior slopes have been mowed.
- The pool elevation of the Main Ash Pond A has been lowered approximately five feet. Material was dredged from the interior of the Main Ash Pond A in order to create a channel for water to flow to the spillway/siphon system once the pool was lowered (Photo 1).
- A series of six siphons have been installed to transport water from the Main Ash Pond A to the Upper Stilling Pond (Photo 27; Item 96). The existing spillway system is no longer in use. Pool elevation for the Main Ash Pond A is controlled by the new siphon system.
- A rock buttress has been constructed along the northeast exterior slope (Photos 28-29; Items 119-120). Approximately 1,700 linear feet has been installed from the bridge which connects the Main Ash Pond Complex to the Gypsum Stack to the northern corner of the Main Ash Pond A. The buttress consists of a layer of sand, small stone, and large stone. This was an Immediate Risk Reduction Plan (IRRP) designed by URS.
- A seepage blanket/rock buttress has been installed on the northwest exterior slope (Photo 30; Items 108-109, 121). Approximately 3,100 linear feet has been installed from the Red Water Pond pump station to the north dike of the red water pond. From there, it extends along the northeast exterior slope of Main Ash Pond A.
- Riprap has been placed along the southeast interior slope of the Main Ash Pond A

(Photo 31; Items 122-123). The riprap was placed for an approximate length of 100 feet.

- A rock check dam was constructed across the rim ditch on the northwest side of the pond (Photo 10; Item 124). The rock check dam is used to control the pool elevation in the upstream portion of the rim ditch since the pool elevation of the Main Ash Pond A was lowered.
- Seep 18 on the northeast exterior slope was repaired in conjunction with the construction of an access road to allow trucks to access the toe of the slope during construction of the rock toe buttress (Photos 23 and 28).
- Animal burrows noted during the 2010 annual inspection have been repaired (Photo 25)
- Signage has been installed at the Main Ash Pond A. This includes traffic control signs as well as signs marking the location of known seepage areas.

6. Recommendations

URS Corporation is moving forward with design of pond modifications to reduce the hazard classification, improve operational conditions, and to ultimately achieve closure in FY 2015-2016. These design efforts will address slope stability, seepage, and spillway issues previously identified in Phase I and Phase II reports. The capital projects under design/construction include rock toe buttressing and spillway replacements.

Therefore, the recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design and operation issues currently being addressed by URS. TVA Dam Safety priority codes are included in parentheses and described in Enclosure M.

- Continue to mow the exterior slopes as needed.
- Erosion was noted on interior slopes where there is a lack of vegetation or wave wash protection (Photos 2-8; Items 19-24). The erosion should be repaired in accordance with the general guidelines provided in Enclosure N (Priority 4).
- There is rutting evident on the 626 bench. All ruts should be filled with compacted clay and seeded to re-establish vegetation (Photos 14, 20, and 21; Items 27-28, 70; Priority 4).
- During the inspection, Stantec observed one animal burrow along the northeast exterior slope (Photo 24; Item 30). It is recommended that the burrow be repaired in accordance with the guidelines shown in Enclosure N (Priority 4).
- Future TVA inspections should continue to monitor the activity of burrowing animals, and repairs may need to include filling the burrow/tunnel with flowable fill. A wildlife

management expert should be consulted to better control the burrowing problem.

- Continue inspections and monitor locations of known seepage problems. Refer to the Widows Creek Seepage Action Plan which is maintained by the RHO&M Construction Manager for more information.

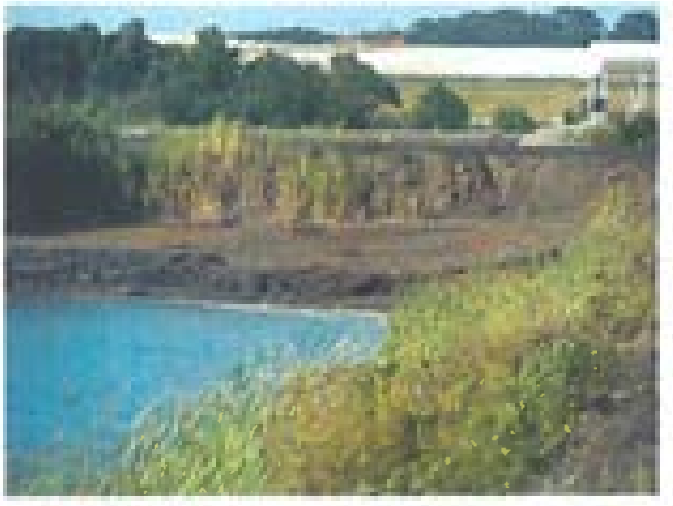


Photo 1

Northeast corner Interior slope. Note the bottom ash where vegetation is not present and the lower pool level controlled by new siphon system.

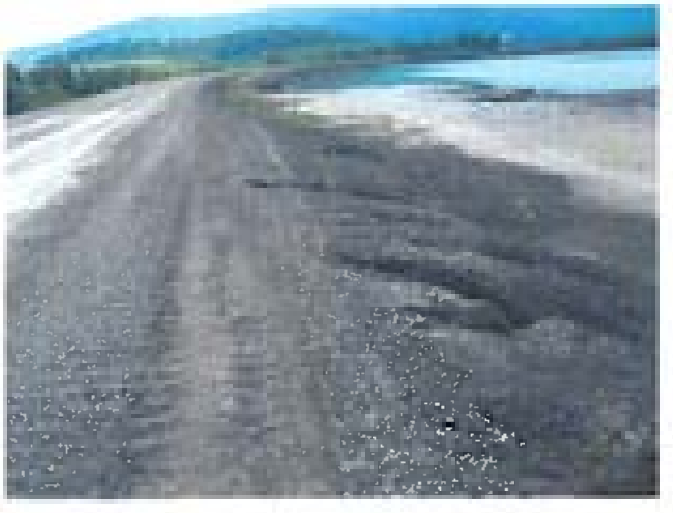


Photo 2

Northeast interior slope typical erosion rills along roadway and lowered pool elevation.



Photo 3

Typical erosion rills along the southeast interior slope between the Main Ash Pond A and the Dredge Cell (Item 19).



Photo 4

Cracking on interior slope of the divider dike (Item 20).

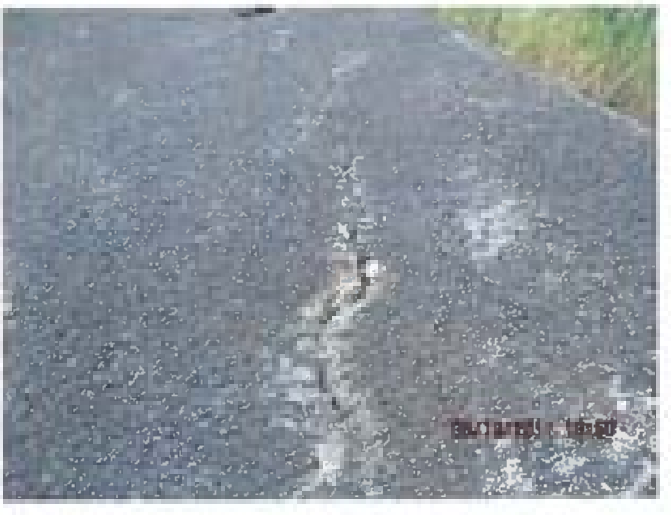


Photo 5

Cracking on the interior slope of the divider dike (Item 21).



Photo 6

Erosion around discharge pipe between Main Ash Pond A and the Dredge Cell under the road (Item 22).



Photo 7

Erosion on interior slope between the southwest edge of the Dredge Cell and Main Ash Pond A (Item 23).



Photo 8

Erosion on interior slope between the southwest edge of the Dredge Cell and Main Ash Pond A (Item 24).



Photo 9

Current rim ditch along the northwest side of the Main Ash Pond A. The rim ditch has been moved inward towards the pond.



Photo 10

Current rim ditch along the northwest side of the Main Ash Pond A. Note the rock check dam installed across the rim ditch to maintain water elevation as a result of lowering the Main Ash pond pool elevation approximately 5 feet.



Photo 11

Current rim ditch at the northwest side of the Main Ash Pond A.



Photo 12

Northwest crest, looking southwest, covered in crushed stone. Note the water and cattails along the interior slope of the old rim ditch.

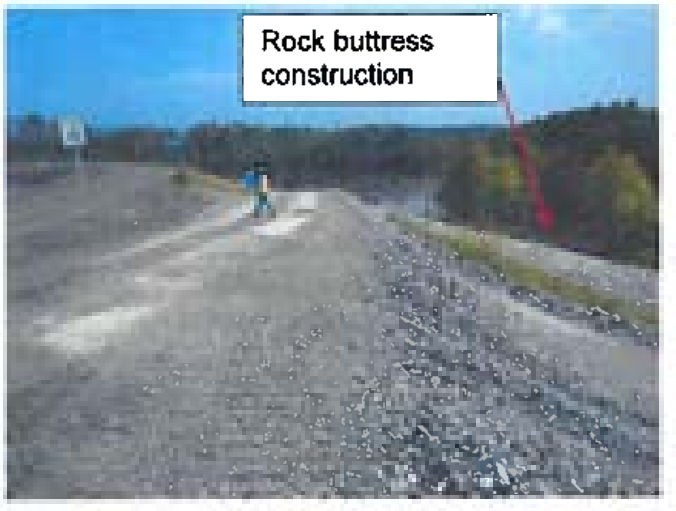


Photo 13

Northwest corner exterior slope with crushed stone coverage and the ramp to the toe of the exterior slope where rock buttress construction activities are ongoing.



Photo 14

Northwest exterior slope adjacent to the Red Water Pond. Note the adequate grass vegetation and the bare spots along the tire ruts.



Photo 15

Grass vegetation and rock buttress construction along the toe of the northwest slope north of the Red Water Pond.



Photo 16

Grass vegetation and rock buttress construction at the corner of the northeast and northwest slopes. Note the tree damage from April 27, 2011 tornado event.



Photo 17

Large trees damaged from April 27, 2011 tornado at the toe of the northwest slope along the edge of Widows Creek.



Photo 18

Erosion from a backhoe bucket on the north dike exterior slope (Item 29).



Photo 19

Erosion from a backhoe bucket on the north dike exterior slope (Item 31).



Photo 20

Slough/cracking on the northwest dike exterior slope (Items 27-28)



Photo 21

626 bench on the northeast slope, looking northwest. Note the cracking along the bench.



Photo 22

Erosion on the northeast exterior slope (Item 32).



Photo 23

Seepage area observed on the northwest exterior slope during the 2010 Annual Inspection. Seep has been repaired and is identified as Seep 18. Note the rock buttress construction at the toe of the slope along Widows Creek.



Photo 24

Animal burrow observed along the northeast exterior slope (Item 30).



Photo 25

Typical animal burrow repair. The animal burrow was previously identified during the 2010 Annual Inspection.

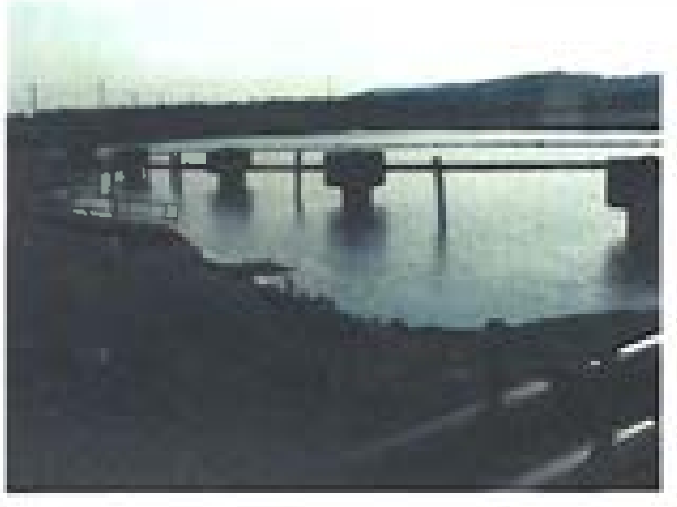


Photo 26

Spillways located in the southern portion of the Main Ash Pond A. These spillways transport storm and process water to the Upper Stilling Pond.



Photo 27

New siphon discharge pipes form the Main Ash Pond A to the Upper Stilling Pond.



Photo 28

View of rock buttress construction at the toe of the northeast exterior dike along Widows creek. Note the access road which was constructed for the construction of the rock buttress.

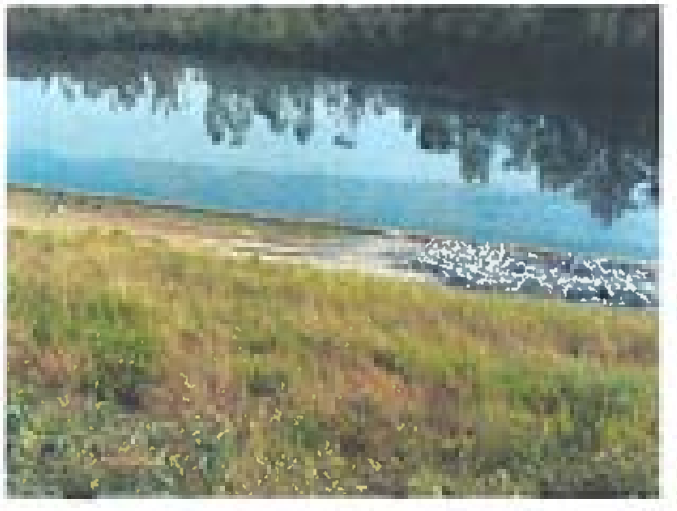


Photo 29

View of rock buttress construction at the toe of the northeast exterior dike along Widows creek.

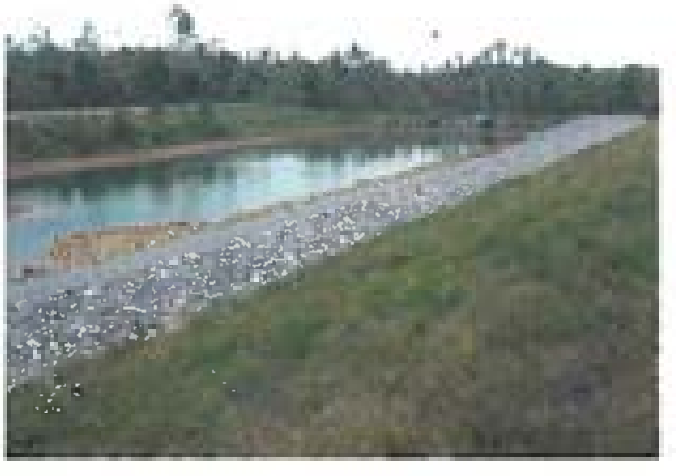


Photo 30

View of rock buttress construction at the toe of the northwest exterior dike along the Red Water Pond.



Photo 31

View of northeast interior slope rock buttress construction.



Photo 32

Wooden skimmers along the southeast edge of the pond. These appear to be abandoned.

N
Main Ash Pond A
Feet
0
400
800
Stantec
Plan View - Main Ash Pond A
WCF 2015 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
1 OF 1

Enclosure B

Bottom Ash Stack

TVA 2011 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Bottom Ash Stack

1. General Facility Information

Facility Status: Active

Surface Area: 40 Acres

Maximum Height (toe to top of stack): 45 feet

2. Site Visit Information

Stantec Inspection Team: Chris Jones, EIT and Robert Fuller, PE

TVA Staff Present: James Mullins

Field Inspection Date: June 13, 2011

Weather/Site Conditions: Sunny, 85 degrees, dry conditions

3. History/Current and Future Operations

History: Commissioned pre-1967 to provide a stacking area for Bottom Ash.

Current Operations: The Bottom Ash Stack is currently being operated following the Trans Ash three-year stacking plan. This plan consists of stacking ash and maintaining the fly ash ditch and bottom ash ditch which flow through the stack to the Main Ash Pond A.

Future Planned Operational Changes: None at this time.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Exterior Slopes and Benches

Vegetation: Sparse grass vegetation is located along the northwest slope (Photo 1). The south and southwest slopes have grass vegetation with adequate coverage in most locations. However, some locations have sparse/stressed vegetation (Photos 2-4), (Items 75-77). Dense grass vegetation is located on the south slope next to the sluice lines (Photo 5). Vegetation lacking along the interior slopes of the perimeter ditch (Photo 6) and near the top of the exterior slope on the southwest slope (Photo 7). The 615 elevation bench on the northwest slope of the Bottom Ash Stack has adequate vegetation (Photo 8). The

	626 bench on the northwest slope is covered in bottom ash/crushed stone (Photo 9).
Trees:	Trees located on the northwest slope below the 626 bench near the Red Water Pond (Photo 10).
Erosion:	Some areas of localized erosion rills/gullies noted throughout, specifically where vegetation is lacking (Photos 11-12; Items 77-78). Erosion was also noted due to pipe outfall where rip-rap is not present along the south slope (Photo 13; Item 36).
Instabilities:	None observed.
Uniform Appearance	The northwest slope appears to be uniform. The south slope varies (Photo 14).
Benches:	Benches are only located on the northwest slope. There is a 10 feet wide bench at elevation 615 feet, 45 feet wide bench at elevation 626 feet, and 20 feet wide bench at elevation 636 feet (Photos 1, 9, 15-16).
Slope:	On the northwest slope, 2H:1V below the 615 bench, 2.5H:1V to 3H:1V between the 615 and 626 bench, and 3H:1V between the 626 bench and 636 perimeter road and above the 636 perimeter road where material has been placed. The slope along the scrubber road on the south and southeast dikes is not uniform, with a slope ranging from 2H:1V where the sluice lines run up the slope to the bottom ash ditch to 4H:1 along the southeast slope. Slopes were measured from a 2009 TVA survey.
Height:	Approximately 45 feet, measured at the northwest side of the stack above the Red Water Pond.
Other:	There are five, inactive, corroded, steel pipes penetrating the exterior slopes below the 626 and 615 feet benches (Photos 17-20; Items 34, 71-74).
	Seepage has been a recurring issue along the scrubber road at the toe of the Bottom Ash Stack. The extent of seepage ranges from saturated slopes to flowing seepage at the southern corner of scrubber road. Based on the field observations, it appears the source of the seepage is from the Chemical Ponds located in the footprint of the stack and from the bottom ash ditch and fly ash ditch which transport the slurry through the Bottom Ash Stack to the Main Ash Pond A (Photos 21-24).

4.2. Perimeter Drainage Ditches and Down-Drains

Vegetation:	Vegetation is present in the bottom of the perimeter ditch in some locations where channel lining is lacking (Photo 24).
Rip-Rap Channel Lining:	Channel lining is present north of the Units 7/8 sluice lines to the south corner of scrubber road. The channel lining is placed above the ditch bottom on the slopes of the ditch (Photos 25-27).
Erosion:	None observed.
Sedimentation in Ditches:	No major issues observed.
Standing Water in Ditches or on Benches:	No standing water observed on the benches. Standing water is present in the ditches around the entire perimeter of the stack (Photos 28-29).
Silted/Impeded Drainage Pipes:	No issues observed.
Other:	An inlet missing a grate was identified (Photo 30; Item 35).

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Bottom Ash Stack since the last annual inspection.

- The exterior slopes of the stack have been mowed.
- It appears the area just south of the sluice lines at the three-way intersection was excavated and stone was placed along the entire face of the exterior slope (Photo 28; Item 79).
- Bottom ash has been used to build up the bottom ash stack between the chemical ponds and the bottom ash/fly ash sluice ditches (Photo 31; Item 80).

6. Recommendations

URS Corporation is moving forward with a closure design for the Bottom Ash Stack. These design efforts will address slope stability and seepage issues previously identified. Therefore, the recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design issues currently being addressed by URS. Priority codes are included in

parentheses and described in Enclosure M.

- Continue to mow the exterior slopes of the stack as needed.
- The south and southeast exterior slopes should be seeded to re-establish vegetation where vegetation is lacking (Photos 2-4; Items 75-76). This has caused erosion rills to form and bottom ash has been exposed in some locations (Photos 11-12; Items 77-78). These should be repaired in accordance with the general repair guidelines included in Enclosure N. (Priority 4)
- Riprap should be placed below the two outlet pipes on the southwest slope of the stack (Photo 13; Item 36).
- A grate should be placed over the inlet which is missing the grate cover on the west side of the stack (Photo 30; Item 35).
- The corroded steel pipes along the northeast slope of the Bottom Ash Stack should be removed in accordance with an engineered plan to properly mitigate the pipe penetrations through the perimeter dikes (Photos 17-20; Items 34, 71-74).
- Stantec developed a work plan to repair sloughing and control the seepage (Photos 23-26) along the south and southwest slopes of the stack in 2009. This work plan was submitted to TVA for review. It is recommended that this plan be finalized and implemented to address the red water seepage and associated slope instability along this perimeter segment of the stack. (Priority 4)



Photo 1

Sparse grass vegetation along the northwest exterior slope between the stack and the Red Water Pond.

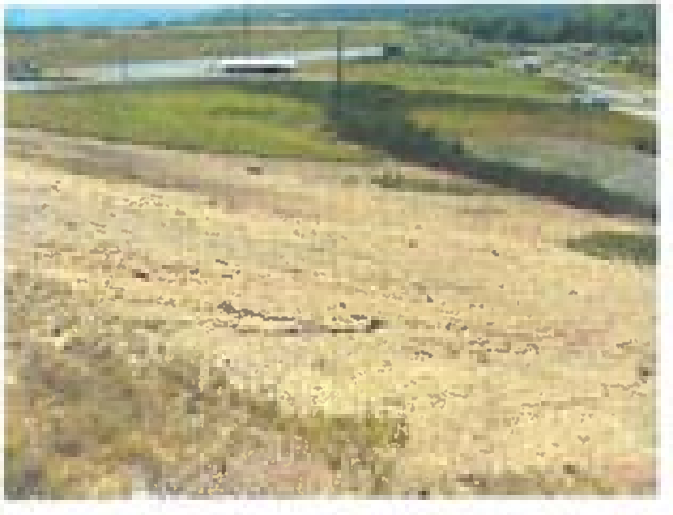


Photo 2

Sparse vegetation along the southeast slope of the stack. Note the recent seeding and erosion control cover material (Items 75-76).

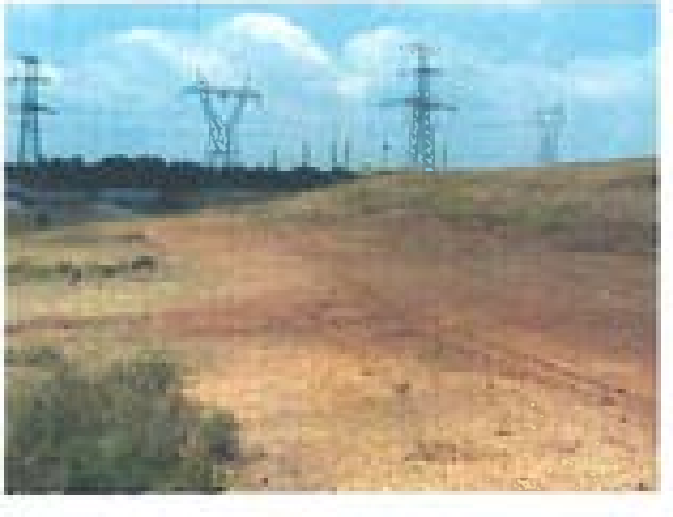


Photo 3

Sparse vegetation along the south slope of the stack from red water seepage. Note the recent seeding and erosion control cover material (Items 75-76).



Photo 4

Toe of the south slope lacking vegetation looking west (Items 75-76).



Photo 5

Dense grass vegetation on the south slope near the sluice lines for Units 7/8.



Photo 6

Sparse vegetation in the area of the red water seepage on the south slope. Note sediment deposition in the perimeter ditch (Item 78).



Photo 7

Lack of vegetation on the southwest slope above the perimeter ditch. Note the erosion rills (Item 78).



Photo 8

Southwest exterior slope and perimeter ditch to the Red Water Pond. Note vegetation coverage along slope.



Photo 9

The 626 bench with bottom ash access road and the 636 bench with crushed stone perimeter road on the northwest side of the stack.

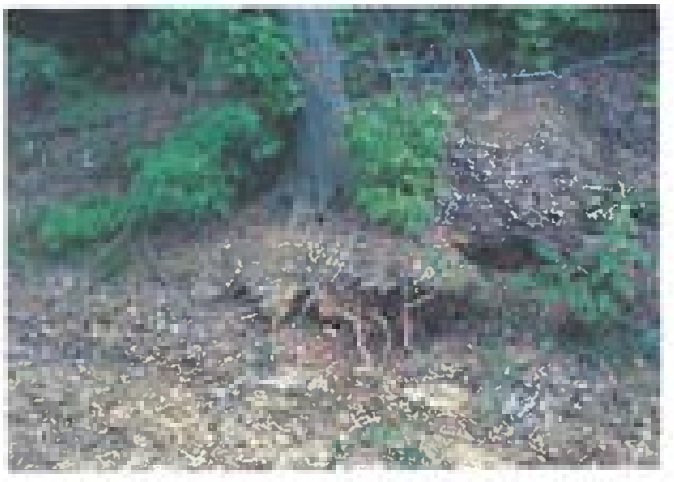


Photo 10

615 bench on the northwest slope, looking southwest. Typical large tree growth in this area. Note the erosion at the base of tree undercutting the roots.



Photo 11

Surface erosion due to lack of vegetation on the south slope of the stack. Note the exposed bottom ash (Item 77).



Photo 12

Surface erosion rills due to lack of vegetation on the south slope of the stack (Item 78).



Photo 13

Erosion due to pipe outfall and lack of channel lining on the south exterior slope (Item 36).

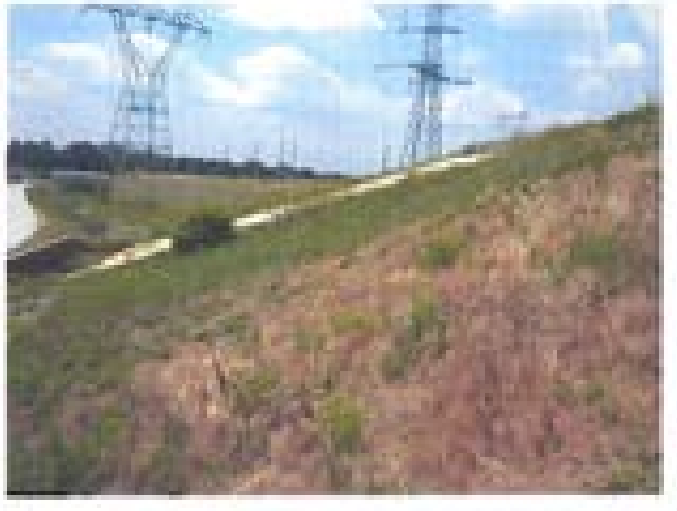


Photo 14

View of the south slope, looking west. Note the non-uniform appearance of the slope, with slopes varying from 2H:1V to 4H:1V.



Photo 15

View of the 626 bench with the bottom ash and crushed stone access road.



Photo 16

View of the 636 bench with the crushed stone perimeter road on the northwest slope.



Photo 17

View of corroded steel pipe penetrating the 626 feet bench (Item 72).

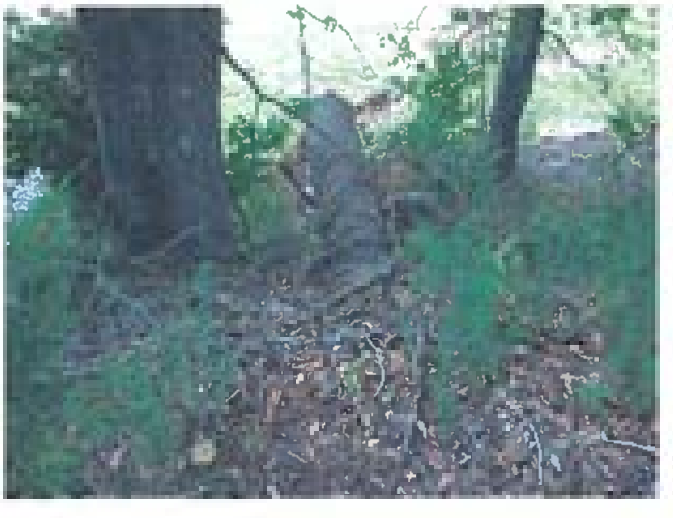


Photo 18

View of corroded steel pipe penetrating the 615 feet bench (Item 73).



Photo 19

View of corroded steel pipe penetrating the 615 feet bench (Item 34).



Photo 20

View of corroded steel pipe penetrating the 615 feet bench (Item 71).

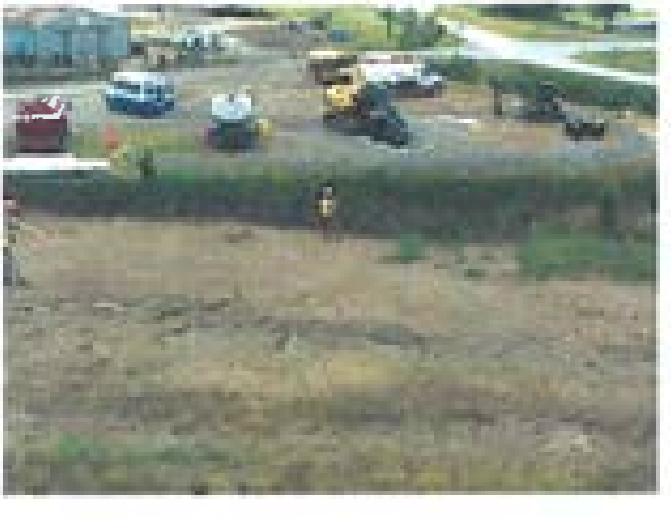


Photo 21

Red water seepage on the south slope. Note the cattail growth in this area.



Photo 22

Red water seepage at the toe of the southwest slope that has recently been seeded.

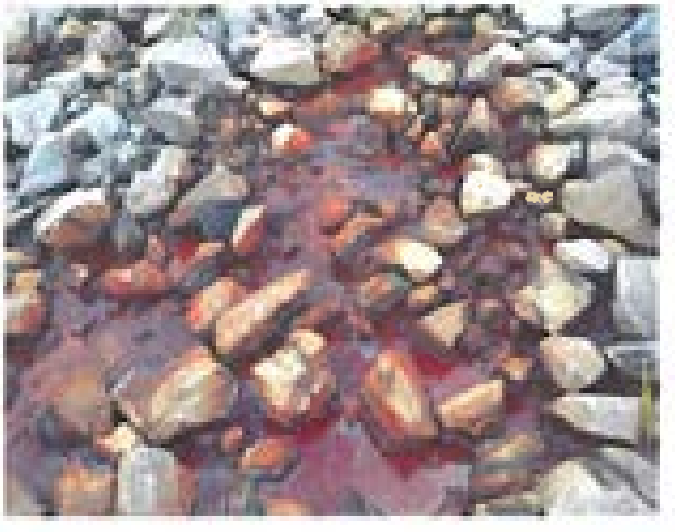


Photo 23

Flowing red water seepage at the toe of the slope near piezometer STN-86.



Photo 24

Flowing red water seepage at the toe of the south portion of the northwest slope.



Photo 25

Perimeter ditch along the south side of the stack. Note the vegetation growing in the ditch and the channel lining placed on the slope above the ditch.



Photo 26

Perimeter ditch along the south side of the stack. Note the channel lining which has been stained by red water flow. Also note the vegetation in the ditch.



Photo 27

Perimeter ditch along the south side of the stack near STN-85/86. Note the culvert installed under the access pad to the instruments.



Photo 28

Units 1-6 sluice lines crossing over the perimeter ditch on the southwest side of the stack. Note the standing water in the perimeter ditch (Item 79).



Photo 29

Perimeter ditch along the southwest side of the stack. Note the relatively steep interior slopes of the ditch and erosion along the slopes.



Photo 30

Inlet with a missing grate cover (Item 35).



Photo 31

Bottom ash ditch sluice channel near the sluice line outfall. Note the stockpiled material near the ditch (Item 80).

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Bottom Ash Stack
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Stantec
Plan View - Bottom Ash Stack
WCF 2011 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
1 OF 1

Enclosure C

Dredge Cell

TVA 2011 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Dredge Cell (DRAFT)

1. General Facility Information

Facility Status:	Inactive	**NID Identification:**	N/A
Surface Area (inside dikes):	116 Acres	**TVA Hazard Classification:**	Significant
Maximum Height (toe to top of dike):	53 feet	**Dike Length:**	8,622 feet
Plant Discharge to Facility:	0 cfs	**Current Pool Elevation:**	N/A

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT, Robert Fuller, PE, Briggs Evans, and Greg Budd
TVA Staff Present:	James Mullins
Field Inspection Date:	June 13, 2010
Weather/Site Conditions:	Sunny, 85 degrees, dry conditions

3. History/Current and Future Operations

History:

The Dredge Cell is also known as the Old Scrubber Sludge Pond. The initial dike was constructed of rolled earth fill to an elevation of 626 feet. A second rolled earth dike was constructed above the initial dike sometime before 1973, bringing the top of dike elevation to 636 feet. The gypsum/scrubber sludge product was discharged into the southwest corner, south of the deflector dike running from west to east across the central part of the pond. Once TVA switched to a Forced Oxidation Gypsum Scrubber in the mid-1980's, a new gypsum stack disposal area was created on the west side of Widows Creek. After the Gypsum Stack was brought into service in 1986, the Scrubber Sludge Pond was used as a dredge cell for the Main Ash Pond A. In the mid 2000's the perimeter dikes were raised to their current elevation of 646 feet using Bottom Ash fill. In 2007, plans were developed to again raise the dikes around this area, as the remaining capacity would be exhausted by 2009. This dike raising was not constructed.

Current Operations:	The Dredge Cell is currently inactive and being allowed to dewater. An area on the northwest edge on top of the dredge cell is currently being used as a material staging area for the immediate risk reduction plans being constructed around the Main Ash Pond Complex. In preparation for closure, dredged material from the Main Ash Pond A which was dewatered at the Bottom Ash Stack had been placed on top of the Dredge Cell in various locations since the 2010 annual inspection. This operation is not currently ongoing.
Future Planned Operational Changes:	Based on discussions with TVA, URS Corporation is charged with designing a closure plan for the Dredge Cell.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	None observed.
Trees:	None observed.
Wave Wash Protection:	None observed.
Erosion:	The interior slopes of the Dredge Cell are constructed of bottom ash material. Small erosion rills from surface water runoff noted along the interior slopes of the entire perimeter (Photos 1-3).
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	N/A
Encroachments:	None observed.
Slope:	2H:1V

4.2. Crest

Crest Cover and Slope:	The crest of the Dredge Cell is covered in bottom ash/crushed stone (Photos 4-5). The crest appears relatively flat on the northeast, southeast, and southwest sides of the cell, with the northwest crest sloping away

	from the Dredge Cell towards the Main Ash Pond A.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	30 feet

4.3. Exterior Slopes

Vegetation:	The exterior slope along the west dike, above the bench at elevation 636 feet, is comprised of bottom ash that is sparsely vegetated and exhibits erosion rills throughout (Photo 6). Dense grass vegetation is present on the exterior slopes below the bench at elevation 636 feet, with locations of sparse/stressed vegetation along the top and toe of the slopes (Photos 7-9). On the southeast slope, there is no vegetation just below the 636 bench (Photo 8). Vegetation is lacking on the exterior slopes between the 636 bench and the crest (Photo 9). Exposed bottom ash is located on the northwest slope and exposed bottom ash/clay cover is located on the southwest, northeast, and southeast slopes.
Trees:	Trees are growing adjacent to the discharge pipe from the Gypsum Stack Stilling Pond across the northeast exterior slope (Photo 10; Item 85).
Erosion:	Erosion rills/gullies located on the exterior slopes above the 636 bench along the entire perimeter of the stack (Photo 6). A possible seep and erosion rills was noted on the southeast exterior slope just below the 636 bench where vegetation is lacking (Photos 11-12; Items 16-17). An erosion rill located along the discharge channel leading to the outfall to the Guntersville Lake was identified (Photo 13; Item 18). Additional erosion areas were identified located along the southeast, southwest and northeast slopes (Photos 14-17; Items 81-84).
Instabilities:	None observed.
Uniform Appearance:	Yes

Seepage:	A possible seepage area was noted on the southeast exterior slope (Photo 11; Item 16). The area appears to have dried up and should not be assigned a seep number at this time.
Benches:	There is one bench at approximate elevation 636 feet. This bench is located on the southwest, southeast, and northeast slopes. The bench is covered in crushed stone/bottom ash (Photo 18).
Foundation Drains, and Seepage Collection Systems:	None observed.
Instrumentation:	Twenty-three piezometers were installed in 2009 by Stantec as part of a Phase 2 dike slope stability and seepage evaluation. Piezometer readings are being taken by URS on a monthly basis.
Animal Burrows:	None observed.
Slope:	2.5H:1V on the northeast exterior slope below the 636 bench, 3H:1V on the southeast and southwest slope below the 636 bench, and 4H:1V on all slopes above the 636 bench (as measured in 2009 TVA survey used for Stantec geotechnical evaluation).
Height:	Approximately 42 feet.

4.4. Spillway Weirs/Riser Inlets

Number:	2
Size, Type and Material:	3'L x 1'W x ~12'D, Aluminum frame, fiberglass stop log riser structures.
Height of Riser Inlets:	Approximately 12 feet.
Access:	Adequate access to both structures.
Joints:	In working order.
Mis-Alignment:	None observed.
Closed/Abandoned Conduits:	None

4.5. Outlet Pipes

Number:	2
Size, Type and Material:	18 inch diameter corrugated HDPE
Headwall:	None
Joint Separations:	Buried, no evidence of joint separation.
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None observed or reported.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Dredge Cell since the last annual inspection.

- The exterior slopes have been mowed.
- A water drop has been installed on the road along the southeast toe of the Dredge Cell near the discharge pipe outfall from the Gypsum Stack Stilling Pond (Photo 19; Item 87).
- A material stockpile (Photos 20-21; Item 88) has been constructed along the northwest crest of the Dredge Cell by placing bottom ash to build a platform. Materials used for the construction of the immediate risk reduction plans (IRRP) are stockpiled on top of the dredge cell where the material is loaded into articulating dump trucks and hauled and placed at the construction location.
- As recommended in the 2010 annual inspection, the culvert pipe under the road from the Main Ash Pond Dike to the Dredge Cell has been replaced with a new culvert pipe. The ditch in this area has been cleaned out (Photos 22-23; Item 89).
- Bottom ash has been placed on top of the Dredge Cell. The bottom ash was placed in the western portion extending from the intersection of the Main Ash Pond Dike road and the Dredge Cell perimeter crest road (Photo 24; Item 90).

6. Recommendations

URS Corporation is moving forward with a closure design for the Dredge Cell. These design efforts will address slope stability and seepage issues previously identified. Therefore, the recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design issues currently

being addressed by URS. Priority codes are included in parentheses and described in Enclosure M.

- Continue to mow the exterior slopes as needed to allow for future inspections of the slopes/dikes.
- Repair the erosion in accordance with the repair guidelines included in Enclosure N (Photos 6 and 11-17; Items 16-18, 81-84; Priority 4).
- Areas lacking vegetation should be re-seeded/fertilized in order to establish vegetation (Photos 5, 7-9, 14-16; Items 81-83; Priority 4).
- The trees growing adjacent to discharge pipe from the Gypsum Stack Stilling Pond should be removed in accordance with the guidelines included in Enclosure N (Photo 12; Item 85; Priority 4).
- The pipe installed during the dredging operation of Widows Creek under the 636 bench on the northeast side of the Dredge Cell should be removed (Photo 18; Item 86). It is currently acting as a discharge pipe for the perimeter ditch, which is not the intended purpose (Priority 4).
- There is a dislodged concrete manhole cover on the northeast exterior slope that should be repaired (Photo 17; Item 84; Priority 4).
- Remove the saplings which are growing on top of the Dredge Cell (Photo 24). The excessive vegetation growing on top of the cell can cause issues during closure. These should be removed in accordance with the general guidelines included in Enclosure N (Priority 4).

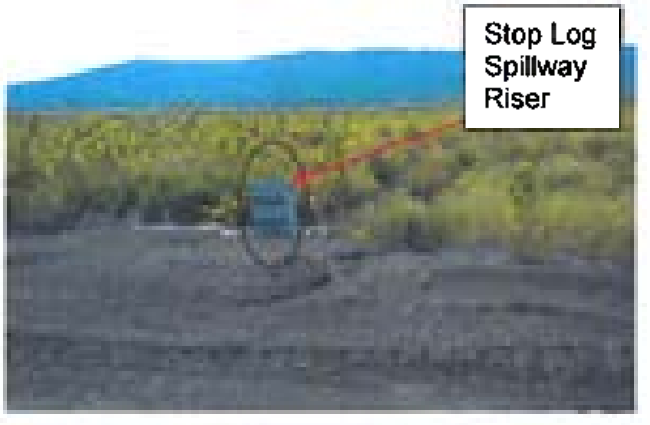


Photo 1

Interior slopes of the Dredge Cell in the western corner of the cell. Note the erosion rills along the interior slope and a spillway riser and thick vegetation across the cell.

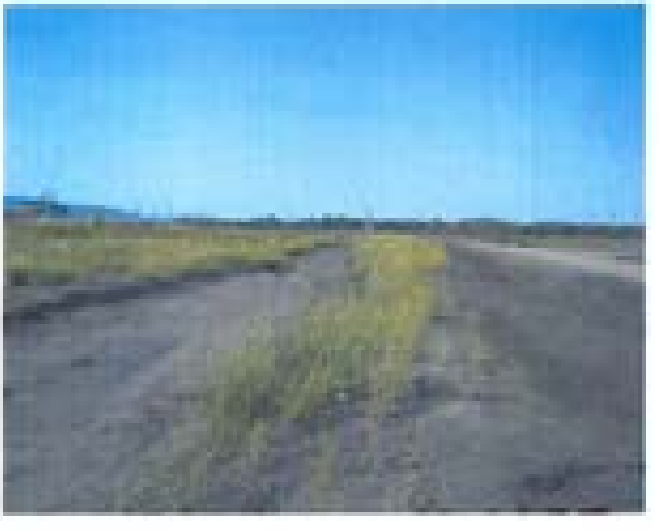


Photo 2

Northwest side of the Dredge Cell looking southwest.



Photo 3

Erosion on the interior slope along the northwest side of the Dredge Cell. Note the material stockpile staging area in the background.

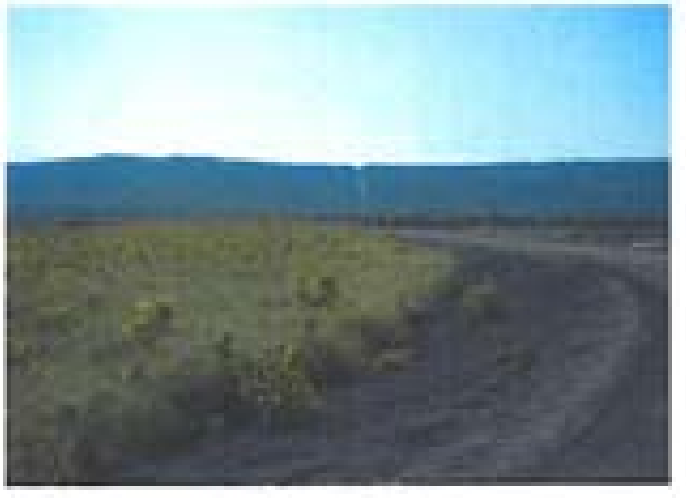


Photo 4

Crest in the northwest corner of the Dredge Cell. Note the crushed stone/bottom ash coverage and thick vegetation across the cell.

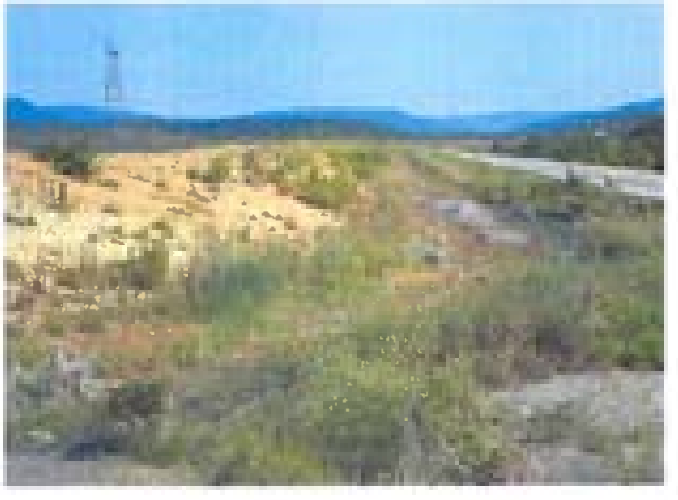


Photo 5

Southwest crest of the Dredge Cell looking northeast. Note sparse vegetation.



Photo 6

Erosion rills and the rim ditch southwest exterior slope above the Upper/Lower Stilling Pond.



Photo 7

Grass vegetation on the southeast exterior slope and no vegetation at the toe of the slope (Item 15).



Photo 8

Dense grass vegetation on the northeast exterior slope. Note the sparse coverage near the top of the slope just below the 636 bench.

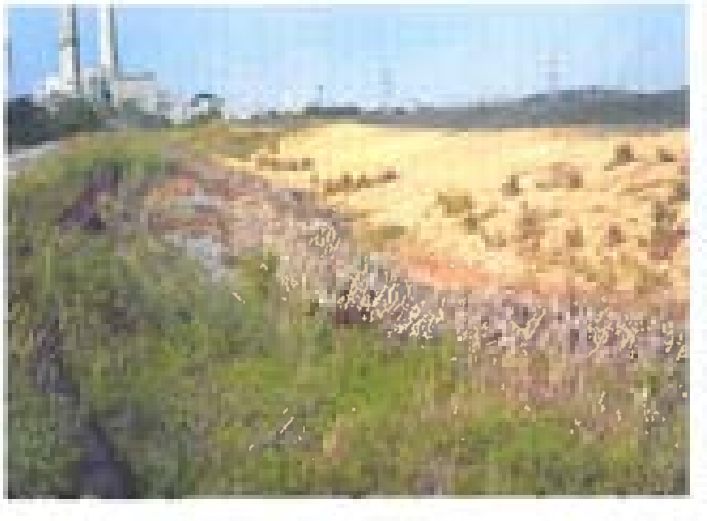


Photo 9

Lack of vegetation on the exterior slope above the 636 bench at the eastern corner of the cell.



Photo 10

Trees growing adjacent to the discharge pipe from the Gypsum Stack Stilling Pond along the northeast exterior slope corner (Item 85).



Photo 11

Lack of vegetation/possible seep location on the southeast exterior slope just below the 636 bench (Item 16).

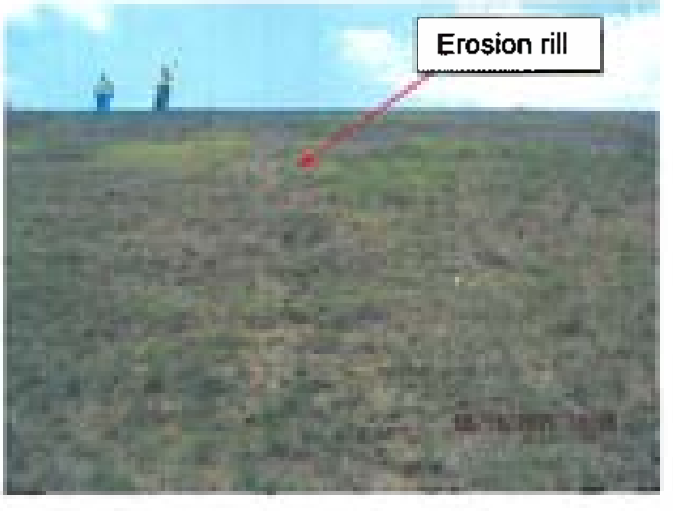


Photo 12

Exterior slope erosion rill below the 636 bench and on the southeast side of the Dredge Cell (Item 17).



Photo 13

Erosion rill located along the discharge channel to the outfall at the cove leading to Lake Guntersville (Item 18).



Photo 14

Lack of vegetation at the toe of the southeast corner exterior slope below the 636 bench (Item 81).

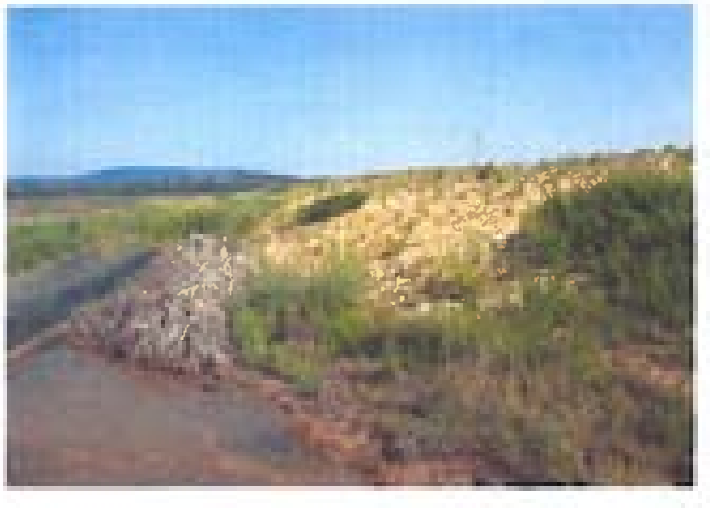


Photo 15

Erosion on the 636 bench on the southwest side of the Dredge Cell (Item 82).



Photo 16

Erosion on the northeast exterior slope below the 636 bench (Item 83).



Photo 17

Manhole access to discharge pipe from the Gypsum Stack Stilling Pond along the toe of the northeast exterior slope. Note the dislodged concrete cover (Item 84).



Photo 18

Pipe installed under the road on the 636 bench on the northeast side of the Dredge Cell during the dredging operations of Widows Creek after the January 2009 event. It is understood this was used in the dredging of Widows Creek (Item 86).



Photo 19

Discharge pipe outfall from the Gypsum Stack Stilling Pond into the discharge channel to the cove leading to Lake Guntersville outfall. Note the recently installed water drop piping (Item 87).



Photo 20

Construction material stockpile staging area along the north dike looking west (Item 88).



Photo 21

Construction material stockpile staging area along the north dike looking south (Item 88).



Photo 22

Northwest inlet of the culvert pipe under the road from the Main Ash Pond Dike to the Dredge Cell (Item 89).



Photo 23

Southeast inlet of the culvert pipe under the road from the Main Ash Pond Dike to the Dredge Cell (Item 89).



Photo 24

Top of the Dredge Cell from the northwest crest, looking east. Note the vegetation growth and the bottom ash which has been placed on top of the Dredge Cell since the 2010 annual inspection (Item 90). Note tree sapling growth.

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Plan View - Dredge Cell
WCF 2011 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
1 OF 1

Enclosure D

Upper-Lower Stilling Pond

Stantec

TVA 2011 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Upper/Lower Stilling Pond (DRAFT)

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	8 acres	**TVA Hazard Classification:**	Significant
Maximum Height (toe to top of dike):	21 feet	**Dike Length:**	2,980 feet
Plant Discharge to Facility:	77.4 cfs	**Current Pool Elevation:**	610.8 feet

2. Site Visit Information

Stantec Inspection Team: Chris Jones, EIT and Robert Fuller, PE

TVA Staff Present: None

Field Inspection Date: June 13, 2011

Weather/Site Conditions: Sunny, 85 degrees, dry

3. History/Current and Future Operations

History: Commissioned in 1986 as the third of four ponds in a series. The upland watershed includes Bottom Ash Stack, Main Ash Pond, and Dredge Cell.

Current Operations: Effluent from the Main Ash Pond enters the Stilling Ponds from a system of six siphons (Photos 1-3). The stilling pond discharges into the pump pond via five spillways, maintaining a pool elevation of approximately 610.8 feet. The Upper and Lower Stilling Ponds are hydraulically connected and partially separated by a deflector dike where carbon dioxide is added to balance pH. The flow of water is directed around an approximately 100-foot deflector dike separating the Upper Stilling Pond from the Lower Stilling Pond.

Future Planned Operational Changes: URS Corporation is tasked with the closure of the Main Ash Pond Complex. Currently, the future plan for the stilling ponds is unknown.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Partially covered in grass, partially covered in stone for wave wash protection. Adequate coverage in most locations. Some locations have bare spots with no vegetation (Photos 1-9). Also bare spots from erosion along the west dike at the top of slope from caustic water from bottom ash flows down the slope (Photo 10), (Item 94) and along the southwestern interior slope where wave wash protection is absent. Bare spots noted along the north, east and west dikes slopes (Photos 2, 3, 5, 7, and 8).
Trees:	Two small trees along the waterline of the southeast dike (Photo 8; Items 92-93).
Wave Wash Protection:	Stone wave wash protection located on the southeastern side between the Stilling Pond and Pump Pond, the southwestern side, and the interior deflector dike (Photos 11-15).
Erosion:	Small erosion rills were noted on the southwest interior slope where wave wash protection is absent (Photo 16; Item 95) and wave wash erosion at the waterline along the north dike (Photos 17; Item 25). Surface erosion rills were also noted on the northeast interior slope just below the 626 bench (Photo 10; Item 94).
Instabilities:	None observed.
Animal Burrows:	Animal burrow was located at the toe of the north dike near the waterline (Photo 18; Item 26).
Freeboard:	2.5 feet measured at southeast dike.
Encroachments:	None observed.
Slope:	2H:1V southeast and southwest 3H:1V northeast and northwest

4.2. Crest

Crest Cover and Slope:	Crushed stone/bottom ash surface on the southwest, northwest, and southeast sides, ash surface on the northeast side. Crest slopes toward pond on northeast, northwest, and southwest dikes, and appears to slope towards the Pump Pond on the southeast dike.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	45 feet measured on the northwest dike, 18 feet measured on the southwest dike, 25 feet measured on the northeast dike, and 30 feet measured on the southeast dike.

4.3. Exterior Slopes

Vegetation:	The Stilling Pond is located adjacent to the Main Ash Pond A and Dredge Cell. There is only an exterior slope on the southwest and southeast dikes. The southwest side has adequate grass coverage. The southeast dike is lacking vegetation and is covered with crushed stone/ash.
Trees:	None observed.
Erosion:	Substantial erosion noted at two locations on the southeast slope. The first location is a localized rill from surface runoff. The second location is the eastern portion of the dike where a slide appears to have occurred as noted on the 2010 annual inspection report. Erosion rills are located along this portion of the dike. See the Pump Pond report for photos.
Instabilities:	A slide approximately 60 feet in length occurred on the eastern portion of the southeast dike which was noted in the 2010 annual inspection report (Photo 19). See the Pump Pond report for additional description and photos.
Uniform Appearance:	Yes, the exterior slopes appear uniform.
Seepage:	Yes, seepage noted on both the southeast and southwest

	slopes (Photos 20 and 21).
Benches:	None observed.
Foundation Drains, and Seepage Collection Systems:	None reported or observed.
Instrumentation:	Two piezometers installed in February, 2010 on the northwest dike between the Stilling Pond and the Main Ash Pond A. Two piezometers installed in May, 2009 on the northeast dike between the Stilling Pond and the Dredge Cell.
Animal Burrows:	None observed.
Slope:	2H:1V southwest side 1.5H:1V southeast side
Height:	11 feet measured on the southeast side

4.4. Spillway Weirs/Riser Inlets

Number:	Five (5)
Size, Type and Material:	48-inch diameter push together, 4-feet barrel section man hole-type concrete riser structure with 10-foot steel skimmer (Photos 22-24).
Height of Riser Inlets:	19.5 feet
Access:	Access to one riser structure via a steel walkway (Photo 24).
Joints:	N/A
Mis-Alignment:	Last video inspection reported joint leakage.
Closed/Abandoned Conduits:	None reported or observed.

4.5. Outlet Pipes

Number:	Five (5)
Size, Type and Material:	36-inch diameter RCP
Headwall:	None, outlet under at least six feet of water in the middle of the Pump Pond.

Joint Separations: N/A

Mis-Alignment: N/A

Closed/Abandoned Conduits: None reported or observed.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance activities and improvements have been performed for the Stilling Pond since the last annual inspection.

- The interior slopes have been mowed (Photos 25-27).
- The floating ash/cenospheres in the Stilling Pond have been removed from the surface of the pond.
- The broken handrail noted in the 2010 annual inspection has since been repaired (Photo 28; Item 96).
- Previously identified animal burrows along the northeast slope have been repaired.
- A new siphon system has been installed from the Main Ash Pond A to the Upper Stilling Pond (Photos 1-3; Item 96). See the Main Ash Pond A report for additional information on the new siphon system.

6. Recommendations

URS Corporation is moving forward with a closure design for the Upper and Lower Stilling Pond. These design efforts will address slope stability and seepage issues previously identified. The mitigation project should include the construction of compacted clay dikes on the southeast and southwest dikes. Existing bottom ash should be removed from dikes and replaced with compacted clay. The alignment of the southwest dike should be straightened (Photo 27). This work should be completed in a 6-month time frame. The recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design issues currently being addressed by URS. The following maintenance recommendations are offered for the Upper/Lower Stilling Pond. Priority codes are included in parentheses and described in Enclosure M.

- Continue to mow the interior slopes as needed.
- Continue to maintain the floating booms (turbidity curtains) across the pond to collect the floating cenospheres (Photos 24-26). Clean these cenospheres from the pond as needed.

- Repair the wave wash erosion and erosion rills noted on the southwest and northwest interior slopes (Photos 16-17; Items 25 and 95). Install wave wash protection along interior slopes where necessary in accordance with the guidelines provided in Enclosure N.
- Repair animal burrows near the waterline on the northwest interior slopes in accordance with the guidelines provided in Enclosure N (Photo 18; Item 26; Priority 4).
- Remove the small trees growing along the northeast interior slope (Photos 4, 8 and 25; Items 92-93) and in the skimmer structure (Photos 22 and 24; Item 91) in accordance with the tree removal guidelines provided in Enclosure N (Priority 4).
- Seed bare spots on the northeast and northwest interior slopes near the crest of the dike to re-establish vegetation. A new seed/fertilizer mixture may need to be used to enable vegetative growth (Priority 4).
- Repair erosion rills along the northeast interior slope (Photo 10; Item 94; Priority 4).



Photo 1

Northwest crest/interior slope looking southwest. Note the new siphon outlets from the Main Ash Pond A and repaired groin (foreground, center).

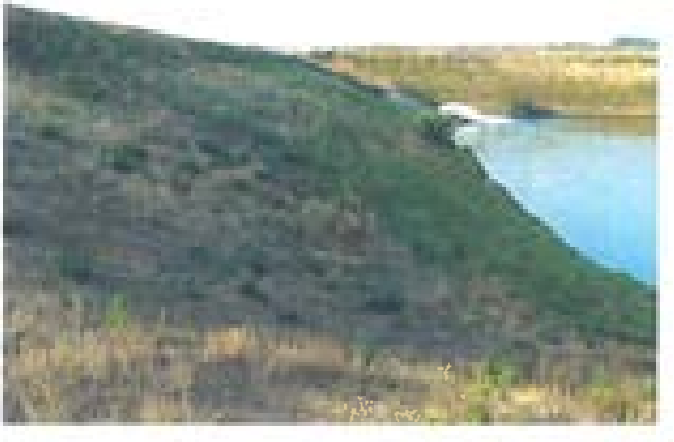


Photo 2

Well maintained north dike interior slope, looking northeast.



Photo 3

Northwest interior slope, looking north. Note the new siphon outlets from the Main Ash Pond A and sparse vegetation.

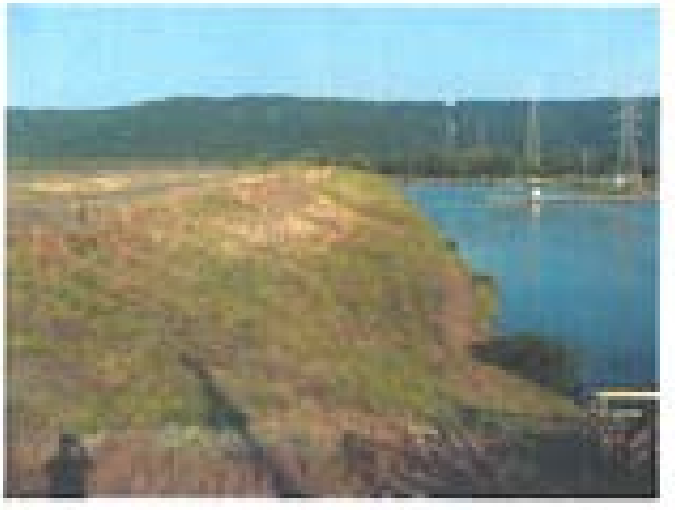


Photo 4

East dike interior slope looking south from the crest of the north dike. Note the sparse vegetation on the slope just below the crest and woody vegetation at waterline.

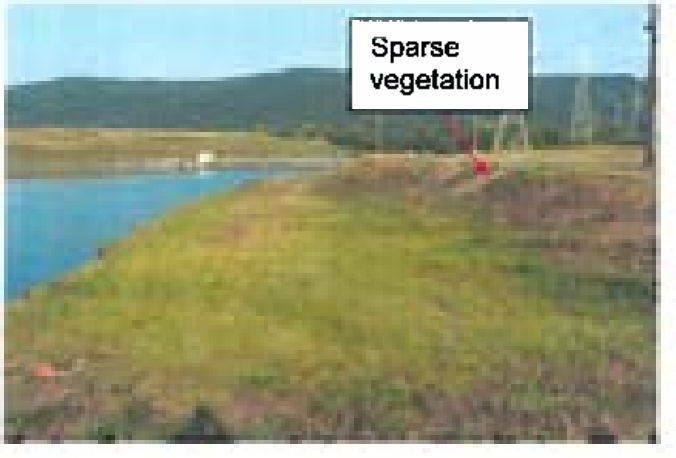


Photo 5

Northwest dike crest/interior slope, looking south. Note the sparse vegetation along the slope just below the crest.

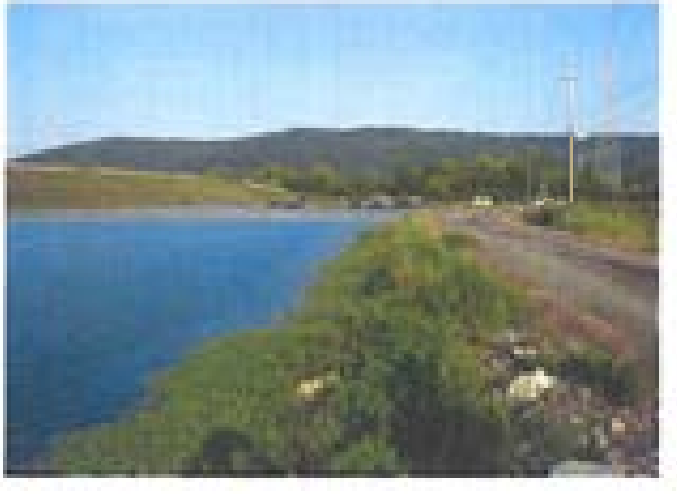


Photo 6

Southwest dike crest/interior slope, looking south.



Photo 7

Northeast dike looking north. Note the erosion from storm water runoff at the crest and sparse vegetation.

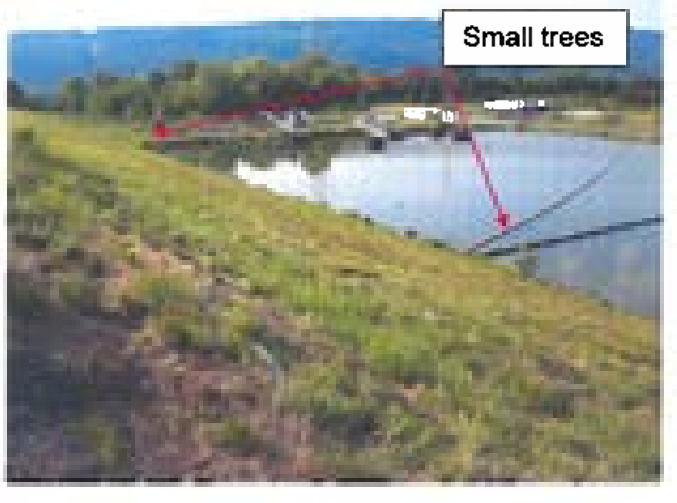


Photo 8

Northeast dike interior slope looking south toward south dike and spillways to Pump Pond. Note the floating booms across the pond for cenosphere management and the two small trees (Items 92-93).



Photo 9

Crest of the northeast dike, looking west toward the groin between the southwest and northwest dike.



Photo 10

Surface runoff rill erosion at the 626 bench on the northeastern dike. Note the lack of vegetation at the top of the slope where caustic water from the bottom ash surface flows down the slope (Item 94).



Photo 11

Wave wash protection on the interior deflector dike.



Photo 12

View of the interior deflector dike from northeast slope, looking west. Note the wave wash protection along the deflector dike and the CO_2 injection system.



Photo 13

Southern slope of the interior deflector dike with wave wash protection.



Photo 14

Southwestern interior slope and deflector dike with wave wash protection and vegetation.

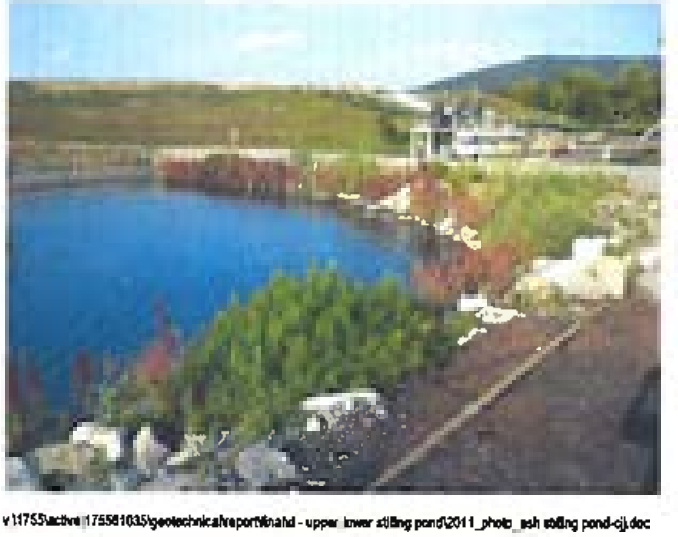


Photo 15

Wave wash protection and vegetation in the southern corner of the stilling pond.

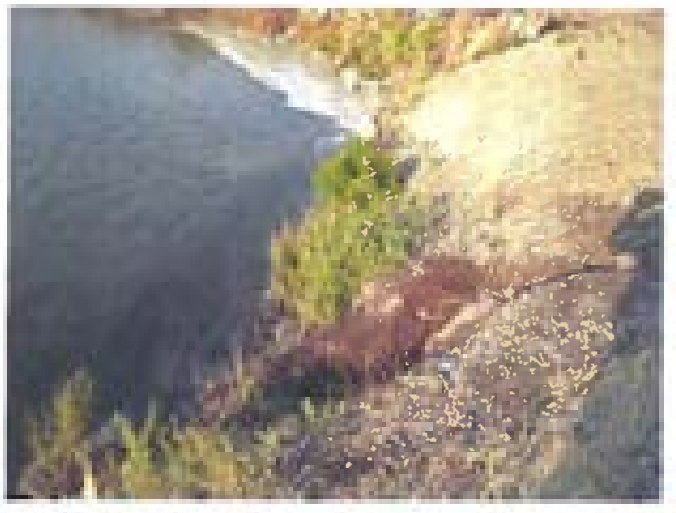


Photo 16

Erosion rill on the southwest interior slope due to lack of vegetation/wave wash protection on the slope surface (Item 95).



Photo 17

Wave wash erosion along the north dike (Item 25).

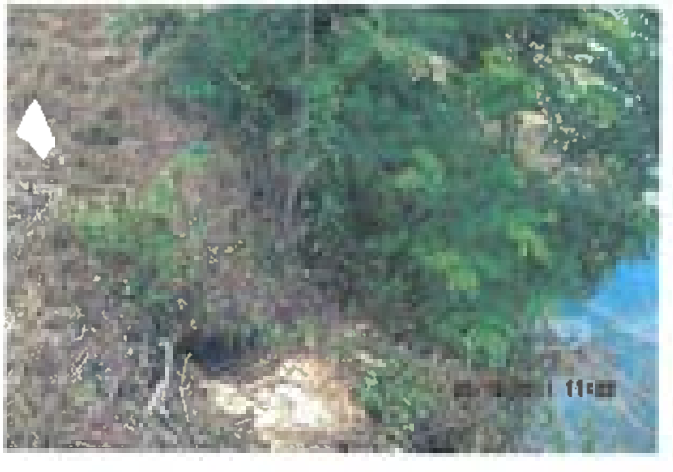


Photo 18

Typical animal burrow encountered at the waterline of the north interior dike (Item 26).



Photo 19

Southeastern dike crest. Note some rutting in the right center. Also note the slough that occurred at the eastern end of the dike, removing approximately five feet of crest width. This was noted in the 2010 annual inspection.



Photo 20

Southwestern dike exterior slope, looking south. Note the dense vegetation on the slope. The crest is covered in crushed bottom ash. There is seepage (noted as Seep 14) at the toe of the exterior slope.

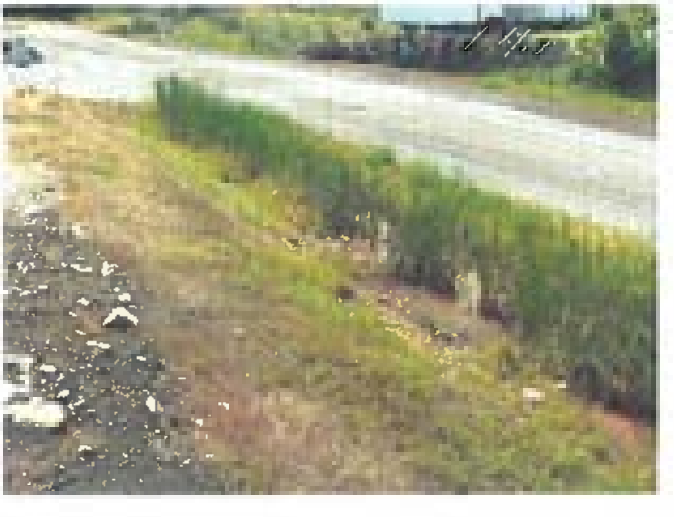


Photo 21

Seepage at the toe of the southwest dike exterior slope looking west.



Photo 22

Five spillway riser structures with steel skimmers at the southeastern end of the pond. Note the tree growing in the far skimmer structure (Item 91).



Photo 23

48 inch diameter concrete riser structure with ten foot steel skimmer.



Photo 24

Steel walkway for access to the middle spillway riser. Note the tree growing in the skimmer structure to the left (Item 91).



Photo 25

Southeastern interior slope with woody vegetation. Note the floating booms across the pond for cenosphere management (Item 92).

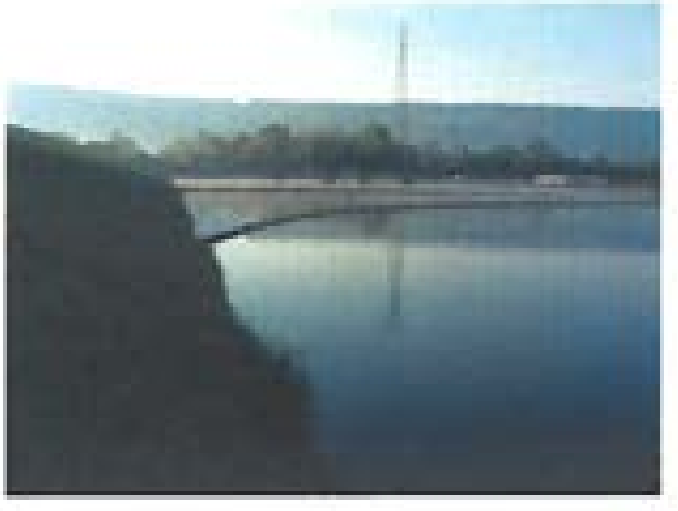


Photo 26

Northeastern slope with dense vegetation, looking southeast. Note the floating boom across the pond and the five spillways with steel skimmers in the background.



Photo 27

View of stilling pond from the northwest dike, looking southeast. Note the mis-alignment of the southwest dike.



Photo 28

New northern platform of the interior deflector dike (broken hand rail repaired since 2010 Annual Inspection). Note the floating boom (turbidity curtain) in the background used for cenospheres management.

N
Upper Stilling Pond
Lower Stilling Pond
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Stantec
Plan View - Upper/Lower Stilling Pond
WCF 2011 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
1 OF 1

Enclosure E

Pump Pond

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	0.4 acres	**TVA Hazard Classification:**	Significant
Maximum Height (toe to top of dike):	16 feet	**Dike Length:**	425 feet
Plant Discharge to Facility:	77.4 cfs Normal Operating conditions (recycle 100% of inflow back to plant)	**Current Pool Elevation:**	607.4 feet

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT and Robert Fuller, PE, Greg Budd, Briggs Evans
TVA Staff Present:	James Mullins
Field Inspection Date:	June 13, 2011
Weather/Site Conditions:	Sunny, 85 degrees, dry conditions

3. History/Current and Future Operations

History: The Pump Pond was commissioned in 1986 to recirculate process water from the Ash Pond Complex back to the power plant intake as the fourth of four ponds in a series. The upland watershed includes the Bottom Ash Stack, Main Ash Pond A, Dredge Cell, and Upper/Lower Stilling Pond.

Current Operations: The Pump Pond receives process and storm water from the Lower Stilling Pond via five spillway outlet pipes. The pond operates using a two-pump system. The large pump in the middle of the pond (U7/U8 Intake Pump Station) recirculates the water back into the sluice system. The pump located in the southern corner (Ash Pond Recirculation Pump Station) pumps water to the ball mill as makeup water. If the pumps are disables/non-functioning, the effluent and storm water flows through three spillway conduits to NPDES Outfall 001. See Photo 1 for an overview of the Pump Pond. At the time of

	the inspection, both pump stations were inactive.
Future Planned Operational Changes:	TVA plans to close the Pump Pond by filling the impoundment with controlled fill and creating a permanent Lower Stilling Pond outfall system with a spillway outfall into the cove leading to the Lake Guntersville. The design phase has yet to commence.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Dense vegetation on the northeast and southeast interior slopes. Sparse vegetation located on the northwest and southwest slopes.
Trees:	Cluster of small trees (2" diameter) noted in the eastern corner of the pond (Photo 2), (Item 98).
Wave Wash Protection:	Wave wash protection located along the entire southwest interior slope (Photo 3) and along the base of the northwest interior slope (Photo 4). Wave wash protection also located along the southeast slope around the three outlet pipes (Photos 5-8).
Erosion:	Substantial erosion noted at two locations on the northwest slope. The first erosion area is a localized rill from surface runoff (Photo 9; Item 99). The second location is the eastern portion of the dike where a slough appears to have occurred (Photo 10-11; Item 101). The slough was noted in the 2010 annual inspection report. Additionally, a rill was observed along the south interior dike adjacent to the intake pump area (Photo 12; Item 100).
Instabilities:	Prior to 2008, a slough occurred on the eastern portion of the northwest dike (see above) leaving the entire interior slope of the northwest dike relatively steep and subject to erosion due to surface runoff, seepage from the stilling pond (Photo 11), and cyclic rapid draw down conditions from the pump station operation.
Animal Burrows:	None observed.

Freeboard:	9 feet
Encroachments:	None observed.
Slope:	1.5H:1V

4.2. Crest

Crest Cover and Slope:	Crest covered in crushed stone with adequate coverage. The northwest, northeast, and southwest crests slope toward the interior of the pond, while the southeast crest slopes away from the pond toward a cove to the river.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	30 feet wide on northwest dike, 25 feet wide on northwest dike in location of slough, and 100 feet wide on southeast dike.

4.3. Exterior Slopes

Vegetation:	The Pump Pond is surrounded by the Old Scrubber Sludge Pond (Dredge Cell) to the northeast (Photo 13), the Upper/Lower Stilling Pond to the northwest and a constructed wetlands to the southwest. The only exterior slope is on the southeast dike, between the Pump Pond and a cove leading to the Lake Guntersville. This downstream slope has dense woody vegetation on either side of an existing discharge channel (Photo 14).
Trees:	Yes, numerous trees noted.
Erosion:	None observed.
Instabilities:	None observed.
Uniform Appearance:	No, the slope does not appear to be uniform.
Seepage:	None observed.
Benches:	None observed.

Foundation Drains, and Seepage Collection Systems:	No provisions for seepage collection.
Instrumentation:	None.
Animal Burrows:	None observed.
Slope:	3H:1V
Height:	16 feet

4.4. Spillway Weirs/Riser Inlets

Number:	The Pump Pond does not have spillway weirs nor skimmers under normal operating conditions. The outlet system consists of two pump stations. The pump in the middle of the pond (Photo 15) is the U7/U8 Intake Pump Station which recirculates water back to the power plant for re-use. The pump in the southern corner (Photo 16) is the Ash Pond Recirculation Pump Station (Ball Mill Pump Station) which circulates water to the limestone ball mill.
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None reported or observed.

4.5. Outlet Pipes

Number:	If the pumps are disabled/non-functioning, three (3) spillway conduits penetrating the southeast dike serve as the outfall for the Pump Pond (Photos 6-8). Outlet channel appears well protected with large boulder-size stone.
Size, Type and Material:	36 inch x 60 inch elliptical corrugated metal pipe (CMP).
Headwall:	None.
Joint Separations:	None observed.

Mis-Alignment: None observed.

Closed/Abandoned Conduits: None reported or observed.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Pump Pond since the last annual inspection. The pond elevation was approximately five feet higher from the previous year's annual inspection during the time of inspection. Some areas noted on the previous inspection were unable to be observed due to the higher water elevation.

- The interior and exterior slopes of the pond have been mowed.
- Abandoned above ground sluice lines have been removed from the south dike area.

6. Recommendations

Based on discussions with TVA, it is understood that URS is currently designing a new spillway system in the Lower Stilling Pond to discharge process and storm water flows to through Outfall 001 and removing the existing pump stations thus eliminating the Pump Pond. Therefore, the recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design issues currently being addressed by URS. Priority codes are included in parentheses and described in Enclosure M.

- Continue to mow the interior slopes as needed.
- Remove the cluster of trees and root systems in the eastern corner of the pond in accordance with the guidelines provided in Enclosure N (Photo 2; Item 98; Priority 4)
- Two areas of erosion were noted during the inspection. One area was on the interior slope of the northwest dike between the Lower Stilling Pond and Pump Pond (Photos 9-10; Item 99). The second area of erosion is on the southeast interior slope near the pump station (Photo 12; Item 100). It is recommended that these rills be repaired in accordance with the general guidelines provided in Enclosure N. All unprotected interior slopes should be flattened uniformly with structural fill overlaid by wave wash protection consisting of ALDOT Class I Rip-Rap. (Priority 4)
- The apparent slough located on the eastern portion of the northwest dike (Photo 11; Item 101) which was noted in the 2010 annual inspection should be repaired to reconstruct the slopes with structural fill to a final grade of 3:1 or flatter. It is recommended that this dike be evaluated and a design developed to mitigate the risks against future slides/instabilities. (Priority 4)

- A culvert installed to direct the flow of storm water from the low-lying wetlands into the Pump Pond is not functioning properly. The invert of the culvert was installed at a higher elevation than the water surface of the pool that develops in the engineered wetlands after a rainfall event. A board has been placed across the outlet end of this culvert into the Pump Pond (Photo 19; Item 102). It is recommended that this pipe be redesigned and reconstructed at a proper elevation to fulfill the intended purpose of draining the engineered wetlands/ adjacent area to the pond. (Priority 5)
- It appears a 12-inch steel pipe was abandoned in place by placing a blind flange (cover) over the inlet end (Photo 20). It is recommended that this abandoned pipe (Photo 21) be identified, excavated, and removed, if found appropriate (Items 103-104; Priority 5)

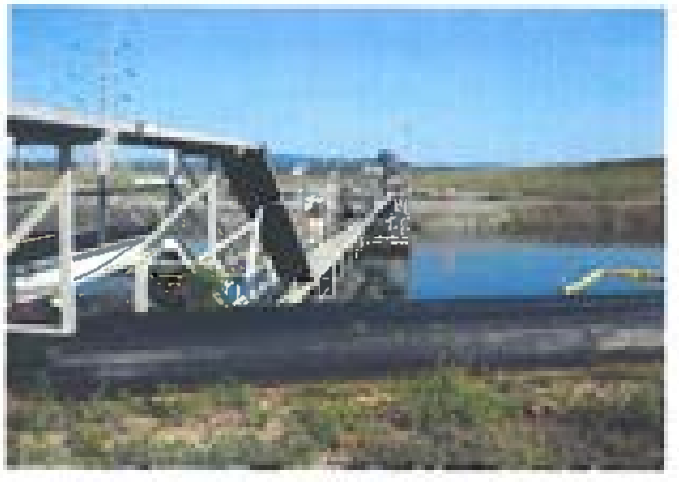


Photo 1

U7/U8 Intake Pump Station and Pump Pond, looking northwest.

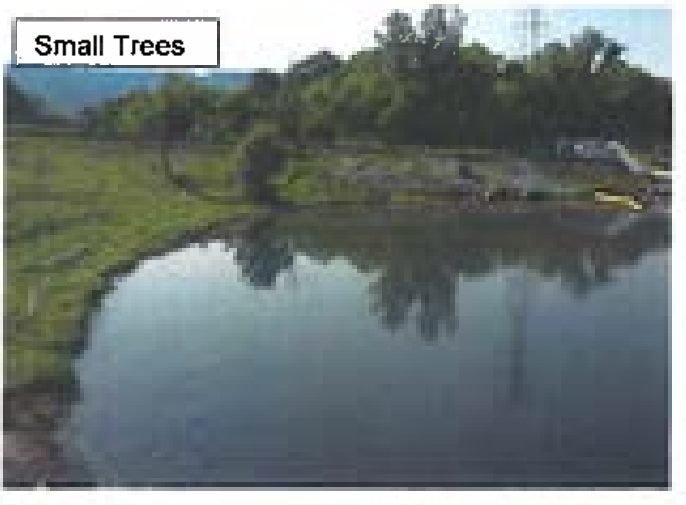


Photo 2

Southwest dike with two of the three spillway conduit inlets shown. Rip-rap placed around the spillway inlets. Note the cluster of small trees in the eastern corner of the pond (Item 98).



Photo 3

Rip-rap placed along the southwest interior slope of the Pump Pond. Note the lack of slope protection on the northwest slope.



Photo 4

View of the northwest interior slope



Photo 5

Rip-rap and grass vegetation on the southwest interior slope.



Photo 6

Three (3) 36 inch x 60 inch oval corrugated metal pipes that serve as the outfall for the Pump Pond when the recirculation system is turned off. Note that the far right conduit has had a section removed.



Photo 7

Two of the spillway conduits. For this report, identified as SC-1 and SC-2 from left to right.



Photo 8

Third spillway conduit identified as SC-3.

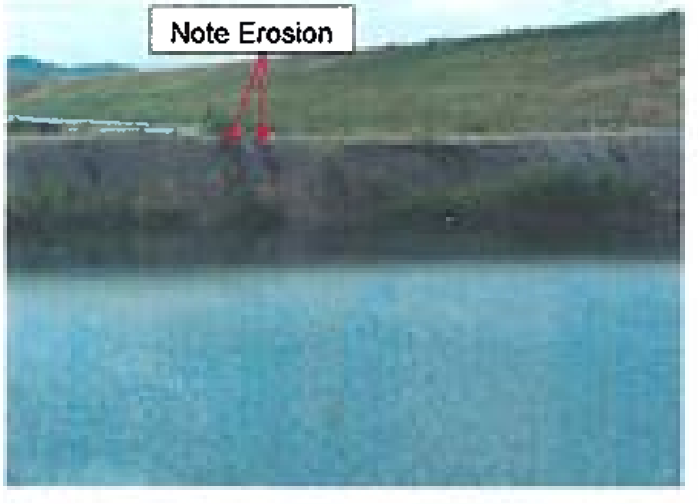


Photo 9

Erosion on the northwest dike between the Lower Stilling Pond and Pump Pond (Item 99).



Photo 10

Slough on the northwest dike (Item 101). Note the erosion present in this location.

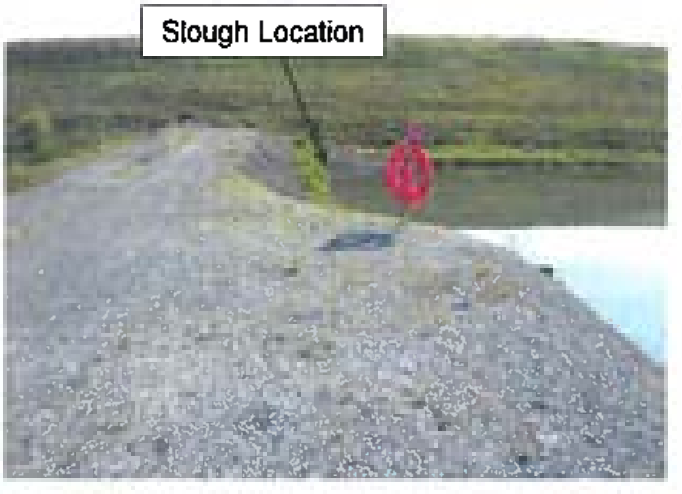


Photo 11

Crest of the northwest dike between the Lower Stilling Pond and Pump Pond, looking northeast. Note the apparent slough that occurred on the interior slope of the dike.



Photo 12

Erosion rill on south dike adjacent to pump intake (Item 100).



Photo 13

Southeast dike looking northeast. Note the Dredge Cell exterior slope in the background of the photo which lies to the northeast of the Pump Pond.



Photo 14

Spillway discharge channel leading to a cove of the Lake Guntersville, looking southwest.

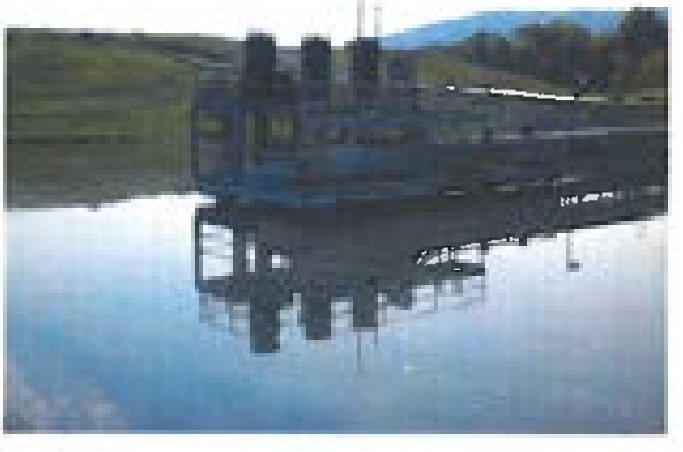


Photo 15

U7/U8 Intake Pump Station, looking east.

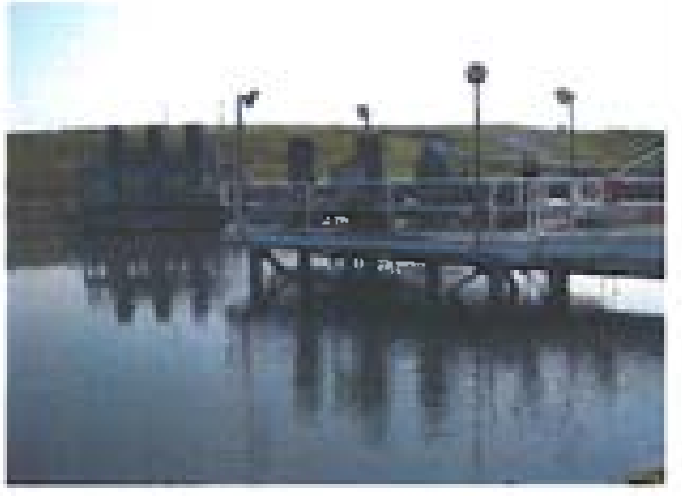


Photo 16

Pump station in the southwest corner of the Pump Pond. This pump recirculates water back to the Ball Mill.



Photo 17

Sluice pipelines along south dike crest to Dredge Cell.



Photo 18

Pipelines adjacent to the engineered wetlands.



Photo 19

Board placed across the outlet end of the culvert from the low-lying wetlands into the Pump Pond (Item 102).



Photo 20

Blind flange placed on the 12-inch diameter steel pipe from the U7/U8 intake pump station (Item 103). Note the inlet end of the abandoned pipe no longer in use.

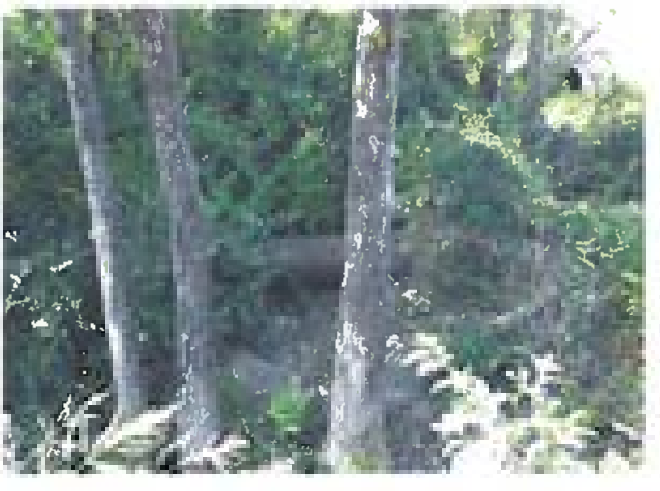


Photo 21

Abandoned 12-inch diameter steel pipe which is no longer in use (item 104). The inlet end of this pipe is shown in Photo 20. This pipe penetrates the southeast dike of the Pump Pond.

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Plan View - Pump Pond
WCF 2011 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
1 OF 1

Enclosure F

Chemical Pond

Stantec

TVA 2011 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Chemical Ponds (DRAFT)

1. General Facility Information

Facility Status:	Inactive	**NID Identification:**	N/A
Surface Area (inside dikes):	Iron Pond: 2 acres; Copper Pond: 1.5 acres	**TVA Hazard Classification:**	N/A
Maximum Height (toe to top of dike):	N/A	**Dike Length:**	Iron Pond: 1,300 feet; Copper Pond: 1,000 feet
Plant Discharge to Facility:	N/A	**Current Pool Elevation:**	Iron Pond: 628.4 ft; Copper Pond: 626.7 ft

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT and Robert Fuller, PE
TVA Staff Present:	None
Field Inspection Date:	June 14, 2011
Weather/Site Conditions:	Sunny, 85 degrees, dry conditions

3. History/Current and Future Operations

History:	The Iron Pond and Copper Pond were first shown on TVA-Drawings 10W213-R5 in 1976. The ponds are located within the interior of the Ash Pond Complex perimeter dikes in the south of the Bottom Ash Stack. This area was previously the Ash Disposal Area for Units 7-8.
Current Operations:	The chemical ponds are currently not receiving any discharge from the plant and were used in a cleaning process no longer part of plant operations.
Future Planned Operational Changes:	None reported at this time.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	The Iron Pond is lined with wave wash protection with no vegetation. The Copper Pond has dense grass vegetation with adequate coverage on all interior slopes. There are some locations with woody growth. Vegetation is overgrown in some locations (Photos 1-2).
Trees:	None observed.
Wave Wash Protection:	Some wave wash protection noted on the southeast and southwest interior slopes of the Copper Pond (Photo 2 and 3). Wave wash protection is present along all interior slopes of the Iron Pond (Photos 4, 5, and 6).
Erosion:	None observed.
Instabilities:	None observed.
Animal Burrows:	One animal burrow was observed along the crest of the Copper Pond (Photo 7).
Freeboard:	5 feet, measured at the southeast dike of the Iron Pond. It should be noted that these ponds are located within the interior of the Ash Pond Complex perimeter dike, and the freeboard was measured for the immediate dike surrounding the ponds. Should dike overtop, the overflow would drain into a basin still within the limits of the main perimeter dike.
Encroachments:	None observed.
Slope:	Iron Pond - 2H:1V Copper Pond - 2H:1V

4.2. Crest

Crest Cover and Slope:	The crest of both ponds is covered with bottom ash and appears relatively flat (Photos 1-6).
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.

Width:	30 feet

4.3. Exterior Slopes

Vegetation:	The Chemical Ponds are located within the perimeter dikes of the Ash Pond Complex. The exterior slopes associated with these dikes will be discussed within the Bottom Ash Stack report.
Trees:	N/A
Erosion:	N/A
Instabilities:	N/A
Uniform Appearance:	N/A
Seepage:	N/A
Benches:	N/A
Foundation Drains, and Seepage Collection Systems:	N/A
Instrumentation:	N/A
Animal Burrows:	N/A
Slope:	N/A
Height:	N/A

4.4. Spillway Weirs/Riser Inlets

Number:	The Chemical Ponds does not have spillway weirs nor skimmers. The outlet system is via a pump station located in each pond. The pumps are located in the center of the southeast dike of each pond (Photos 8-10). Water is pumped from the pond to a ditch traversing through the Bottom Ash Stack. The ponds no longer receive discharge from the plant, and currently maintain the existing pool elevation which is controlled by manually operating the dewatering pumps.
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A

Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None observed or reported.

4.5. Outlet Pipes

Number:	See section 4.4 for description of the pump outlet system.
Size, Type and Material:	N/A
Headwall:	N/A
Joint Separations:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None observed or reported.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Chemical Ponds since the last annual inspection.

- The drainage basin south of the chemical ponds appears to have been cleaned. Vegetation which was on the bottom of the basin in the 2010 annual inspection is now gone, and the basin appears to be covered in bottom ash with sparse vegetation (Photo 11). There appears to be a new inlet pipe from the bottom ash stack into the drainage basin (Photo 12; Item 106).

6. Recommendations

Based on the yearly inspection, the following maintenance recommendations are offered for the Chemical Ponds. Priority codes are included in parentheses and described in Enclosure M.

- Mow the interior slopes and vegetation in the drainage basin as needed.
- A closure plan should be developed to dewater and close the ponds. The pond footprint could be used for the stacking of dry ash.
- Repair the animal burrows noted near the copper pond in accordance with the

general repair guidelines provided in Enclosure N (Photo 8; Item 105; Priority 4).

- The hydraulics of the drainage basin south of the chemical ponds need to be better defined. It was unclear during the annual inspection how this area is intended to drain and what purpose it serves. A study should be conducted to determine the effectiveness and purpose of this drainage area (Photos 11 and 12; Item 106).



Photo 1

East interior slope, looking northwest, of the Copper Pond with dense grass vegetation. Note the small woody growth still present in some locations on the slope.

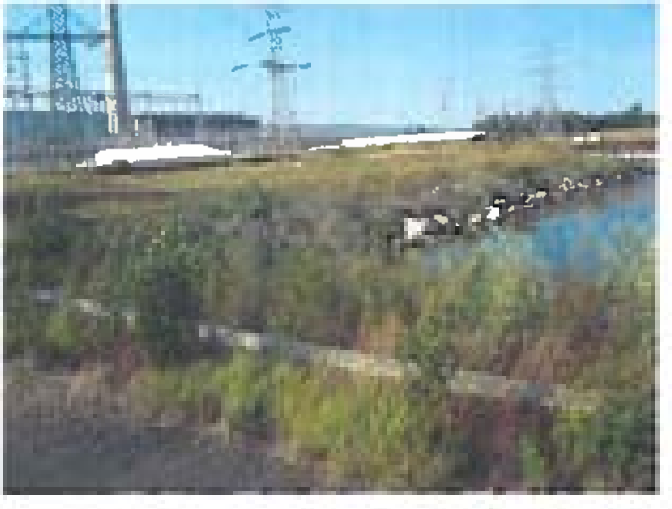


Photo 2

East corner of the Copper Pond with dense grassy vegetation and wave wash protection along the southeast interior slope.

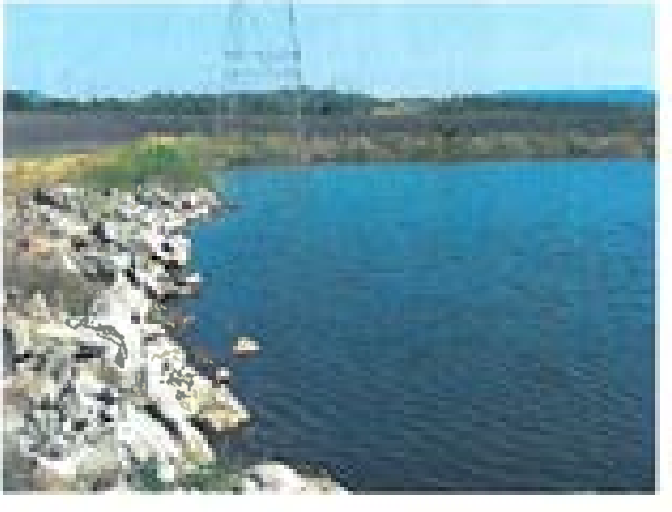


Photo 3

Wave wash protection along the southwest interior slope of the Copper Pond.



Photo 4

View of the Iron Pond north and east interior slopes, looking northeast. Note the wave wash protection along the interior slopes.



Photo 5

Southwest interior slope of the Iron Pond, looking northwest. Note the wave wash protection along the interior slope.



Photo 6

Wave wash protection along the southeast interior slope of the Iron Pond.



Photo 7

Animal burrow observed on the crest of the north dike of the Copper Pond (Item 105).



Photo 8

Pump station located at the southeast end of the Iron Pond.



Photo 9

Pump station located at the southeast end of the Copper Pond.



Photo 10

Pump station located at the southeast end of the Copper Pond. Note the woody growth present in the foreground of the picture on the northwest interior slope.



Photo 11

Drainage basin located to the southeast of the Chemical Ponds (Item 106).



Photo 12

12 inch diameter inlet pipes to the drainage basin (Item 106).

N
Fly Ash Pond
Copper Pond
Feet
0
400
Stantec
Plan View - Chemical Ponds
WCF 2017 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
1 OF 1

Enclosure G

Red Water Pond

Stantec

TVA 2011 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Red Water Pond (DRAFT)

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	16 acres	**TVA Hazard Classification:**	N/A
Maximum Height (toe to top of dike):	12 feet	**Dike Length:**	340 feet
Plant Discharge to Facility:	N/A	**Current Pool Elevation:**	Pumped down 3 to 4 feet below normal pool for construction activities

2. Site Visit Information

Stantec Inspection Team:	Greg Budd and Briggs Evans
TVA Staff Present:	Stuart Harris and Jake Booth
Field Inspection Date:	June 13, 2011
Weather/Site Conditions:	Sunny, 85 degrees, dry conditions

3. History/Current and Future Operations

History:	The Red Water Pond Dike was design in 1987 (TVA Drawing 107466-1 and 107466-2).
Current Operations:	The Red Water Pond collects surface runoff directly from the northwest slopes of the Main Ash Pond and indirectly from the northwest, west, and south slopes of the Bottom Ash Stack and Chemical Ponds via a perimeter ditch along Scrubber Road. The pond is controlled by a pump station, operating at a normal pool elevation between 600 and 604 feet. Currently the pool has been pumped down due to rock buttress construction activities along the east dike interior slope
Future Planned Operational Changes:	None reported at this time.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Eastern interior slope between the Main Ash Pond A and Red Water Pond is covered in dense grass vegetation, with occasional bare spots present prior to mowing and from the tracks on the mower (Photos 1-4). The west slope between the railroad tracks and Red Water Pond has thick woody/brush vegetation and some storm damage to trees (Photos 5-7). The north slope has vegetation where wave wash protection is not present (Photo 8).
Trees:	Small trees along the west and north dikes (Photos 1, 3, 8; Item 107). Large trees have been uprooted due to recent tornado damage (Photos 6 and 8; Item 37).
Wave Wash Protection:	Wave wash protection covered in thick vegetation present on the interior slope of the north dike, referred to herein as the Red Water Pond Dike (Photo 9).
Erosion:	Erosion/scouring observed at toe of the of the west interior slope below the inlet pipes (Photo 7; Items 38-39) and along the south end of the east dike below the 620 bench (Photos 10 -11).
Instabilities:	Uprooted and erosion/undercut trees along the west interior slope and south end of the east dike (Photo 10). Three corroded 12 inch metal pipes penetrating the south end of the 620 feet bench were observed. The pipes are decayed and erosion is occurring around and/or under the pipes leading to potential instabilities (Photos 12-16; Items 34, 71-74). These are discussed in the Bottom Ash Stack report.
Animal Burrows:	None observed.
Freeboard:	10 feet, measured at the emergency overflow spillway on the north dike
Encroachments:	None observed.
Slope:	2H:1V southeast dike below the pump station access ramp and 620 bench located at the south end of the pond (Photos 14-16). 3H:1V west dike north of the access ramp to the pump

station (Photos 5 and 9).
Slope measurements along the northeast dike have changed due to construction activities associated with the rock buttress construction (Photos 1-2).

4.2. Crest

Crest Cover and Slope: West dike crest is covered in railroad ballast along the railroad track and asphalt along the paved road (Photos 17-18). The west dike crest slopes toward the pond. The east dike is covered in crushed stone and appears crowned, with half the crest sloping towards the pond and half sloping towards the perimeter ditch of the ash pond. (Photo 19) The north dike is covered in grass vegetation and riprap (Photo 20).

Erosion: None Observed

Alignment: No issues observed.

Settlement/Cracking: There are cracks in the asphalt roadway and the guardrail dips along the west dike crest, possibly indicating subsidence (Photo 17; Items 40-42).

Bare Spots/Rutting: Bare spots along the north dike crest.

Width:
20 feet: west dike (railroad)
50 feet: west dike (roadway)
20 feet: north dike
20 feet: east dike

4.3. Exterior Slopes

Vegetation: The Red Water Pond is located immediately adjacent to the west side of the Main Ash Pond A and the east side of the roadway/railroad tracks. The only exterior slope is located on the north dike. The exterior slope of this dike is covered in dense vegetation with thick vegetation along either side of the spillway notch (Photo 21). Channel Lining covers the west side of slope at the emergency overflow spillway notch and discharge channel.

Trees: None observed.

Erosion: None observed.

Instabilities:	The discharge channel outlet pipes to the cove leading to Widows creek are rusted and damaged and there are a number of uprooted trees adjacent to the channel.
Uniform Appearance:	The slope of the dike appears uniform; however, the eastern portion of the slope is covered in vegetation while the western portion of the dike is covered in Channel Lining.
Seepage:	None observed.
Benches:	None observed.
Foundation Drains, and Seepage Collection Systems:	None observed.
Instrumentation:	None observed.
Animal Burrows:	None observed.
Slope:	3H:1V
Height:	12 feet

4.4. Spillway Weirs/Riser Inlets

Number:	The Red Water Pond does not have spillway decant structures or skimmers. The outlet system is via a pump station under normal conditions. The pump is located on the eastern side in the middle of the pond (Photo 22). Water is pumped from the pond up the slope across the old rim ditch/current perimeter ditch for the Main Ash Pond A (Photo 23). The Red Water Pond (north) dike has an rip-rap lined emergency overflow spillway notch and discharge channel (Photos 24-25). Approximately 60 feet downstream of the north dike, two culverts leading to Widows Creek are corroded and damaged (Photo 25).
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A

Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None observed or reported.

4.5. Outlet Pipes

Number:	See Section 4.4 for description of pump outlet system.
Size, Type and Material:	N/A
Headwall:	N/A
Joint Separations:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None observed or reported.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed at the Red Water Pond since the last annual inspection.

- Approximately 1,800 linear feet of rock buttress was constructed along the east dike form the north end to the pump station (Photo 26; Items 108-109). The water elevation of the Red Water Pond was lowered for the construction of this buttress. Construction was still ongoing at the time of the annual inspection.
- A temporary pump station was built to pump down the pool elevation to facilitate the rock buttress construction discharging the water to the Main Ash Pond A rim ditch (Photos 9, 21, 27; Item 110).
- The interior slopes of the pond have been mowed.
- The cluster of trees and root systems has been removed from the north end of the western interior slope. However, new woody vegetation has established along the slope (Photo 8).
- An access ramp to the north dike has been covered in crushed stone and the access stairway has been removed (Photo 28; Item 111).

6. Recommendations

Based on the yearly inspection, the following maintenance recommendations are offered

for the Red Water Pond. Priority codes are included in parentheses and described in Enclosure M.

- Continue to mow the interior slopes as needed.
- The uprooted tree along the west interior dike should be removed and the depression filled with compacted clay (Photo 6; Item 37; Priority 4).
- The inlet pipes along the west interior slope below the railroad tracks should be cleared of vegetation and the erosion/scouring should be repaired (Photo 7; Items 38-39; Priority 4).
- Remove the small trees and root systems along the north dike interior slope in accordance with the guidelines provided in Enclosure N (Photo 8; Item 107; Priority 4).
- Corroded pipes penetrating the perimeter dike located on the 620 bench were observed and have previously been recommended to be abandoned by URS as noted in the 2011 second quarter dike inspection report (Photos 14-16; Items 34, 71-74; Priority 4).
- Areas lacking vegetation were noticed on the east interior slope and the west end of the north dike crest. It is recommended that these locations be seeded to re-establish vegetation (Photos 4, 19, 20; Priority 4).
- Cracking was observed in the asphalt roadway and the guard rail is dipping along the crest of the west dike (Photo 17; Items 40-42). The interior slope below the road should be monitored for any potential sloughs that could be causing the loss of pavement support (Priority 4).
- Continue to monitor the previously identified seepage area (Seep 7) located at the toe of the north dike (Photo 21; Priority 3).
- The inlet pipes to the south end of the Red Water Pond from under the roadway should be inspected and cleaned of debris (Photo 29; Items 43-44; Priority 4).
- A fallen tree obstructing access to a piezometer along the shoulder of the 626 bench access road should be removed (Photo 30; Item 33; Priority 3).



Photo 1

East dike interior slope between the Main Ash Pond A, looking south. Note the new rock buttress at the toe of the interior slope and sparse vegetation along tire rut tracks.

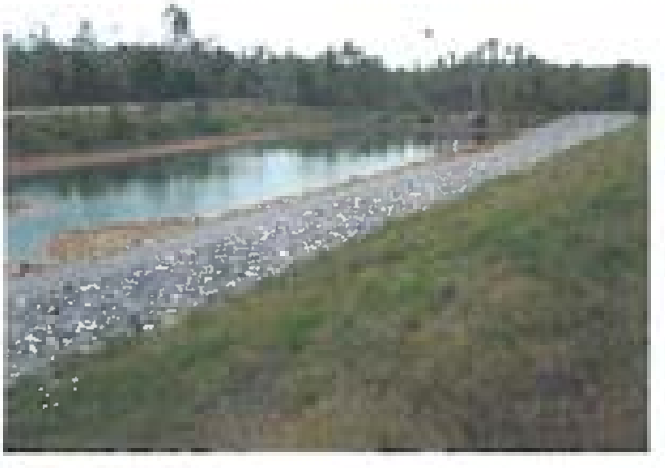


Photo 2

East dike and west dike interior slopes, looking north toward north dike. Note the new rock buttress at the toe of the interior slope and the pumped down pool elevation (item 108).



Photo 3

East dike and west dike interior slopes, looking south. Note the bush/woody growth along the west dike interior dike between the Red Water Pond and roadway.



Photo 4

Lack of vegetation and cracking in ruts formed by tracked mower along east dike interior slope.



Photo 5

View of Red Water Pond from the north dike, looking south. Note the increased of vegetation on the west dike interior slope, rock buttress on the toe of the east dike, and the pumped down pool elevation.



Photo 6

West dike interior slope below the railroad tracks. Uprooted tree with depression forming a possible sink hole (item 37).

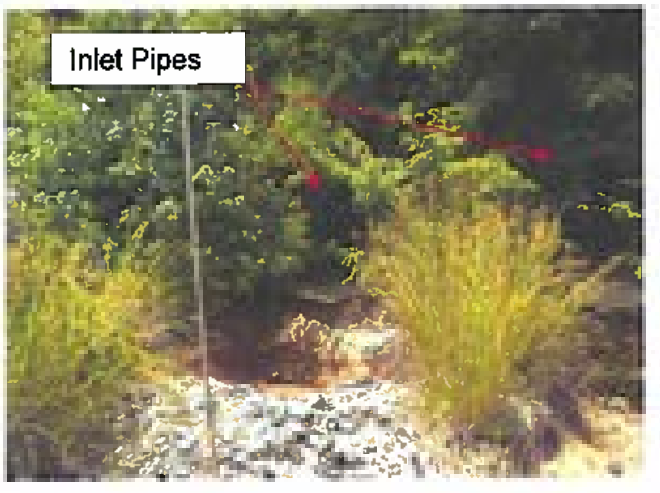


Photo 7

Inlet pipes through the west dike. Note dense brush/woody vegetation, and scouring below pipes on the interior slope (Items 38-39).



Photo 8

Vegetation at the north end of the west dike on the interior slope and large trees uprooted from the tornado damage below the north dike. Note the small trees growing on the interior slope of the north dike (Item 107).

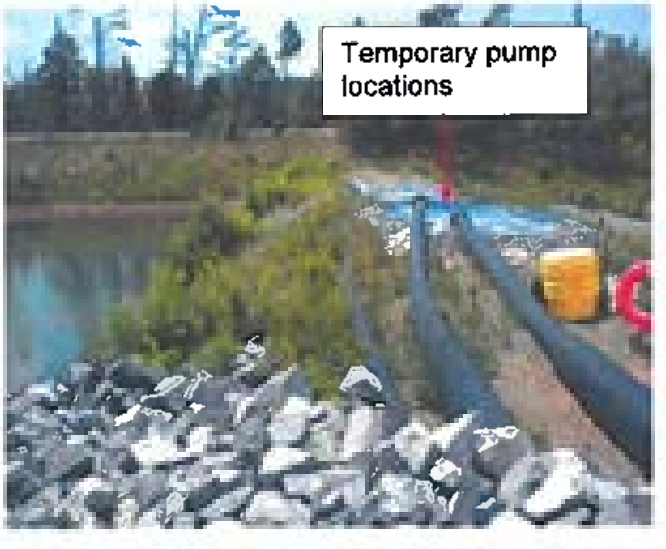


Photo 9

Vegetation covering wave wash protection on the interior slope of the north dike, pipe lines to the former pump locations on the crest of the north dike and new vegetative growth along the west dike in the background.

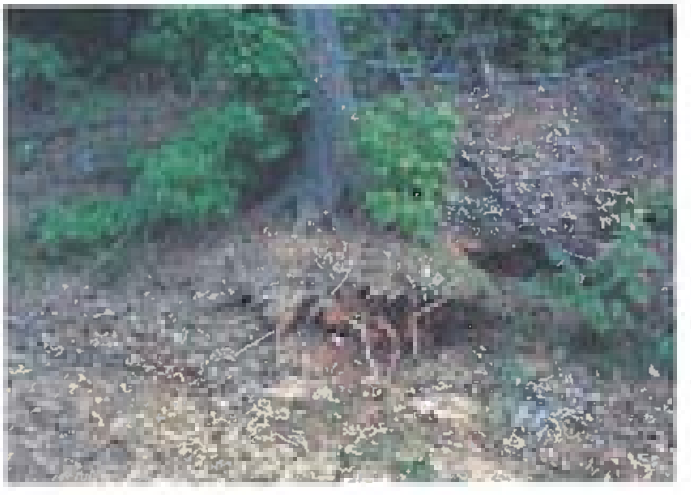


Photo 10

Erosion undercutting along on the interior slope at the south end of the east dike along the 620 bench.



Photo 11

Erosion rill located on the interior slope at the south end of the east dike below the 620 bench.



Photo 12

Erosion along a corroded 12" steel discharge pipe located on the 620 bench below the east dike.



Photo 13

Location of Corroded 12" steel discharge pipe located on the 620 bench below the east dike.



Photo 14

Potential instability interior slope of the east dike below the 626 bench. Note the12 inch corroded steel pipe penetrating the 626 feet bench.



Photo 15

Relatively steep (2H:1V) interior slope of the east dike below the 620 bench. Note the 12 inch corroded steel pipe penetrating the 620 feet bench.



Photo 16

Relatively steep (2H:1V) interior slope of the east dike below the 620 bench. Note the12 inch corroded steel pipe penetrating the 620 feet bench.



Photo 17

West dike crest cracks in asphalt roadway and dip in the guard rail associated with possible subsidence (Items 40-42).



Photo 18

West dike crest along the railroad tracks looking north. Note the rock buttress construction along the toe of the east dike.



Photo 19

Crest of the Red Water Pond/Main Ash Pond A covered in crushed stone/bottom ash.



Photo 20

Vegetation on the north dike crest and slopes looking east. Note the rip-rap crest coverage on the emergency spillway and lack of vegetation along the west end of the dike. Note the beginning of the rock buttress in the background (Item 108).

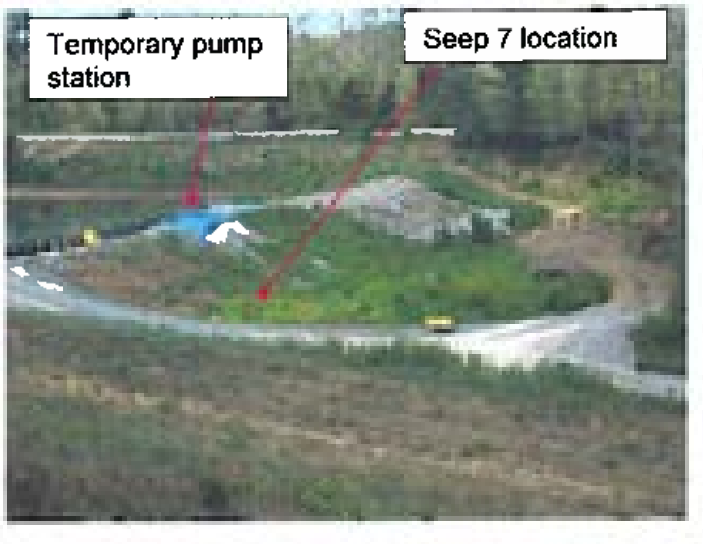


Photo 21

Exterior slope of the north dike. Note the vegetation in the center of the picture, rip-rap on the right side of the picture and pump lines and pump locations left side of the picture. Previously indentified Seep 7 was not evident at the toe of the dike. Note Main Ash Pond A pool has been lowered approximately five feet.



Photo 22

Red Water Pond Pump Station. Note lowered pool elevation and end of the rock buttress at time of inspection (Item 109).

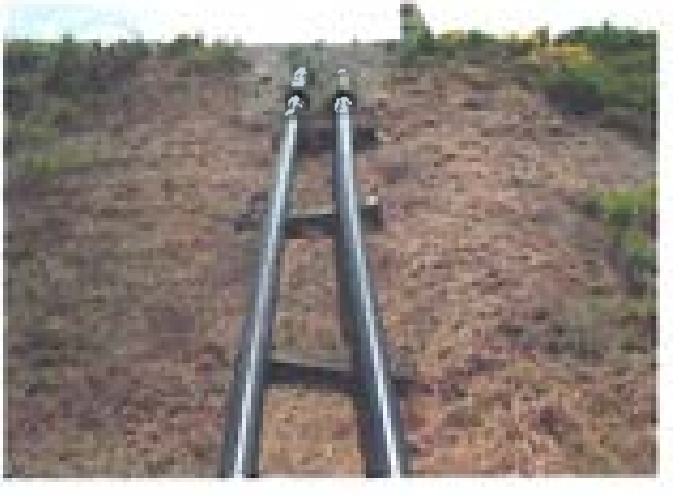


Photo 23

Two pipe lines from the pump station up the slope of the east dike.

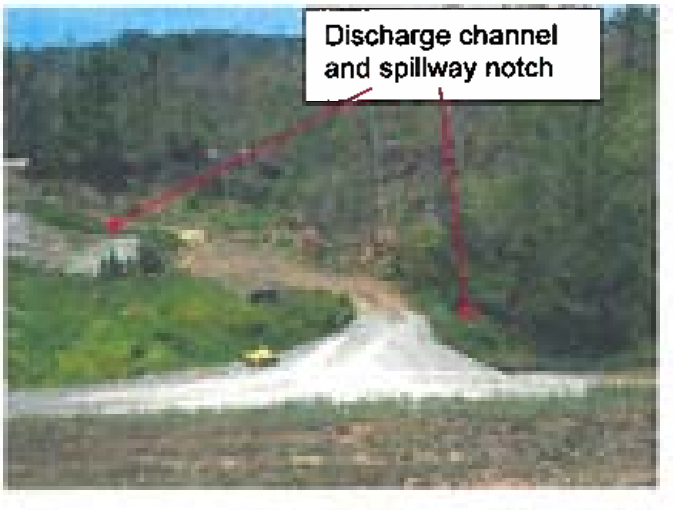


Photo 24

Red Water Pond emergency spillway notch and discharge channel.

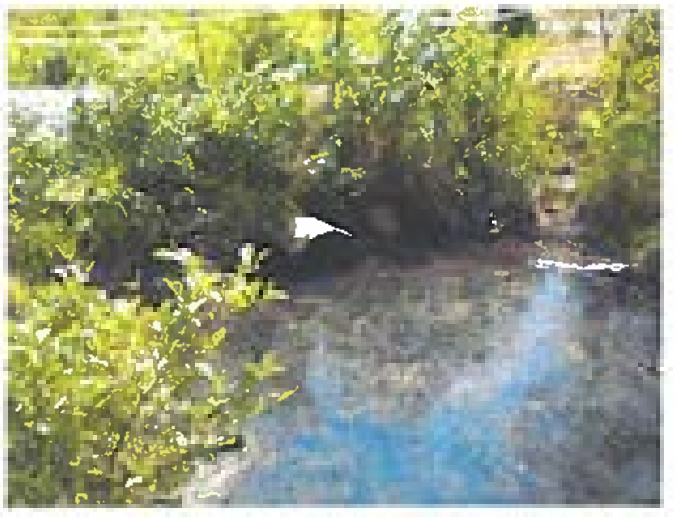


Photo 25

Discharge channel outlet pipes into the cove leading to Widows Creek. Note the condition of the corrugated metal pipe.



Photo 26

Rock buttress construction along toe of east dike (Items 108-109).



Photo 27

Outlet pipes from the temporary pump station along the north dike to the Main Ash Pond A rim ditch (Item 110).



Photo 28

New access ramp to the north dike from the crest of the east dike (Item 111).



Photo 29

Storm water inflow discharge culverts located at the southwest (Headwater) end of the Red water Pond (Items 43-44).



Photo 30

Fallen tree covering a previously installed piezometer on the east interior slope (Item 33).

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Feet
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400
800
Stantec
Plan View - Red Water Pond
WCF 2011 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
1 OF 1

Enclosure H

Gypsum Stack

Stantec

TVA 2011 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Gypsum Stack (DRAFT)

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	95 Acres	**TVA Hazard Classification:**	High
Maximum Height (toe to top of dike):	80 feet	**Dike Length:**	10,350 feet
Plant Discharge to Facility:	5,000 gpm (Unit 7/8 Combined)	**Current Pool Elevation:**	Pond 3 - 670.4 feet

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT, Robert Fuller, PE, Briggs Evans, and Greg Budd
TVA Staff Present:	Jake Booth, Ben Phillips, Stuart Harris
Field Inspection Date:	June 13, 2011
Weather/Site Conditions:	Sunny, 85 degrees, dry conditions

3. History/Current and Future Operations

History: Commissioned in 1986. Contains sluiced material from Units 7 and 8. A failure of an abandoned decant weir occurred on January 9, 2009 releasing gypsum - fly ash material into the Stilling Pond and Widows Creek. Material placed around a newly constructed spillway outlet pipe in Pond 2B washed out, causing gypsum - fly ash material to spill down the slope into the Stilling Pond on January 4, 2010. Since January, 2010, Pond 2A/2B has been dewatered and will remain dry with the use of pumps. Pond 3 will continue to operate as the only clarification pond on top of the stack.

Current Operations: The Gypsum Stack is currently being operated using wet handling methods in the east and west rim ditches, alternating between the two. Material is excavated from the rim ditch and rolled into windrows. This material, once dried, is then transported to the interior of Ponds 1 and 2A/2B where it is mechanically placed following the requirements outlined in the 12-month operations plan.

Future Planned Operational Changes:	URS Corporation is tasked with developing a closure plan for the Gypsum Stack facility.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	No vegetation present on any interior slopes (Photos 1-2).
Trees:	None observed.
Wave Wash Protection:	None observed.
Erosion:	Yes, small erosion rills located on all interior slopes of Pond 3, Pond 2A/B, and the rim ditch (Photo 3).
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	Five feet
Encroachments:	Material is currently being mechanically placed inside the limits of Pond 2A/2B.
Slope:	2H:1V

4.2. Crest

Crest Cover and Slope:	Material is actively being placed at the Gypsum Stack. Because of this, there is no vegetation/stone coverage on the crest of the stack. The crest is exposed gypsum - fly ash material (Photos 4-6).
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	Width ranges from 25 feet wide to over 100 feet wide.

4.3. Exterior Slopes

Vegetation: The exterior slopes of the stack generally have adequate coverage of grassy vegetation (Photos 7-12). However, there are a few areas of sparse grassy vegetation. Many of these areas were recently hydroseeded at the time of the inspection (Photo 11). Sparse vegetation is typical at the toe of the exterior dikes along the perimeter ditch (Photo 10). Additionally, there are areas of sparse vegetation/erosion along the edge of the west exterior slope and the rock buttress above the Stilling Pond (Photo 9).

Trees: None observed.

Erosion: Erosion features were noted throughout the exterior slopes around the perimeter of the stack from the crest to the toe (Photos 13-20; Items 1, 2, 5-8, 10, 14). Erosion rills are typical due to the lack of a bench drain system. RHO&M repairs erosion on a regular basis when the erosion rills become large enough to warrant a repair.

Erosion was also present along the inside of the south access road (Photo 21; Item 112). There is not a well defined roadside ditch which causes storm water runoff to erode the interior of the access road.

Instabilities: None observed.

Uniform Appearance: The exterior slopes are generally uniform around the perimeter of the stack.

Seepage: Seepage is evident around the perimeter of the stack. Two possible new seeps were identified during the 2011 TVA 3rd Quarter Dike Inspection. One was located on the northwest side of the stack along the perimeter road (Photo 25; Item 13) and the other was located at the base of the haul road north of the stilling pond (Photo 26; Item 9). Additionally, numerous seeps previously identified during TVA quarterly dike inspections were noted. See photos 22 through 34 for a description of each seep location.

Benches: The lowest bench around the perimeter of the stack is located at the top of the original clay starter dike at

	elevation 620 feet on the west side of the stack which linearly increases in elevation to approximately 628 at the northwest corner (Photo 34). The first tier bench around the perimeter of the stack is located at elevation 655 feet on the west, north, and east sides of the stack, transitioning to elevation 650 on the south side of the stack (Photo 35-36). The grading plan for the stack is provided in TVA Drawing Series 10W235. The benches are relatively flat with no provisions for handling storm water runoff.
Foundation Drains, and Seepage Collection Systems:	The seepage collection system for the Gypsum Stack is depicted on TVA Drawing Series 10W235 showing a total of fifteen planned outlets. Two outlets of this system located along the west dike of Pond 2B between the Gypsum Stack and Stilling Pond have not been installed. The typical headwall for the outlet system has up to eight outlet pipes. The two outer-most pipe outlets (#1 and #8) are the laterals for the lowest toe drain. Moving inward, the next two pipes (#2 and #7) are the laterals for the first tier (650/655) bench seepage collection system. Based on the design drawings, the four inside outlet pipes are planned laterals at the 685 and 720 bench levels. See photos 37 to 49 for a picture and description of each outlet (Items 52-58, 60-64). Additionally, observation wells installed above the seepage collection systems have been painted (Photo 50). One of the observation wells located above seepage collection system 4 was broken and missing the cover (Photo 51; Item 3).
Instrumentation:	Piezometers are installed around the stack and typically are protected by a steel cover, concrete pad and steel bollards (Photo 52). Three slope inclinometers are installed. One is located near the midpoint of the north dike above the perimeter ditch, one is located on the west dike north of the stilling pond above the perimeter ditch, and one is located on the divider dike between Pond 2A/B and Pond 3.
Animal Burrows:	None observed.
Slope:	The slope ranges from 2.4H:1V to 2.8H:1V below the 655 bench around the perimeter of the stack. On the west dike between Pond 2A/B and the Stilling Pond, the slope below the 655 bench is 4H:1V. The slope is 2.5H:1V

	around the perimeter of the stack above the 655 bench. The slope of the starter dike is 3H:1V.
Height:	The maximum height of the Gypsum Stack is approximately 80 feet measured on the south side of the stack.

4.4. Spillway Weirs/Riser Inlets

Number:	Two (2), one in Pond 2B and one in Pond 3 (Photos 53-54).
Size, Type and Material:	48 inch diameter concrete riser.
Height of Riser Inlets:	The approximate height of the riser in Pond 2B is 15 feet and the approximate height of the riser in Pond 3 is 6 feet.
Access:	No access to the riser inlets.
Joints:	Unable to be observed.
Mis-Alignment:	No issues observed.
Closed/Abandoned Conduits:	Work Plans 5 and 6 were completed in 2009. As part of these work plans, conduits which were found penetrating through the perimeter dike were grouted and abandoned in place. For a complete inventory of existing conduits, reference is made to the WCF Pipe Inventory Report dated April, 2010.

4.5. Outlet Pipes

Number:	Two (Photo 55)
Size, Type and Material:	30 inch diameter solid HDPE pipe
Headwall:	None observed.
Joint Separations:	Unable to be observed.
Mis-Alignment:	No issues observed.
Closed/Abandoned Conduits:	See 4.4 for discussion on abandoned conduits.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Gypsum Stack since the last annual inspection.

- The exterior slopes of the stack have been mowed. Woody growth noted on the previous annual inspection report was no longer present.
- An erosion rill/slough was repaired on the southeast slope below the 655 bench. The repair consisted of placing sand and stone along the exterior slope from the perimeter ditch to the 655 bench (Photo 56; Item 113).
- Erosion rills and ruts due to surface water runoff on the exterior slopes of the stack were repaired by filling in the rills/ruts with compacted clay. The repairs were hydroseeded to establish vegetation on the slopes (Photo 57).
- The crest around the top of the stack surrounding Pond 2A/B and Pond 3 has been widened since the last annual inspection.
- Immediate Risk Reduction Plans (IRRP) have been installed at various locations around the Gypsum Stack. A seepage blanket/rock buttress consisting of sand, small stone, and medium stone (Photo 59) has been installed below the 620 feet elevation perimeter road. This has been installed along the southeast slope for approximately 800 linear feet (Photo 58; Items 66-67), the southwest slope for approximately 650 linear feet (Photos 59 and 23; Items 68 and 69), and the west slope north of the stilling pond, along the entire northwest slope to the northern corner of the stack for approximately 3,400 linear feet (Photos 60-63; Items 59 and 65). Instrumentation was protected during the construction of the IRRP by extending the piezometer standpipes and placing a steel casing around the piezometer above the newly installed rock buttress.
- Various seepage repairs were completed. These repairs are discussed below. See the current Seepage Log for the locations of seepage areas/repairs at the Gypsum Stack.
- Seep 20 was repaired by constructing a rock buttress approximately 120 feet in length and 10 feet in elevation (Photo 34).
- Seep 23 was repaired by constructing a seepage blanket as an extension to the IRRP (Photo 23).
- Seep 27 was repaired by constructing a rock buttress approximately 250 feet in length and 5 feet in elevation (Photo 28)
- Seep 28 was repaired by constructing a rock buttress approximately 100 feet in length and 5 feet in elevation (Photo 30).

- Seep 34 was repaired by constructing a rock buttress approximately 35 feet in length and 5 feet in elevation (Photo 34).
- Signs have been installed around the stack. These signs include warnings of high voltage overhead (Photo 64), seepage areas, and traffic control.
- Multiple monitoring wells which were damaged during the previous annual inspection have been repaired by extending the PVC pipe and placing new covers. The covers have been painted yellow (Photo 50).
- Piezometers have been protected by installing bollards and protective steel casing (Photo 52). Not all piezometers were protected at the time of the annual inspection.

6. Recommendations

URS Corporation is moving forward with design of gypsum stack modifications to reduce the risk of slope stability failure, improve operational conditions, and to ultimately achieve closure. The initial design efforts will address slope stability and seepage issues previously identified primarily around the perimeter clay dike and lower portions of the stack below elevation 650 feet.

The recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design and operation issues currently being addressed by URS and Stantec. TVA Dam Safety priority codes are included in parenthesis and described in Enclosure M.

- Continue to mow the exterior slopes as needed.
- It is recommended that all instrumentation be protected with bollards and steel casing. It was Stantec's understanding that this would occur during the instrumentation automation project. Depending on the status of that project, it may be necessary to install protective bollards and casing prior to the automation similar to that shown in Photo 52 (Priority 4).
- Erosion should be repaired in accordance with the general repair guidelines included in Enclosure N (Photos 13-20; Items 1, 2, 5-8, 10 and 14). On the west dike, the silt fence should be reinstalled once erosion repairs are completed to prevent gypsum from washing down the exterior slope (Photo 18; Items 6-7; Priority 4)
- The ditch along the access road on the south side of the stack should be improved/maintained. Currently, there is erosion along the interior groin where the road and exterior slope are joined due to the lack of a defined roadside ditch (Photo 21; Item 112; Priority 4).
- The damaged observation well should be repaired in the same manner as other observation wells. The pipe should be extended and a new yellow cover should be placed on top of the well (Photos 50-51; Item 3; Priority 4).

- Seep 5 should be investigated in the field. There are similar locations around the stack at the same elevation of Seep 5 which has an 8" diameter outlet pipe with water flowing from the pipe into the perimeter ditch. The area around Seep 5 should be carefully excavated/probed to determine if a buried pipe is located in this area (Photo 22; Priority 3).
- The two new seeps identified during the 3rd quarter inspection (Photos 25-26; Items 9 and 13) should be monitored for any changes in flow or sediment transport. Previously identified seeps should also be monitored in accordance with the Seepage Action Plan.



Photo 1

View of the rim ditch where it flows into Pond 3 through an approximate 36" diameter pipe. Note the lack of vegetation on all interior slopes.



Photo 2

View of rim ditch at the north end of the stack. Note the lack of vegetation on the interior slopes/crest of the pond.



Photo 3

Typical erosion rills on the interior slopes of the rim ditch. These rills are common on all interior slopes with exposed gypsum – fly ash.



Photo 4

Outlet area of Pond 2A/2B. Material is being placed in the footprint of Pond 2A/2B. Storm water that accumulates is pumped from the pond into the outlet area into the spillway which flows into the stilling pond.



Photo 5

Crest on the west side of the stack. Note stacked exposed gypsum – fly ash.



Photo 6

Discharge pipes to the east rim ditch.



Photo 7

Vegetation on the eastern exterior slope of Pond 3. Note the perimeter ditch sediment deposition.



Photo 8

Vegetation on the southeastern exterior dike. Note the typical seepage collection system observation wells that have been recently painted and repaired.

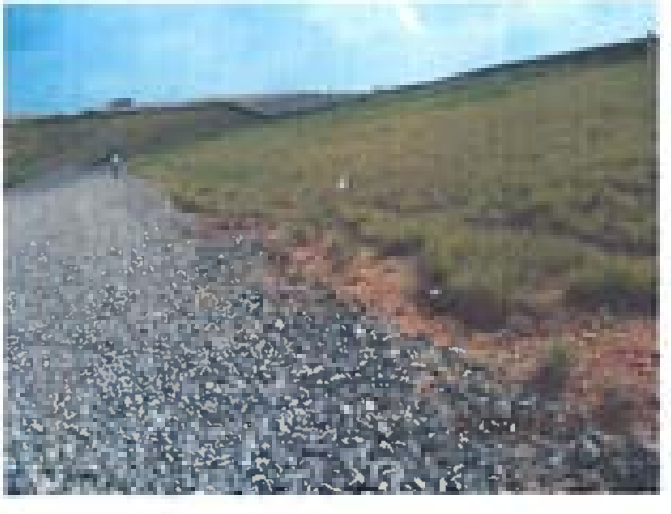


Photo 9

View of southwest exterior slope above the Stilling Pond and below the silt fence along the southwest crest. Note the dense grass vegetation and the erosion at the toe along the rock buttress (Items 5 and 8).

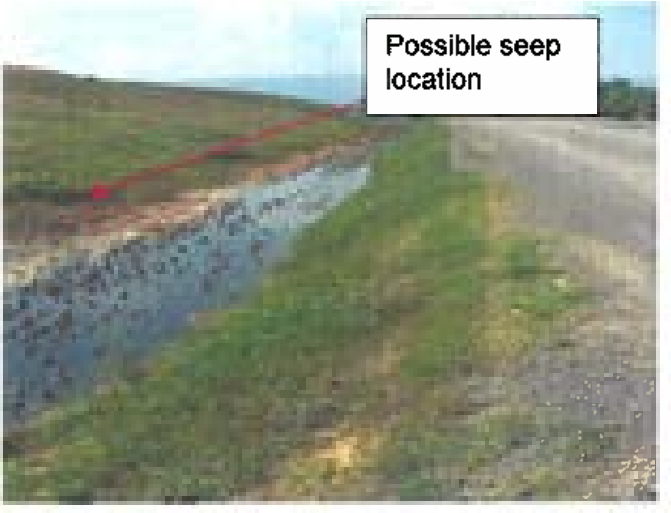


Photo 10

View of west exterior slope and perimeter ditch. Typical dense grass vegetation along the slope and erosion at the toe along the perimeter ditch. A possible seep was identified (Item 21).

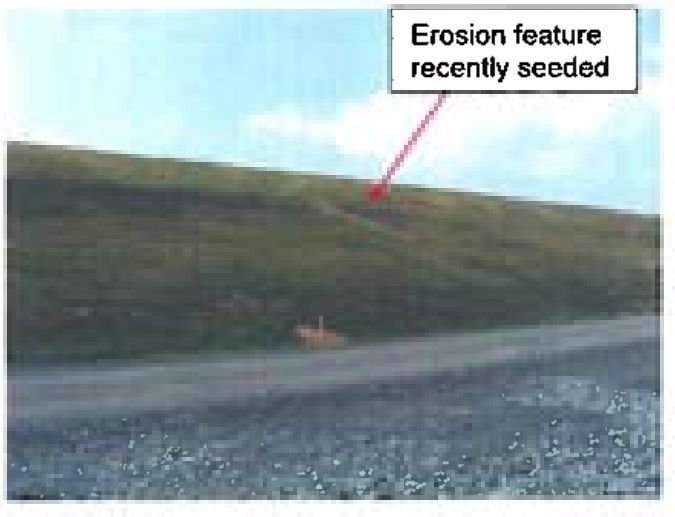


Photo 11

View of northwest exterior slope typical vegetation coverage and erosion features.

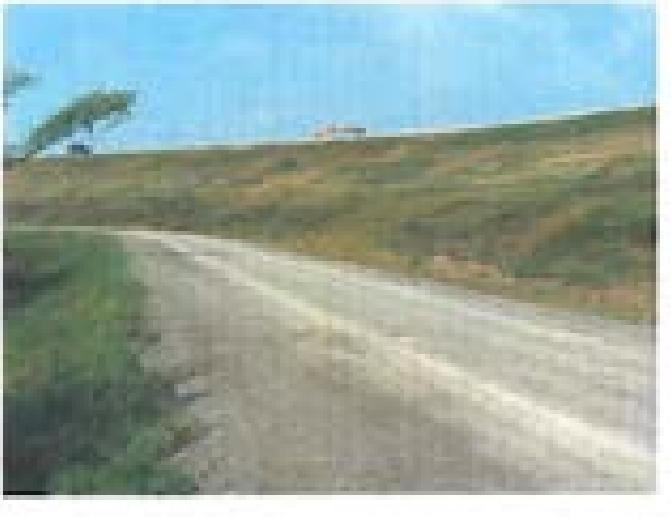


Photo 12

View of northeast exterior slope typical vegetation coverage. Note the sparse vegetation at the toe along the perimeter ditch.



Photo 13

Typical Erosion repair lacking vegetation along the east exterior slope.



Photo 14

Erosion at the top of the northwest exterior slope and crest.



Photo 15

Erosion along northwest exterior slope. These small rills are typical and are generally repaired once the erosion rills are sufficiently large.



Photo 16

Erosion and standing water along the 650 bench of the southeast exterior slope (Item 1).

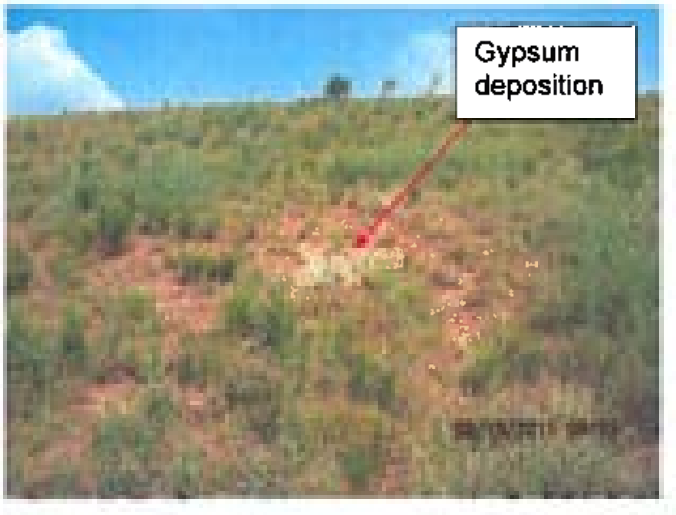


Photo 17

Erosion with evidence of gypsum deposition along the southeast exterior slope (Item 2).



Photo 18

Erosion at silt fence along the top of the southwest exterior slope above the Stilling Pond (Items 6-7).



Photo 19

Typical erosion at toe of southeast exterior slope and deposition in the perimeter ditch (Item 10).



Photo 20

Typical erosion at toe of northwest exterior slope along the perimeter ditch (Item 14).

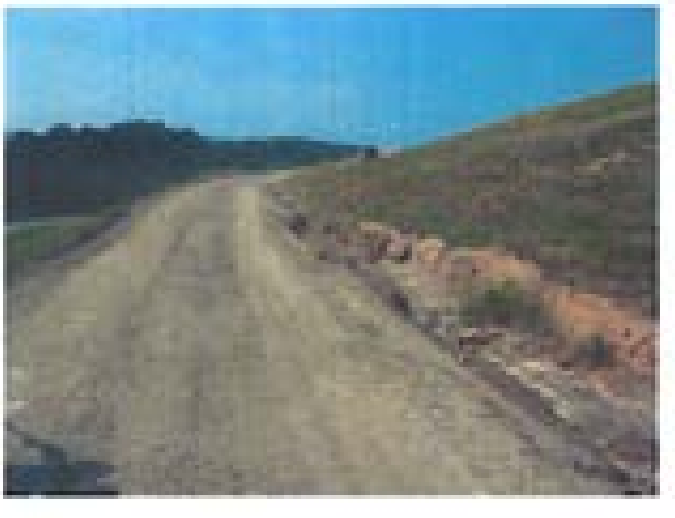


Photo 21

View of southeast exterior slope of the road to the 650 bench. Note the erosion along the exterior slope and edge of the road (Item 112).

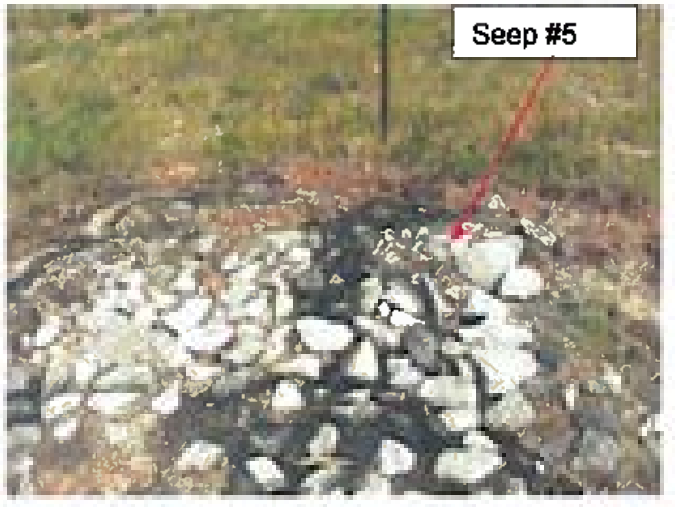


Photo 22

Seep 5 located on the south side of the Gypsum Stack.



Photo 23

View of the southwest dike, southwest buttress construction and the Seep 23 repair completed since the 2010 annual inspection.



Photo 24

Seep 29 located on the west exterior slope near the sluice line valves.



Photo 25

View of possible seep located along the perimeter access road (Item 13).



Photo 26

View of possible seep located at the base of the haul road near the Gypsum Stack Stilling Pond (Item 9).

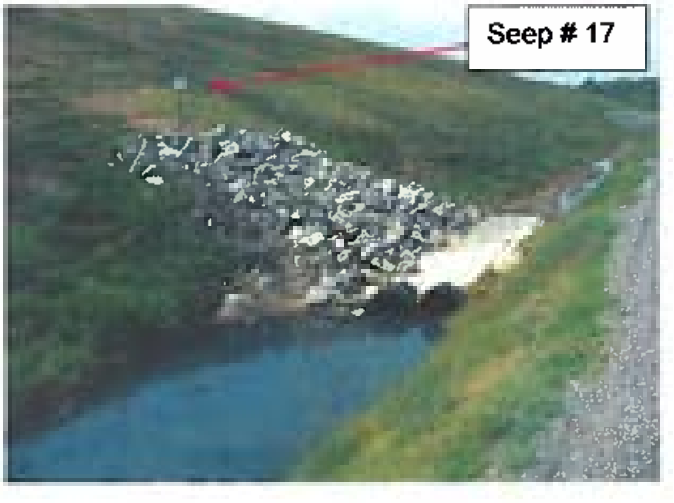


Photo 27

Seep 17 located on the northwest exterior slope.



Photo 28

Seep 27 located on the northwest exterior slope. This seep has been repaired since the 2010 annual inspection.



Photo 29

View of 10 inch pipe adjacent to Seep 27. Note the sediment at the outlet of the pipe.



Photo 30

Seep 28 located on the west exterior slope north of the Gypsum Stack Stilling Pond. This seep has been repaired since the 2010 annual inspection.



Photo 31

Seep 22 located on the southeast exterior slope below the 620 perimeter road.



Photo 32

Seep 25 located on the southeast exterior slope below the 620 perimeter road.



Photo 33

Seep 26 located on the southeast exterior slope below the 620 perimeter road.

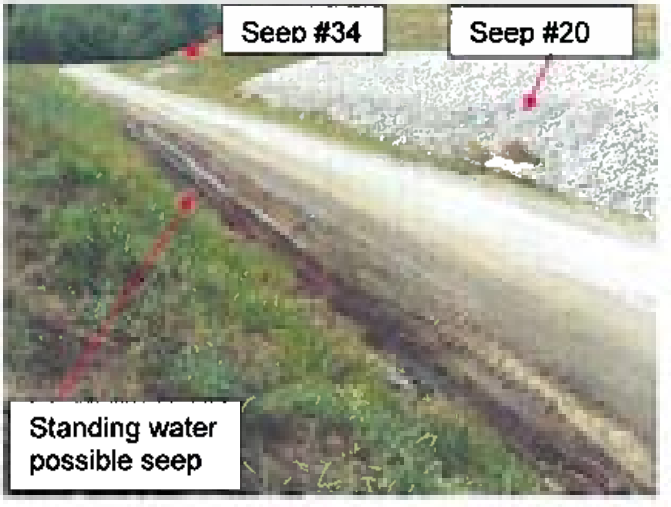


Photo 34

Perimeter road on the east side of Pond 2A/B. The perimeter road varies in elevation from 620 feet to 628 feet around the stack. Note the crushed stone coverage typical of the perimeter road, Seep 20 and Seep 34 repairs, and standing water in tire ruts from possible seep along the toe of the slope.

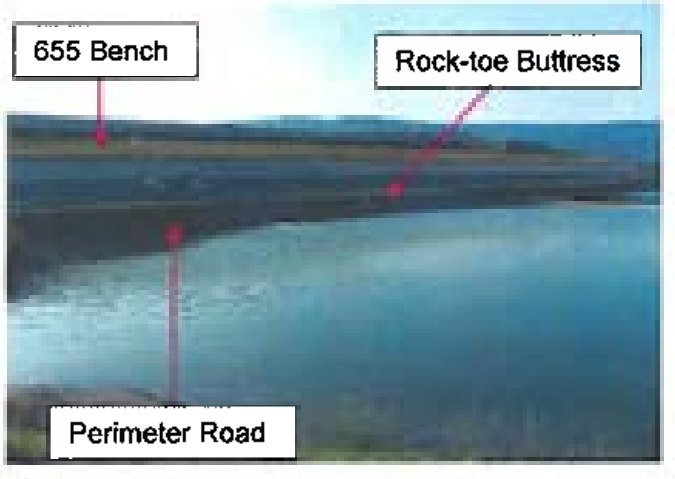


Photo 35

View of the west dike between Pond 2A/B and the Stilling Pond. Note the perimeter road and the 655 bench. Also note the rock-toe buttress unique to this location.



Photo 36

View of the 655 bench on the northwest side of Pond 3. Note the lack of vegetation and exposed clay typical of the 650/655 bench and the unidentified 1 inch pipe.



Photo 37

Seepage Collection System Outlet 3. The outlet pipes appear to be functioning.



Photo 38

Seepage Collection System Outlet 4. Pipes 1, 2, and 8 were flowing and pipe 7 had a slow trickle. It appears that both the toe drain and slope drain are functioning (Item 52).



Photo 39

Seepage Collection System Outlet 5. Pipes 1, 2, 7, and 8 were flowing. It appears that both the toe drain and slope drain are functioning (Item 53).



Photo 40

Seepage Collection System Outlet 6. Pipes 2 and 7 were flowing, while pipe 1 and 8 had no flow. It appears that the slope drain is fully functioning while the toe drain is not functioning, since outlet pipe 1 and 8 had no flow (Item 54).

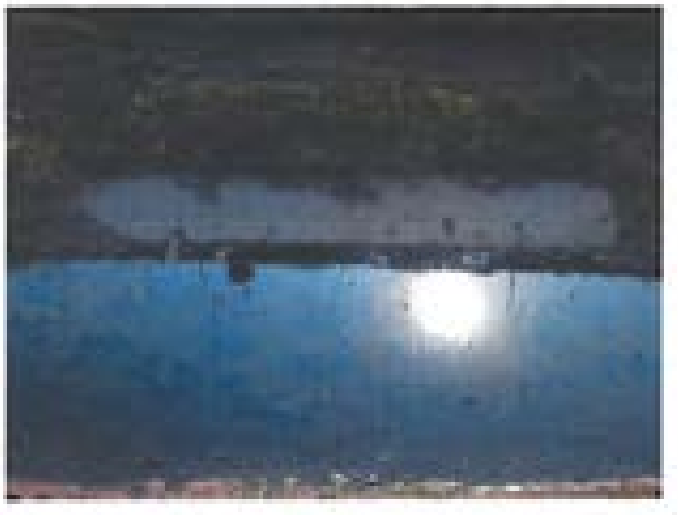


Photo 41

Seepage Collection System Outlet 7. The outlet pipes were submerged in the perimeter ditch so flow/functionality was undetermined (Item 55).

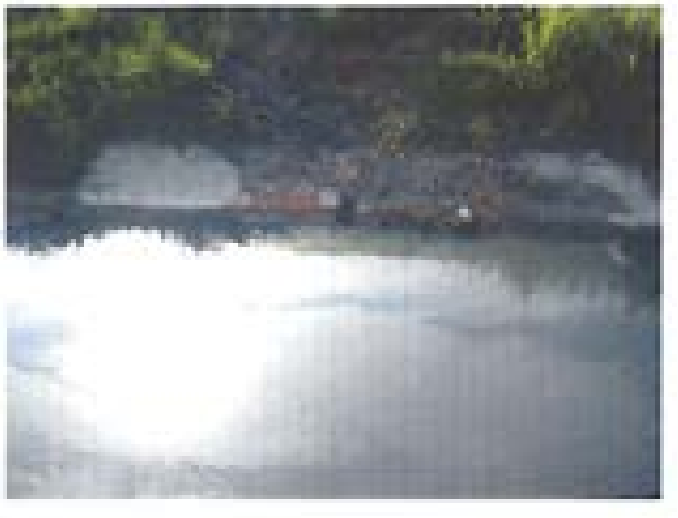


Photo 42

Seepage Collection System Outlet 8. The outlet pipes were submerged in the perimeter ditch so flow/functionality was undetermined (Item 56).



Photo 43

Seepage Collection System Outlet 9. The outlet pipes were submerged in the perimeter ditch so flow/functionality was undetermined. Note the rock material covering the headwall (Item 57).



Photo 44

Seepage Collection System Outlet 10. The outlet pipes were submerged in the perimeter ditch so flow/functionality was undetermined. Note the sediment deposition covering the headwall (Item 58).



Photo 45

Seepage Collection System Outlet 11. Pipes 1, 2, and 7 were flowing, while pipe 8 had no flow. It appears that the slope drain is fully functioning while the toe drain is partially functioning, since outlet pipe 8 had no flow (Item 60).



Photo 46

Seepage Collection System Outlet 12. The outlet pipes were submerged in the perimeter ditch so flow/functionality was undetermined (item 61).



Photo 47

Seepage Collection System Outlet 13. Pipe 1 was flowing, while pipes 2 and 7 had no flow. It appears that the toe drain is partially functioning while the slope drain is not functioning, since outlet pipes 2 and 7 had no flow (Item 62).

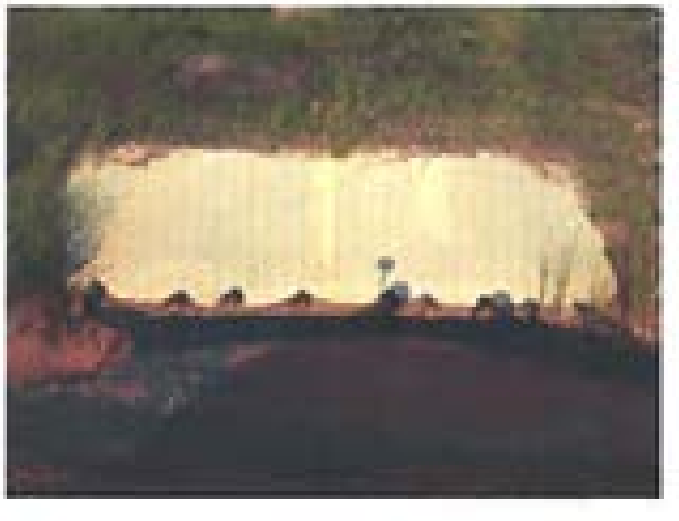


Photo 48

Seepage Collection System Outlet 14. The outlet pipes appear to be blocked with sediment along the edge of the perimeter ditch (Item 63).



Photo 49

Seepage Collection System Outlet 15. The outlet pipes were submerged in the perimeter ditch and vegetation is growing in the ditch so flow/functionality was undetermined. Note the boom placed in the perimeter ditch (Item 64).



Photo 50

View of typical observation well repairs/painting located above the Seepage Collection Systems.

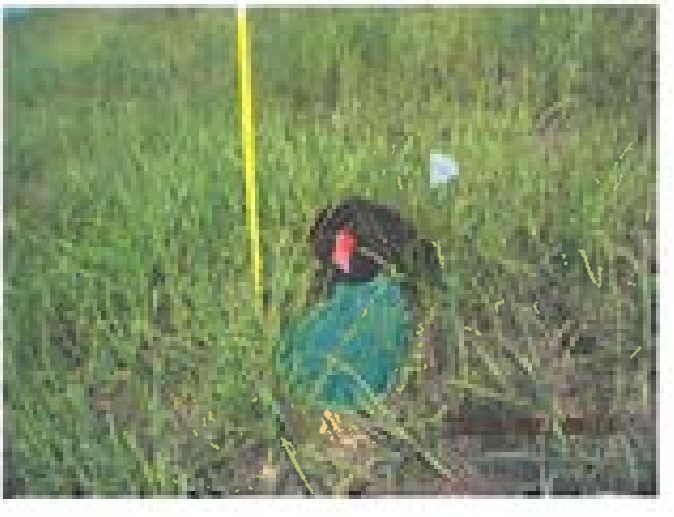


Photo 51

View of broken observation well on southwest exterior slope above Seepage Collection System Outlet 4 (Item 3).



Photo 52

View of typical piezometer concrete pad and bollard protection recently painted.



Photo 53

Pond 3 forty-eight inch diameter concrete spillway riser.



Photo 54

Pond 2A/B forty-eight inch diameter concrete spillway riser. Note this pond is currently dry. The outlet area is divided from Pond 2A/2B and storm water accumulated in the pond is pumped from the pond into the outlet pipe.



Photo 55

30 inch diameter spillway outlet pipes flowing into the Stilling Pond. Also note the 8 inch diameter toe drain outlet pipes and the 18 inch diameter future surface collection system outlet pipe.



Photo 56

View of erosion repair along the southeast exterior slope (Item 113).



Photo 57

Typical erosion repair along exterior slopes recently hydro seeded.



Photo 58

View of the beginning of the rock buttress construction along the toe of the southeast exterior slope. Note the access ramp to the transmission tower in the background (Item 66).



Photo 59

View of the rock buttress construction along the toe of the southwest exterior slope (Item 68).



Photo 60

View of the rock buttress construction along the toe of the northwest exterior slope from the Stilling Pond (Item 65).



Photo 61

View of the rock buttress construction along the toe of the northwest exterior slope. Note the piezometer protective case installed within the buttress.



Photo 62

View of the rock buttress construction along the toe of the northwest exterior slope.



Photo 63

View of the beginning of the rock buttress construction along the toe at the northwest/northeast corner of the exterior slope (Item 59).



Photo 64

New signage has been installed since the 2010 Annual Inspection.

N
Point Number and Direction
Feet
0
600
1,200
Stantec
Plan View - Gypsum Stack
WCF 2011 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
1 OF 1

Enclosure I

Gypsum Stack Stilling Pond

Stantec

TVA 2011 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Gypsum Stack Stilling Pond (DRAFT)

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	7 acres	**TVA Hazard Classification:**	High
Maximum Height (toe to top of dike):	18 feet	**Dike Length:**	3,000 feet
Plant Discharge to Facility:	5,000 gpm	**Current Pool Elevation:**	614.1 feet

2. Site Visit Information

Stantec Inspection Team: Chris Jones, EIT and Robert Fuller, PE, Briggs Evans, Greg Budd

TVA Staff Present: Jake Booth

Field Inspection Date: June 13, 2011

Weather/Site Conditions: Sunny, 85 degrees, dry conditions

3. History/Current and Future Operations

History: Commissioned in 1986 in conjunction with the Gypsum Stack. The spillways in Pond 2B and Pond 3 previously discharged directly onto the West Dike and ran down the exterior slope into the perimeter ditch, which discharged into the stilling pond. Work Plan 5 completed in 2009 routes the discharge of the spillways at the Gypsum Stack into a 30" diameter solid HDPE pipe discharging directly into the Stilling Pond.

Current Operations: The Stilling Pond is currently able to receive discharge from two spillway riser structures located in Pond 3 and Pond 2B of the Gypsum Stack. At this time, Pond 2B is dry, with only the riser structure in Pond 3 discharging process and storm water into the Stilling Pond through the new spillway outlet system installed as part of Work Plan 5. The Stilling Pond also receives flow from the Gypsum Stack perimeter ditch from two directions. The Stilling Pond discharges into a steel spillway with a skimmer structure where it flows to the 008 Outfall.

Future Planned None at this time.

Operational Changes:

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation: Dense grassy vegetation on the interior slope of the south and west dikes (Photos 1-2). Crushed stone coverage along the southeast dike (Photo 3). Vegetation lacking on the top half of the interior slope along the west dike (Photo 4). The north dike has sparse vegetation along the interior slope. Portions of the northeast dike are covered in crushed stone with some locations lacking coverage, resulting in exposed clay (Photos 5).

Trees: None observed.

Wave Wash Protection: Wave wash protection noted around the previously installed pipe outlets from Work Plan 5 on the northeast dike interior slopes and southeast interior slopes (Photos 3-5).

Erosion: Small erosion rills noted on the top half of the west dike interior slope where vegetation coverage is lacking (Photo 4; Item 114). Large erosion feature was noted in the northwest corner on the interior slope (Photo 6; Item 11). Erosion rills and sparse vegetation was noted on the interior slope of the southeast dike (Photos 6-7; Items 115-116).

Instabilities: Slough repaired on the interior slope of the south dike. Previously identified in the 2010 Annual Inspection (Photos 2 and 8; Item 117). Also, instabilities have been noted on the east interior slope. Some instabilities have been repaired by placing approximately 600 linear feet of stone along the interior slope beginning in the southeast corner (Photo 3; Item 118). Sloughing/erosion still appears to be an issue where stone has not been placed due to steep interior slopes.

Animal Burrows: None observed.

Freeboard: 5 feet, measured at the emergency spillway on the

	northern portion of the west dike.
Encroachments:	None observed.
Slope:	2H:1V

4.2. Crest

Crest Cover and Slope:	The crest is covered in crushed stone on each dike (Photos 9-10). The crest appears to slope away from the pond on the south, west, and north dikes, and towards the pond on the east dike.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed
Width:	25 feet on the east, north, and west dikes. 30 feet on the south dike.

4.3. Exterior Slopes

Vegetation:	Dense native grasses noted on the south dike exterior slope and portions of the west dike exterior slope (Photos 11-12). Sparse vegetation noted on top half of the west dike at certain locations along mower tracks (Photo 11).
Trees:	None observed.
Erosion:	None observed.
Instabilities:	None observed.
Uniform Appearance:	Yes.
Seepage:	Non-flowing seepage is evident at the toe of the west exterior slope (Photos 13).
Benches:	None observed.
Foundation Drains, and Seepage Collection Systems:	None installed.
Instrumentation:	None installed.

Animal Burrows:	None observed.
Slope:	3H:1V
Height:	15 feet

4.4. Spillway Weirs/Riser Inlets

Number:	There are no spillway weirs/riser inlets for the Gypsum Stack Stilling Pond. The outlet system for the pond consists of an outlet pipe with a skimmer barrel. This is discussed in 4.5 below.
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None observed or reported. There is a planned emergency overflow notch located in the northwest end of the Stilling Pond (Photo 10). A part of the Phase 2 CCP facility assessments, it has been recommended that the notch and downstream slope and channel be re-established using crushed stone lining. URS is developing these plans.

4.5. Outlet Pipes

Number:	One (1)
Size, Type and Material:	30 inch diameter steel pipe with a 42 inch diameter steel skimmer barrel (Photo 14).
Headwall:	N/A
Joint Separations:	None observed.
Mis-Alignment:	No issues observed.
Closed/Abandoned Conduits:	None observed or reported.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Gypsum Stack Stilling Pond since the last annual inspection.

- The interior and exterior slopes of the pond were mowed.
- Cenospheres floating on the pond surface are being captured with a series of floating booms. The removal of cenospheres by vacuum truck continues on a periodic basis as needed.
- Stone has been placed along the eastern interior slope where instabilities were observed in the spring of 2011 (Photo 3; Item 118).
- The slough which was noted in the 2010 annual inspection along the southern interior slope has been repaired (Photos 2 and 8; Item 117).

6. Recommendations

URS Corporation is moving forward with a closure design for the Gypsum Stack including the Gypsum Stack Stilling Pond. These design efforts will address slope stability and seepage issues previously identified. Therefore, the recommendations that follow are primarily related to maintenance items discovered during the annual inspection, and do not include the design issues currently being addressed by URS. Priority codes are included in parentheses and described in Enclosure M.

- Seed the locations on the interior slope of the west dike to re-establish vegetation. A new fertilizer/seed mixture may need to be developed if vegetation is unable to be established in this area (Priority 4).
- Continue to practice good cenosphere management by continuing to use floating booms (turbidity curtains) across the pond and cleaning the floating cenospheres from the pond surface as required.
- Repair the erosion rills noted on the interior slopes (Photos 6 and 7; Items 11, 114-116; Priority 4).
- The planned emergency overflow notch is located in the northwest end of the Stilling Pond (Photo 10). It is recommended that the notch and downstream slope and channel be re-established using crushed stone lining (Priority 5).

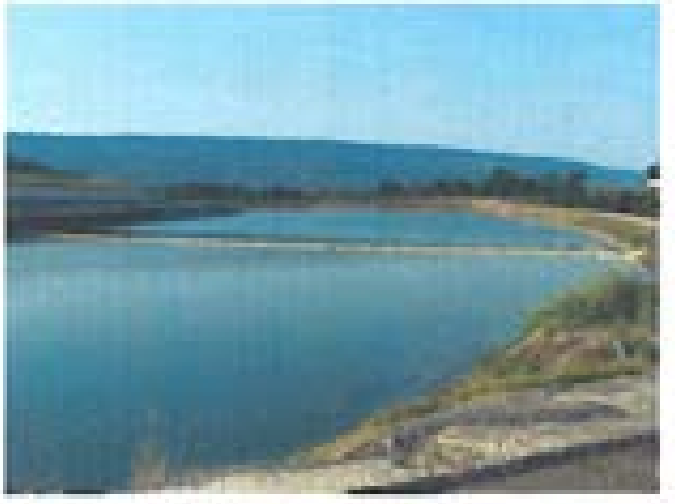


Photo 1

View of the Gypsum Stack Stilling Pond from the north dike, looking south. Note the floating booms for cenosphere management.

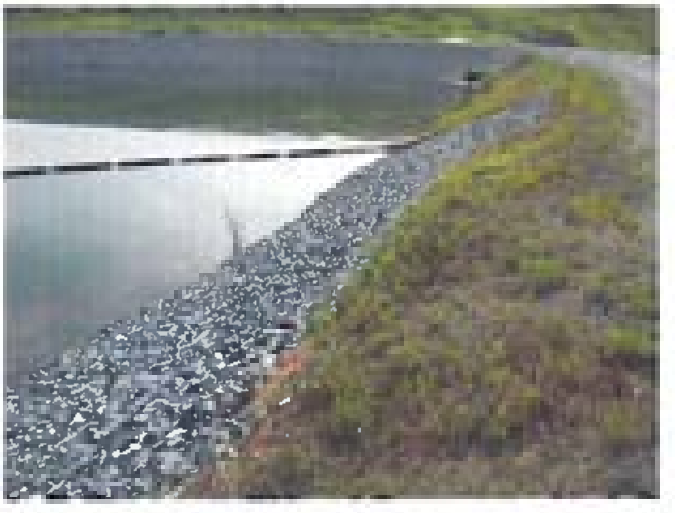


Photo 2

Interior slope of the south dike looking west. Note the slough repair in the southeast corner previously identified in the 2010 Annual Inspection (Item 117).



Photo 3

Interior slope of the east dike. Note the crushed stone coverage along the interior slope.



Photo 4

Lack of vegetation and erosion rills on the top half of the west dike interior slope (Item 114).



Photo 5

Wave wash protection around the outlet pipes into the stilling pond on the northeast interior slope. Note the lack of coverage in some locations.



Photo 6

North interior slope erosion feature (10' L x 3' W x 2' D; Item 11).

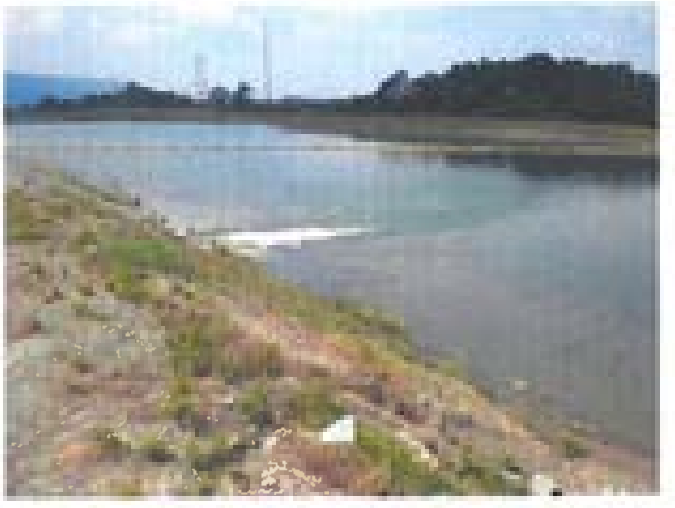


Photo 7

Erosion rills and sparse vegetation on the interior slope of the southeast dike (Items 115-116).



Photo 8

Slough repaired on the interior slope of the south dike. Previously identified in the 2010 Annual Inspection (Item 117).

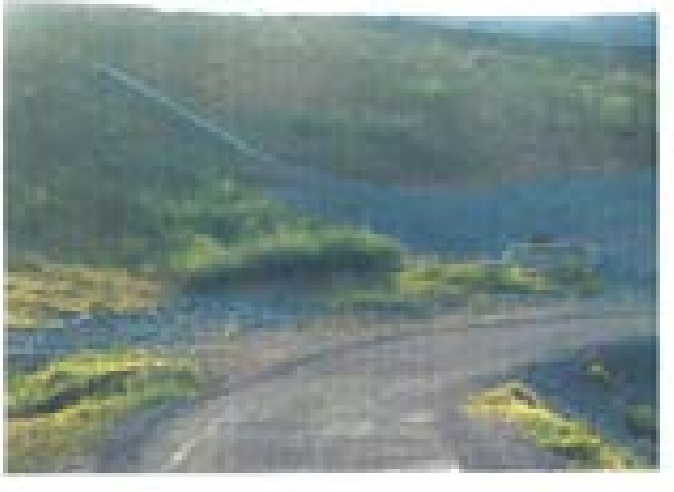


Photo 9

Crest of the north dike/east dike in the northeast corner of the pond. Note the Pond 3 spillway outlet pipe on the exterior slope of the stack and manhole.



Photo 10

Crest of the west dike with crushed stone coverage. Note the sparse grass vegetation on the interior slope. The planned emergency overflow notch is located at this location.

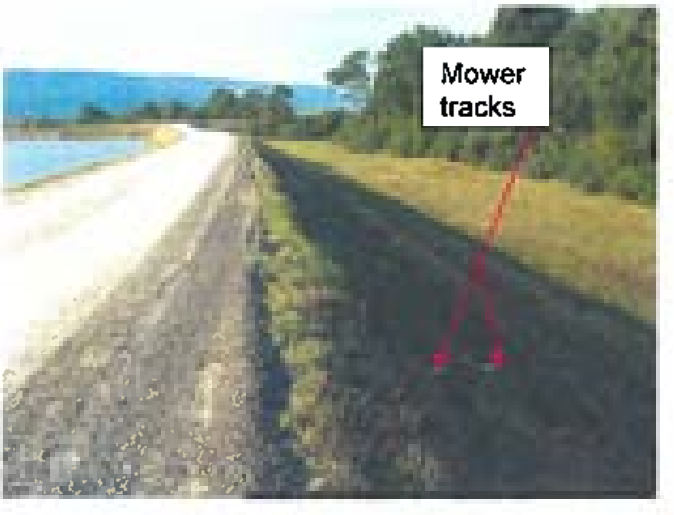


Photo 11

Exterior slope of the west dike with grass vegetation on the exterior slope. Note sparse areas of vegetation along mower tracks.

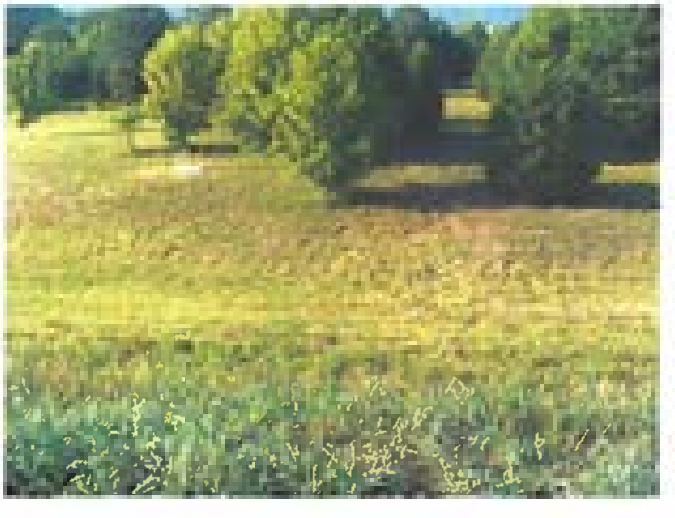


Photo 12

View of the south dike exterior slope.



Photo 13

Non-flowing seepage on the toe of the west dike exterior slope.



Photo 14

New skimmer barrel installed in May 2009 under Work Plan 3.

N
Gypsum Stack
Settling Pond
Feet
0
400
Stantec
Plan View - Gypsum Stack Settling Pond
WCF 2011 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
1 OF 1

Enclosure J

Limestone Drainage Basin

TVA 2011 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Limestone Drainage Basin (DRAFT)

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	LRP: 0.5 acres LRSP: 0.2 acres	**TVA Hazard Classification:**	N/A
Maximum Height (toe to top of dike):	LRP: N/A LRSP:10 feet	**Dike Length:**	LRP: 625 feet LRSP: 400 feet
Plant Discharge to Facility:	N/A	**Current Pool Elevation:**	605.6 feet

2. Site Visit Information

Stantec Inspection Team:	Greg Budd, Briggs Evans
TVA Staff Present:	Jacob Booth
Field Inspection Date:	June 13, 2011
Weather/Site Conditions:	Sunny,85 degrees, dry conditions

3. History/Current and Future Operations

History: The Limestone Drainage Basin was first shown on plans in 1973 (TVA Drawing 10N8221), with the latest revision to add the Limestone Runoff Surge Pond (LRSP) in 1984 (TVA Drawing 10N8224-1).

Current Operations: The Limestone Drainage Basin consists of two ponds, the Limestone Runoff Pond (LRP) and Limestone Runoff Surge Pond. The Limestone Runoff Pond collects surface runoff from the limestone stockpile area and an upland channel containing sluice lines. There is one 48 inch diameter inlet pipe from the slurry line ditch and an inlet trench from the limestone stockpile area. The Limestone Runoff Pond is hydraulically connected to the Limestone Runoff Surge Pond via two 30 inch diameter corrugated metal pipes and outlets to the engineered wetlands. The Limestone Runoff Surge Pond collects runoff from the Limestone Ball Mill via a ditch and a 24 inch diameter corrugated metal pipe. The outlet pipes from the Limestone Runoff Pond to the engineered wetlands are shown on TVA Drawing 10N8221 but were not found and

verified in the field.

Future Planned Operational Changes:	Stantec is unaware of any planned future operational changes at this time.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	For the Limestone Runoff Pond, sparse to dense grass coverage on the interior slopes (Photos 1-3). The Limestone Runoff Surge Pond has dense grass/brush coverage on interior slopes.
Trees:	None observed.
Wave Wash Protection:	None observed.
Erosion:	Substantial erosion rills noted at two locations on the western interior slope of the Limestone Runoff Pond (Photos 4-5; Item 45). Smaller rills noted at various locations where there is a lack of vegetation on the interior slopes.
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	Limestone Runoff Pond: 4 feet Limestone Runoff Surge Pond: 4 feet
Encroachments:	Sediment buildup at the inlet ends of both ponds.
Slope:	1.5H:1V to 2H:1V - Limestone Runoff Pond 3H:1V - Limestone Runoff Surge Pond

4.2. Crest

Crest Cover and Slope:	Crest covered in crushed limestone with adequate coverage, along the Limestone Runoff Pond, sloping toward pond interior. Limestone Runoff Surge Pond grass covered with adequate coverage, sloping toward pond interior.
Erosion:	Small rills along western crest of limestone drainage

	basin.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	25 feet

4.3. Exterior Slopes

Vegetation:	The drainage basin is situated in a depression. The only exterior slope is on the southeast dike of the Limestone Runoff Surge Pond. The slope is densely vegetated with mature trees and has large (5' to 10' diameter) boulders piled along the exterior toe.
Trees:	Densely vegetated with trees.
Erosion:	None observed.
Instabilities:	None observed.
Uniform Appearance:	There does not seem to be any method to the grading of the exterior slope, which cannot be observed due to the large pile of boulders scattered along the slope.
Seepage:	Yes, a seep is still present (Photo 6).
Benches:	None
Foundation Drains, and Seepage Collection Systems:	None observed.
Instrumentation:	None observed.
Animal Burrows:	None observed.
Slope:	2H:1V
Height:	20 feet (Southeast Dike, LRSP)

4.4. Spillway Weirs/Riser Inlets

Number:	There are no spillway weirs/riser inlets for the Limestone Runoff Pond or Limestone Runoff Surge Pond.

Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None reported or observed.

4.5. Outlet Pipes

Number:	LRP: Two (2) 18-inch diameter pipes, under water (Unable to be observed in field, information from TVA Drawing 10N8221 indicates invert elevation of 602 feet). LRSP: Two (2) 30-inch corrugated metal pipes through the northwest dike into the Limestone Drainage Basin (Photos 7-8).
Size, Type and Material:	LRSP: Two (2) 30-inch corrugated metal pipe LRP: two (2) 18-inch pipe (unknown material)
Headwall:	None.
Joint Separations:	None observed.
Mis-Alignment:	None observed.
Closed/Abandoned Conduits:	None reported or observed.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

There does not appear to be any new construction activities since the 2010 annual inspection.

6. Recommendations

Based on the yearly inspection, the following maintenance recommendations are offered for the Limestone Drainage Basin. Priority codes are included in parentheses and described in Enclosure M.

- Remove the debris floating in the Limestone Runoff Surge Pond (Photo 9; Priority 5)

- Clean out the trench running from the limestone stockpile to the Limestone Runoff Pond which has filled in with sediment (Photo 10; Item 46; Priority 5).
- Repair the damaged 48-inch diameter corrugated metal pipe in the southern corner of the Limestone Runoff Pond which was noted in the 2010 annual inspection. It appears that a hole developed in the top of the pipe, causing the material above the pipe to collapse into the pipe, blocking flow (Photos 11-14; Item 47). Remove sedimentation and vegetation blocking the inlet end of this pipe and protect inlet with rip-rap (Photo 13; Item 48; Priority 4).
- Two localized areas of erosion were noted during the inspection on the western dike of the Limestone Runoff Pond (Photos 2, 4, and 5; Item 45). It is recommended that these be filled/graded. Also, smaller rills were noted along the entire interior slopes. These should also be filled and graded (Priority 4).
- Regrade the interior slopes of the Limestone Runoff Pond to 3H:1V (Priority 4).
- Clean and remove sediment from the Limestone Runoff Pond (Photos 3 and 11; Priority 5).
- Remove debris from the Limestone Runoff Surge Pond (Photo 9; Priority 5)
- Seepage from the southeast dike of the Limestone Runoff Surge Pond has been reported for several years. A remedial design should be developed and implemented to repair this dike (Photo 6; Priority 5).
- Conduct vegetation maintenance (mowing) activities along the interior slopes of the Limestone Runoff Pond and Limestone Runoff Surge Pond.
- An H&H engineering analysis of the Limestone Drainage Basin should be conducted to provide engineering recommendations for the improvement of the hydraulics of the basin. It was Stantec's understanding that Worley Parsons was tasked to complete this study as of the 2010 annual inspection, but the status of the study is unknown at this time.

Stantec

TVA 2011 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Limestone Drainage Basin (DRAFT)
Photos



Photo 1

Northeast interior slope looking northwest.

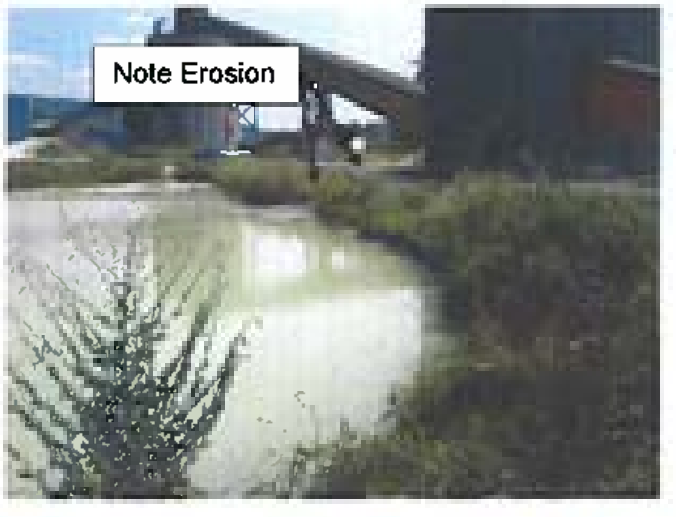


Photo 2

West interior slope looking south. Note the erosion on the west crest and interior slope.

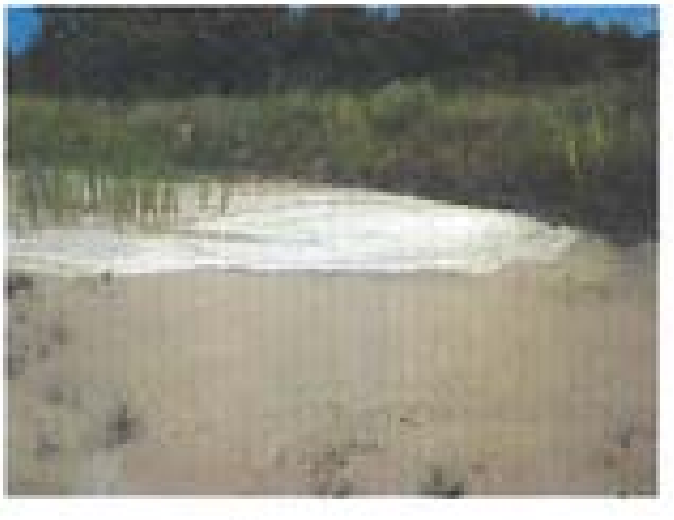


Photo 3

Southeast interior slope looking northeast. Note the sediment buildup in the southern corner of the limestone drainage basin.



Photo 4

Erosion along the west interior slope.



Photo 5

Erosion along the west interior slope (Item 45).



Photo 6

Seepage from the limestone runoff surge pond.



Photo 7

Northwest interior slope of limestone runoff surge pond, looking northeast. Note the two 30 inch diameter corrugated metal pipe outlets from the limestone drainage basin.



Photo 8

Two 30 inch diameter corrugated metal pipes from the limestone drainage basin into the limestone runoff surge pond.



Photo 9

Limestone runoff surge pond, looking north. Note the debris in the pond.



Photo 10

Trench from the limestone stockpile to the limestone drainage basin looking north. Note the sediment deposition in the trench (Item 46).

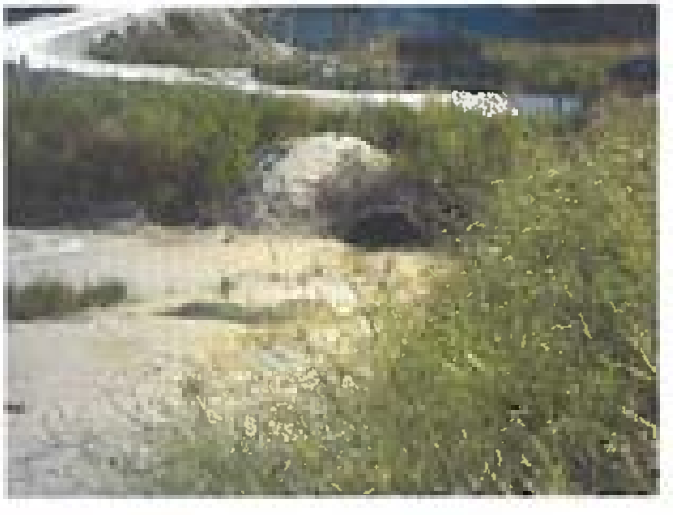


Photo 11

48 inch diameter corrugate metal pipe outlet from the slurry line ditch. Note sediment around the outlet of the pipe.



Photo 12

Interior of the 48 inch diameter corrugated metal pipe from the slurry pipe ditch to the limestone drainage basin. Note the sediment blocking the pipe under the sinkhole shown in Photo 14 (Item 47).

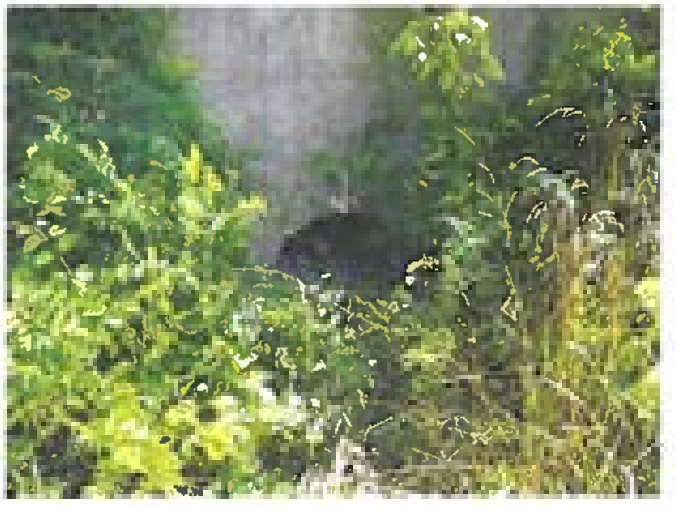


Photo 13

48 inch diameter corrugated metal pipe inlet from the slurry pipe ditch. The pipe inlet is nearly filled with sediment/soil (Item 48).



Photo 14

Sinkhole above the 48 inch diameter corrugated metal pipe.

N
Feet
0
400
Stantec
Plan View - Limestone Drainage Basin
WCF 2011 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
1 OF 1

Enclosure K

Coal Yard

TVA 2011 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Coal Yard Drainage Basin and Perimeter Ditch
(DRAFT)

1. General Facility Information

Facility Status:	Active	**NID Identification:**	N/A
Surface Area (inside dikes):	2 acres	**TVA Hazard Classification:**	N/A
Maximum Height (toe to top of dike):	N/A	**Dike Length:**	1,500 feet
Plant Discharge to Facility:	N/A	**Current Pool Elevation:**	600 feet (assumed)

2. Site Visit Information

Stantec Inspection Team:	Greg Budd and Briggs Evans
TVA Staff Present:	Jake Booth
Field Inspection Date:	June 13, 2011
Weather/Site Conditions:	Sunny, 85 degrees

3. History/Current and Future Operations

History:	The Coal Yard Drainage Basin was designed in 1977 to collect the runoff from the coal stockpile.
Current Operations:	Runoff from the coal stockpile flows into the coal yard perimeter ditch which flows into the coal yard sediment basin via three 18 inch diameter corrugated HDPE pipes. From the sediment basin, the runoff flows into the coal yard drainage basin via three 24 inch corrugated plastic pipes. The coal yard drainage basin is controlled by a pump station, which pumps the water to the main ash pond where it is discharged through Outfall 001.
Future Planned Operational Changes:	None at this time.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.



Photo 1

General view of the west dike looking north. Note the vegetation coverage on the interior slope and crest. Vegetation maintenance should be conducted.

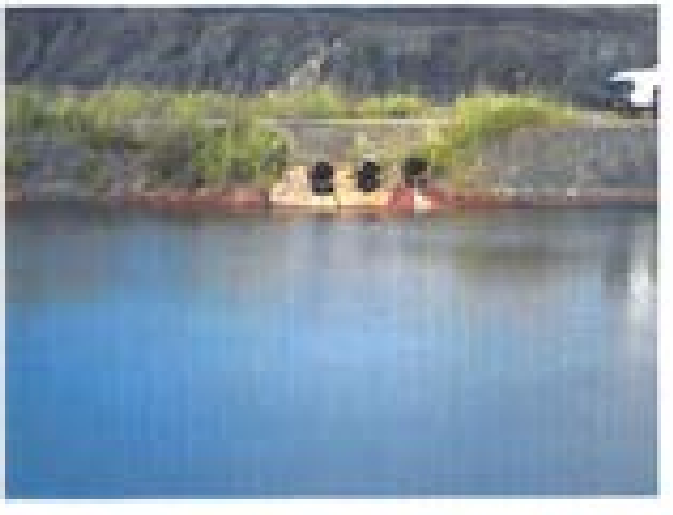


Photo 2

View of the east dike interior slope from the west dike crest. Note the three inlet pipes from the sediment basin and lack of vegetation on the interior slopes.

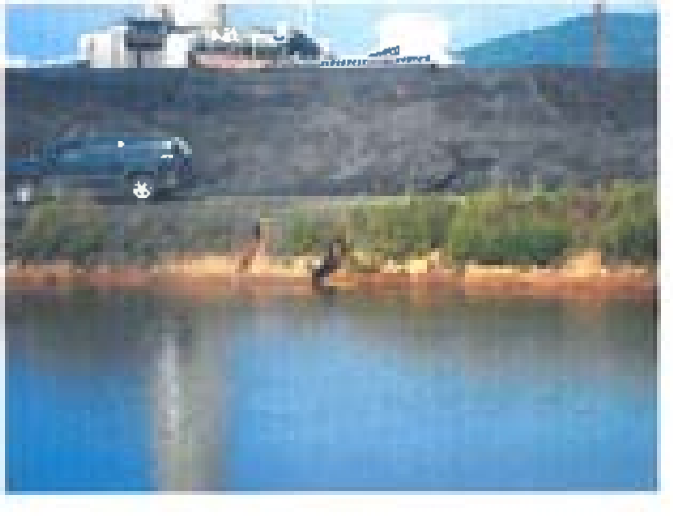


Photo 3

View of the east dike interior slope from the west dike crest. Note the lack of vegetation/erosion on the interior slopes.



Photo 4

View of the east dike crest looking south.

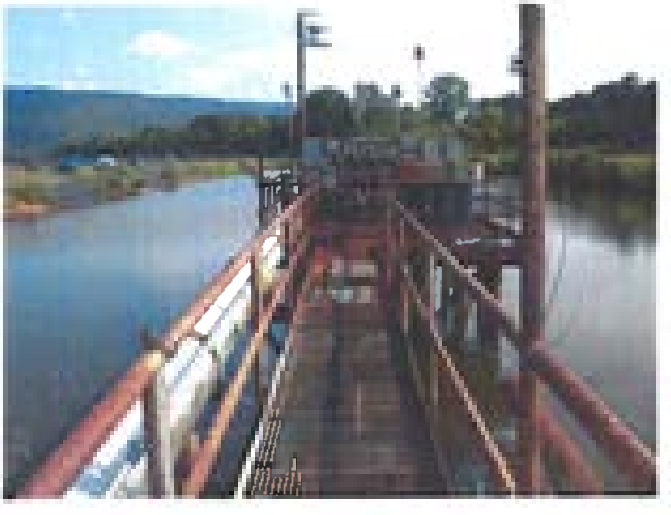


Photo 5

Pump station on north end of drainage basin looking south.



Photo 6

Corroded handrail (typical) on pump station.



Photo 7

Coal yard perimeter ditch on southwest side of coal stockpiles. Convergence of perimeter ditch and interior ditch between the two stockpiles (looking south). Coal fines and sediment delta has been cleaned out since 2010 inspection.



Photo 8

Sediment basin looking north. Note shallow root vegetation maintenance should be conducted.



Photo 9

Outlet pipes from the sediment basin to the drainage basin. Note erosion rills through coal ash cover.



Photo 10

Sediment basin inlet pipes from perimeter ditch. Note the buildup of sediment at the pipe inlets.



Photo 11

Coal yard perimeter ditch on north side of coal stockpile.



Photo 12

Coal yard perimeter ditch on west side of coal stockpile (looking south). Note ash buildup in perimeter ditch impeding runoff flow.



Photo 13

Coal yard perimeter ditch on west side of coal stockpile (looking south). Note ash sloughing into perimeter ditch impeding runoff flow.

4.1. Interior Slopes

Vegetation:	Grass-covered interior slopes with adequate coverage in most locations. The section on the east slope where the inlet pipes from the sediment basin enter the drainage basin is lacking vegetation and is covered in coal fines (Photos 1-3).
Trees:	None observed.
Wave Wash Protection:	None observed.
Erosion:	Erosion noted on the east slope where there is a lack of vegetation south of the inlet pipes entering the pond (Photo 3; Items 50-51).
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	5 feet
Encroachments:	None observed.
Slope:	1.5H:1V to 2H:1V

4.2. Crest

Crest Cover and Slope:	East crest covered in crushed stone and coal fines (Photo 4). South, North, and West crests are grass covered with adequate coverage (Photo 1). The crests slope towards the drainage basin.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	10 feet on the west dike, 15 feet on the east dike.

4.3. Exterior Slopes

Vegetation:	N/A, drainage basin is dug out with no exterior slopes.
Trees:	N/A

Erosion:	N/A
Instabilities:	N/A
Uniform Appearance:	N/A
Seepage:	N/A
Benches:	N/A
Foundation Drains, and Seepage Collection Systems:	N/A
Instrumentation:	N/A
Animal Burrows:	N/A
Slope:	N/A
Height:	N/A

4.4. Spillway Weirs/Riser Inlets

Number:	This basin does not have spillway weirs. The outlet system is via a pump station which pumps the water to the main ash pond (Photos 5-6).
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	None reported or observed.

4.5. Outlet Pipes

Number:	See 4.4 for description of pump outlet system.
Size, Type and Material:	N/A
Headwall:	N/A
Joint Separations:	N/A

Mis-Alignment: N/A

Closed/Abandoned Conduits: None reported or observed.

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The following maintenance and construction activities have been performed for the Coal Yard Drainage Basin since the last annual inspection.

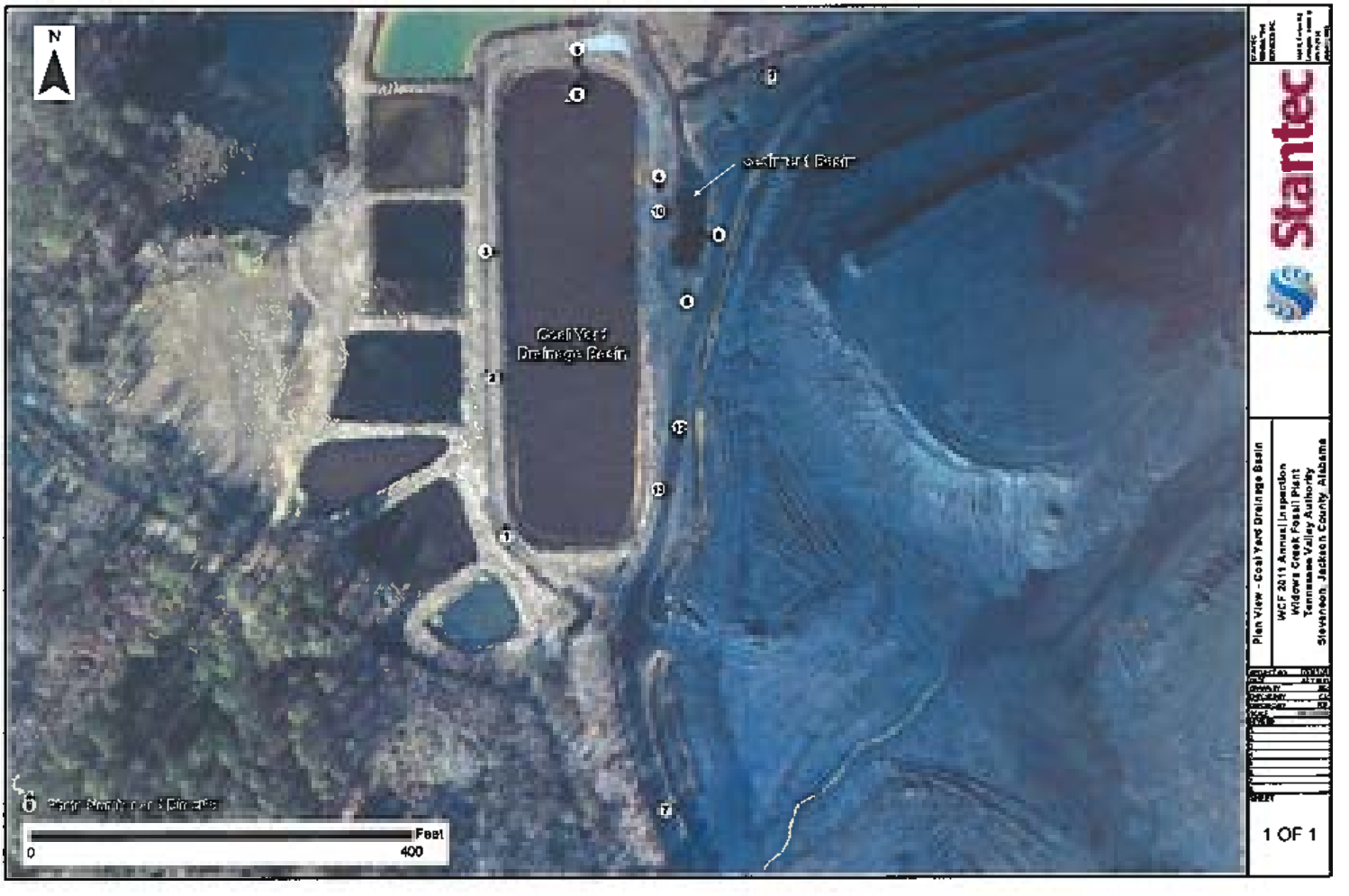
- Erosion noted on the interior slope of the east dike appears to have been repaired by filling in erosion rills with coal ash. New erosion has been observed since the repairs were made (Photo 3).
- The confluence of the interior ditch and perimeter ditch has been cleaned out and the delta that had formed has been removed allowing runoff to flow freely (Photo 7).

6. Recommendations

The following maintenance recommendations are offered for the Coal Yard Drainage Basin and Perimeter Ditch. Priority codes are included in parentheses and described in Enclosure M.

- Repair erosion rills on the interior slope of the east dike. It is recommended that this area be filled and graded with compacted clay and seeded to establish vegetation (Photo 3; Items 50-51; Priority 4).
- Mow the interior slopes and crests of the dikes to maintain access to the pump station (Photos 1 and 4; Priority 3).
- Clean sediment out of the sediment basin and perimeter ditch to promote runoff to travel from the perimeter ditch to the drainage basin via the sediment basin (Photos 8-10; Priority 4).
- Repair the corroded handrails on the pump station as noted in the 2010 annual inspection report. The handrails have deteriorated in multiple locations (Photo 6; Priority 4).
- Clean and regrade the perimeter ditches to promote runoff to be directed to the drainage basin with adequate freeboard during rainfall events. Some locations in the ditch have very little, if any, available freeboard (Photo 11; Priority 4).
- There appears to be high spots in the perimeter ditch. Also, coal fines have sloughed into the perimeter ditch and formed a delta which should be cleaned out (Photos 12-13; Items 52). A hydraulics analysis should be performed to determine ditch capacity and resize the ditch to increase slope in order to obtain adequate self-

cleansing flow velocities (Priority 5).


N
Coal Yard
Drainage Basin
Feet
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400
Plan View - Coal Yard Drainage Basin
WCF 2011 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
1 OF 1

Enclosure L

Abandoned Ash Disposal

TVA 2011 Annual Inspection Program
Widows Creek Fossil Plant (WCF)
Abandoned Ash Disposal Area (DRAFT)

1. General Facility Information

Facility Status:	Inactive	**NID Identification:**	N/A
Surface Area (inside dikes):	136 acres	**TVA Hazard Classification:**	N/A
Maximum Height (toe to top of dike):	N/A	**Dike Length:**	9,375 feet
Plant Discharge to Facility:	N/A	**Current Pool Elevation:**	N/A

2. Site Visit Information

Stantec Inspection Team:	Chris Jones, EIT and Robert Fuller, PE
TVA Staff Present:	None
Field Inspection Date:	June 14, 2011
Weather/Site Conditions:	Sunny, 85 degrees, dry conditions

3. History/Current and Future Operations

History:	Ash disposal area for Units 1-6 commissioned before 1958. It was decommissioned in June of 1969.
Current Operations:	Inactive
Future Planned Operational Changes:	None

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Dense grassy vegetation (Photos 1-2).
Trees:	Large trees present throughout the entire ash disposal area (Photos 1-2).
Wave Wash Protection:	N/A

Erosion:	None observed.
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	N/A
Encroachments:	N/A
Slope:	N/A

4.2. Crest

Crest Cover and Slope:	Dense grass vegetation.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	20 feet

4.3. Exterior Slopes

Vegetation:	Dense grass vegetation.
Trees:	Large trees present throughout entire ash disposal area.
Erosion:	None observed.
Instabilities:	None observed.
Uniform Appearance:	Yes
Seepage:	None observed.
Benches:	None
Foundation Drains, and Seepage Collection Systems:	None observed or reported.
Instrumentation:	None
Animal Burrows:	None observed.

Slope:	N/A
Height:	N/A

4.4. Spillway Weirs/Riser Inlets

Number:	None
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	N/A
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	There are two abandoned spillway weirs/risers noted on previous yearly inspection reports. These were not located during this year inspection due to large brush growth.

4.5. Outlet Pipes

Number:	None
Size, Type and Material:	N/A
Headwall:	N/A
Joint Separations:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	See section 4.4

5. Repairs/Mitigation/New Construction Activities Since Last Annual Inspection

The Abandoned Ash Disposal Area has been closed since 1969. There have been no construction activities or repairs since the previous annual inspection.

6. Recommendations

The following maintenance recommendations are offered for the Abandoned Ash Disposal Area. Priority codes are included in parentheses and described in Enclosure M.

- Continue to mow access roads as needed.



Photo 1

Large tree growth on the west side of the Abandoned Ash Disposal Area. Tree growth is typical of entire area. Note the perimeter road covered in crushed stone.



Photo 2

Dense grass vegetation on the crest of the Abandoned Ash Disposal Area. Note the crest road covered in crushed stone.

N
Abandoned Ash Disposal Area
Feet
0
400
800
Stantec
Plan View - Abandoned Ash Disposal Area
WCF 2011 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama
1 OF 1

Enclosure M

Dam Safety Priorities

Dam Safety Priorities

	Description
1	Urgent - Correct Immediately
2	Complete Within 1 Week of inspection
3	Complete Within 1 Month (30 days) of Inspection
4	Complete Within 6 Months of Original Entry Date
5	Complete Within 1 Year of Original Entry Date
6	Complete Within 3 Years of Original Entry Date
7	Complete Within 5 Years of Original Entry Date
8	Work During Scheduled Outage - Blank Until Outage is Scheduled

Enclosure N

General Repair Guidelines

General Guidelines for Repair of Animal Burrows at TVA Fossil Plants

Identification

Animal burrows are relatively common along slopes of dams and dikes. If left untreated, these burrows can result in the creation of seepage paths through the embankment. Additionally tunnels may eventually collapse resulting in surface irregularities in the embankment. General guidelines for repair of animal burrows are provided below. However, if the burrow extends more than three (3) feet below the embankment surface or extends across a dam, the repair of these features should be evaluated by a geotechnical engineer on a case-by-case basis so that appropriate recommendations can be made.

Guidelines for Burrow Repair

It is recommended that shallow animal burrows (up to 3 feet) shall be repaired with surface treatment methods as follows:

- Animals shall be captured and removed from the area. It is recommended that a local conservation representative be consulted prior to this action.
- The animal burrow shall be excavated and cleaned of excess soil along its pathway up to a depth of 3 feet. With this type of repair, an isolated excavated area of the embankment is exposed.
- The excavated area shall be backfilled with compacted cohesive material.
- If the burrow extends more than three feet into the embankment, a geotechnical engineer shall further evaluate the burrow depth and recommend a deep burrow treatment method or other exploratory methods.
- One possible method which may be recommended to treat a deep burrow can consist of a special grout (flowable fill) pumping system with a hose inserted into the burrow.

Ultimately, these repairs will not prevent rodents from creating new burrows within dam embankments. Accordingly, continual efforts must be made to discourage rodent activity. Mowing of vegetation on the slopes / crest of the embankment and trimming of water-side vegetation at regular intervals will tend to discourage rodents from re-establishing burrows along the dike and will allow timely observation of new activity if it occurs.

General Guidelines for Rill and Gully Erosion Repair at TVA Fossil Plants

Identification

Erosion features can commonly occur along dike slopes, dry stack slopes, or other sloped surfaces at TVA fossil plant facilities. Erosion normally appears in the form of rills (shallow channels) and gullies (larger and deeper eroded channels) and is formed by concentrated flow of storm water runoff, especially on bare slopes or where vegetation is sparse. If left untreated, the rills and gullies can progress in size and could lead to slope instability or other adverse issues. General guidelines for the repair of rills and gullies are provided below. The following guide is intended to be applicable to minor to moderate cases of rill/gully erosion. Where widespread or extensively deep gullies have formed or are recurring, case-specific engineering evaluations may be needed.

Guidelines for Rill and Gully Erosion Repair

Shallow Rills and Gullies:

For cases where shallow rills and gullies are present, repair should consist of the following:

- Dump and spread clay soil to fill, re-grade, and shape affected areas to conform to original ground line. Tracking and blading material with a dozer should be performed until the original ground line is reformed and material is reasonably compacted.

- Repaired areas should be seeded to re-establish vegetative cover. Erosion control blankets should be placed over re-graded areas following seeding. Materials and placement of erosion control blankets should comply with the following specifications, depending on the state in which the work is being performed.

  Kentucky Plants – KYTC Standard Specifications, Sections 212.03.03 E and 827.07

  Tennessee Plants – TDOT Standard Specifications, Section 805

  Alabama Plants – ALDOT Standard Specifications, Section 659

Deep Rills and Gullies:

For deep gullies that cannot be repaired as described above, the following filling procedures apply:

- Clean loose soil/debris from bottom and sides of gullies.

- Place and compact clay in 6 inch lifts using small compaction equipment or hand-held tampers. Vibratory plate compactors are not applicable for clay. Filling should start at the toe (or lowest elevation) and progress upslope.

- In some cases, over-excavation may be required to create benches to facilitate compaction on level surfaces. Benching, if required, will likely have to be performed by hand methods or using small excavation equipment.

- If several side-by-side deeper gullies are present in an area to be repaired, it may be more practical to rework the entire affected area to facilitate use of larger equipment. In this case, slight over-excavation of the slope face will be needed so that foundation benches can be cut to facilitate compaction on level surfaces. Filling should start at the lowest elevation and progress upslope.

- Final filling/shaping to reform the original ground line can be executed by tracking and blading with a dozer.

- Repaired areas should be seeded to re-establish vegetative cover. Erosion control blankets should be placed over re-graded areas following seeding. Materials and placement of erosion control blankets should comply with the following specifications, depending on the state in which the work is being performed.

  Kentucky Plants – KYTC Standard Specifications, Sections 212.03.03 E and 827.07

  Tennessee Plants – TDOT Standard Specifications, Section 805

  Alabama Plants – ALDOT Standard Specifications, Section 659

General Guidelines for Rutting Repair at TVA Fossil Plants

Identification

Rutting due to maintenance vehicle traffic can commonly occur along dike crests, slopes, and other areas at TVA fossil plant facilities. It is typically caused by near-surface materials which have become weak over time because of moisture infiltration. Repeated passes of equipment over weakened materials can lead to rutting. Maintenance traffic/equipment should avoid wet/rutted areas until repairs can be made. General guidelines for the repair of rutting are provided below. The following guide is intended to be applicable for minor to moderate cases of rutting, and generally consists of reworking the upper portion of the affected area, followed by re-shaping to provide positive surface drainage. Where widespread or extensively deep rutting has occurred or is recurring, case-specific engineering evaluations may be needed.

Guidelines for Rutting and Repair

- Drain any standing water and undercut affected areas to remove rutted and overly wet/soft materials. The undercut depth will be determined by TVA in the field, depending on the severity of the rutting.

- Fill undercut area with clay or bottom ash material and compact in 6 to 8 inch lifts to restore original ground line. Excavated material can be re-used if it is free of organics and can be dried to facilitate re-compaction. Otherwise, borrow material will be needed. For compaction, use hand held jumping jacks or small power equipment.

- Grade and shape repaired areas to provide positive/improved drainage. For dike crests, grade the area to drain inwardly toward the pond or perimeter ditch, as applicable. Re-grade surrounding areas and/or drainage ditches to improve drainage, if possible.

- Repaired surfaces or dike crests that are to be used as access roads should be topped with crushed stone or bottom ash. The thickness should be equal to that which was originally in place prior to the repair, or as judged by TVA to be sufficient for the expected amount of vehicle/equipment traffic.

- For other repaired areas, place seed and cover with erosion control blanket to re-establish vegetation. Materials and placement of erosion control blankets should comply with the following specifications, depending on the state in which the work is being performed.

  Kentucky Plants – KYTC Standard Specifications, Sections 212.03.03 E and 827.07

  Tennessee Plants – TDOT Standard Specifications, Section 805

  Alabama Plants – ALDOT Standard Specifications, Section 659

General Guidelines for Tree Removal on Slopes at TVA Fossil Plants

Identification

Trees and heavy brush growth should be controlled on TVA dams and dikes. If left in place, trees can result in the creation of seepage paths within the embankment. Allowing vegetation to become overgrown restricts the level of inspection that can be performed on the structure. General guidelines for removal of trees and maintenance of vegetation are provided below. Evaluations other than those outlined below shall be made by a geotechnical engineer in consultation with facility representatives on a case-by-case basis.

Guidelines for Tree Removal and Maintenance of Vegetation

Tree Removal

At locations where it is not reasonable to remove trees by a mowing them with a bush hog or with similar mowing equipment:

- All trees shall be cut using a handsaw or chainsaw and the cut tree and branches discarded.
- Remove the remaining tree trunk, stump, and rootwad.
- Grub any remaining roots of the tree so that only 2 inches or smaller roots are left in place.
- The resulting cavity from removal of the rootwad shall be cleaned of loose soil and debris.
- The cavity shall then be backfilled with cohesive soil and compacted and the area seeded to re-establish vegetation. If the tree has been removed from along the upstream or downstream face of a slope, benches shall be cut into the slope face where the cavity is to be backfilled. This will allow for a proper bond between the existing dike and the backfill being used to reform the slope. If benches are needed, bench heights shall not exceed 4 to 5 feet in height.

Maintenance of Vegetation

- Mowing is recommended at regular intervals to allow for appropriate inspection of embankment slopes.
- If areas lacking vegetation are observed during mowing and clearing operations or subsequent inspections, the areas should be seeded to re-establish vegetation as soon as practicable.

General Guidelines for Wave Wash Erosion Repair and Riprap Protection at TVA Fossil Plants

Identification

Wave erosion should be controlled on TVA facilities to maintain the integrity of dams and dikes. When present, wave wash erosion typically occurs along interior slopes of dikes near pool level. If left unrepaired, erosion can expand, deepen, and can eventually lead to interior slope sloughing. General guidelines for repair of wave erosion using riprap are provided below.

Guidelines for Wave Wash Erosion Repair and Riprap Protection

The following describes repair of wave wash erosion using riprap protection:

- Vegetation and loose soil should be removed within the affected slope areas to be repaired. This includes undercutting the slope a minimum of 12 inches to remove vegetation and associated roots. The minimum vertical extent of the vegetation removal should extend from one-foot below the pool level upwardly to two feet above pool level.

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

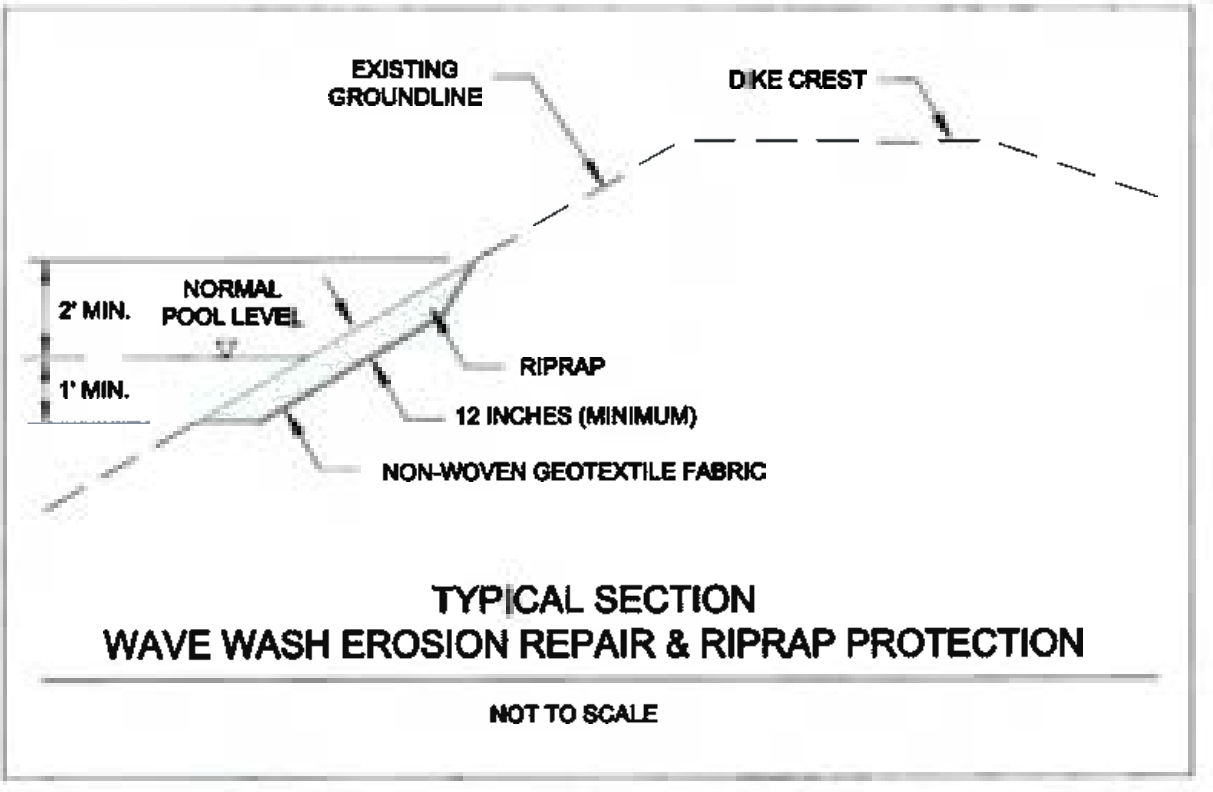
Kentucky Plants -	KYTC Type I Geotextile Fabric
Tennessee Plants -	TDOT Type III Geotextile Fabric
Alabama Plants -	Fabric conforming to Section 608 of ALDOT Standard Specifications

- Place riprap over the geotextile fabric. An excavator should be used to place the riprap in layers (starting from the bottom). Place thickness of riprap to conform to original ground line, or as necessary to create a stable slope face. Use riprap meeting the following designations, depending on the state in which the work is being performed.

Kentucky Plants -	KYTC Class II Channel Lining
Tennessee Plants -	TDOT Class A-1 Machined Riprap
Alabama Plants -	ALDOT Class 2 Riprap

- Field adjustments may be necessary as the work progresses, depending on actual conditions encountered.

A typical cross-section is presented on the following page.



TYPICAL SECTION
WAVE WASH EROSION REPAIR & RIPRAP PROTECTION

NOT TO SCALE

Enclosure O

Items

LOCATION: Widows Creek Fossil Plant - 2011 Annual Inspection
WEATHER: Sunny
INSPECTION BY: Greg Budd, Briggs Evans, Chris Jones, Robert Fuller
DATE: 6/13/2011 and 6/14/2011

ITEM NO	DESCRIPTION	PICTURE NO	POINT NO.	NORTHING[1]	EASTING	RECOMMENDATION
			3RD QUARTER GYPSUM STACK			
1	Standing water	1143	1000	1601338.28	528485.92	Repair in accordance with General Guidelines
2	Gypsum washing down slope	1144	1001	1601454.42	528451.69	Repair erosion and silt fence to prevent gypsum washout.
3	Broken pipe on Monitoring Well	1145	1002	1600643.82	527820.04	Clean out ditch and maintain as warranted
5	Erosion along side top of buttress	1148	1004	1601251.66	527922.35	Repair in accordance with General Guidelines
6	Erosion along silt fence	1149	1005	1601198.13	527078.65	Repair in accordance with General Guidelines
7	Erosion along silt fence	1150	1006	1601573.83	527081.58	Repair in accordance with General Guidelines
8	Erosion along side top of buttress	1151	1007	1601604.20	527020.93	Repair in accordance with General Guidelines
9	Possible seep at base of haul ramp	1152	1008	1601933.36	526905.65	Monitor to determine if seep. If so notify Engineering and add to Seepage Log
10	Sediment in ditch	1047	2000	1601207.44	528509.10	Clean out ditch and maintain as warranted
11	Multiple erosions (10' L X 3'W X 2'D)	1048	2008	1601852.51	526773.65	Repair in accordance with General Guidelines
13	Red water seep	2333	3001	1603453.32	529065.67	Update Seepage Action Plan. Continue to Monitor. If any changes are noticed notify CCP Engineering
14	Erosion	2334	3002	1603462.21	527681.07	Previously identified (FY11 1st quarter PTN 3014). Repair in accordance with General Guidelines.
			3RD QUARTER ASH STACK			
15	No vegetation at toe of slope	1153	1009	1599587.82	526603.95	Reseed in accordance with the TVA T-1 Section 580 Grassing Specifications
16	Possible seep (appears inactive)	1154	1010	1598615.12	527344.23	Monitor to determine if seep. If so notify Engineering and add to Seepage Log
17	Erosion to the corner of dike	1155	1011	1598483.68	527441.17	Repair in accordance with General Guidelines
18	Erosion/rill alongside ditch line	1156	1012	1597239.60	526708.41	Repair in accordance with General Guidelines
19	Erosion (typical inside slope of pond)	1049	2011	1600038.45	525571.61	Repair in accordance with General Guidelines
20	Cracking on Divider Dike (beginning)	1050	2012	1599937.05	525198.52	Monitor cracking for signs of movement. If cracking increases or slough begins, contact engineering.
21	Cracking on Divider Dike (end)	1051	2013	1599887.01	525248.04	Monitor cracking for signs of movement. If cracking increases or slough begins, contact engineering.
22	Large erosion around pipe	1052	2014	1599009.56	524538.71	Repair in accordance with General Guidelines
23	Multiple erosions/ditch settlement (beginning)	1053	2015	1598475.66	524703.80	Repair in accordance with General Guidelines
24	Multiple erosions/ditch settlement (end)	1054	2016	1598275.31	524956.57	Repair in accordance with General Guidelines
25	Wave erosion (typical on this side of slope)	1055	2018	1597987.50	524943.12	Repair in accordance with General Guidelines
26	Animal burrow (0.5' dia)/Wave erosion/Woody growth	1056	2019	1597957.36	524920.39	Repair in accordance with General Guidelines
27	Slough/crack (beginning)	2336	3005	1601378.14	525931.68	Repair in accordance with General Guidelines
28	Slough/crack (end)	2337	3006	1601382.84	525914.42	Repair in accordance with General Guidelines
29	Bucket imprint on dike	2338-2340	3007	1601587.89	525612.22	Repair in accordance with General Guidelines
30	Animal burrow	2341	3008	1601604.08	525588.79	Repair in accordance with General Guidelines
31	Bare ground from bucket imprint	2342-2343	3009	1601774.77	525330.40	Repair in accordance with General Guidelines
32	Erosion (15' area)	2344	3012	1600748.71	523534.78	Repair in accordance with General Guidelines
33	Tree blocking Pizometer	2345	3013	15986?0.89	523025.66	Repair in accordance with General Guidelines
34	Pipe to be removed	2346	3014	1597261.70	522478.61	UR5 to determine pipe removal.
35	Structure with no grate	2347	3015	1596864.94	522609.54	Repair with new grate over structure
36	Scour pool needs rock	2348	3016	1596566.09	523285.57	Add rock as needed
			ANNUAL INSPECTION			
37	Tree uprooted	2349-2350	300	1600516.90	523192.25	Remove tree and repair area around base of tree.
38	Culvert pipe inverts	2351	301	1599743.65	522996.13	Clean sediment from culvert inlets.
39	Culvert pipe inverts	2351	302	1599734.99	522995.21	Clean sediment from culvert inlets.
40	Dips and cracks in road	2352	303	1599820.65	522992.49	Monitor interior slope for signs of sloughing

1. Location not surveyed. Coordinates were determined based on approximate location on aerial photograph
2. Location surveyed in a previous quarterly inspection conducted by TVA.

ITEM NO.	DESCRIPTION	PICTURE NO.	POINT NO.	NORTHING	EASTING	RECOMMENDATION
41	Dips and cracks in road	-	304	1599775.60	522980.17	Monitor interior slope for signs of sloughing
42	Dips and cracks in road	-	305	1599848.45	522997.24	Monitor interior slope for signs of sloughing
43	Culvert invert	2353	306	1598363.91	522649.01	Clean sediment from culvert inlets.
44	Culvert invert	2354	307	1598360.94	522646.54	Clean sediment from culvert inlets.
45	Erosion	2355-2356	308	1596267.77	525313.54	Repair in accordance with General Guidelines
46	Ditch	2357	309	1596387.34	525319.20	Clean sediment from ditch
47	Sinkhole	2358	310	1596131.32	525320.73	Repair pipe which is causing the sinkhole
48	Pipe to be removed	2359	311	1596000.72	525451.34	Clean sediment from inlet of pipe
49	Erosion	2360	312	1593770.99	521975.68	Repair in accordance with General Guidelines
50	Erosion	2361	313	1594040.46	521974.59	Repair in accordance with General Guidelines
51	Slough	2362	314	1593787.02	522027.29	Repair in accordance with General Guidelines
52	Seepage Outlet 4	2363	315	1600615.22	527851.22	Information purposes only. No recommendation/repair necessary
53	Seepage Outlet 5	2364	316	1601133.74	528434.11	Information purposes only. No recommendation/repair necessary
54	Seepage Outlet 6	2365	317	1601788.92	528670.17	Information purposes only. No recommendation/repair necessary
55	Seepage Outlet 7	2366	318	1602360.32	528612.11	Information purposes only. No recommendation/repair necessary
56	Seepage Outlet 8	2367	319	1602828.97	528891.98	Information purposes only. No recommendation/repair necessary
57	Seepage Outlet 9	2368	320	1603436.27	529038.88	Information purposes only. No recommendation/repair necessary.
58	Seepage Outlet 10	2369	321	1604133.74	528891.97	Information purposes only. No recommendation/repair necessary
59	Beginning Construction - North corner of IRRP	7502	322	1604326.51	528803.00	Information purposes only. No recommendation/repair necessary.
60	Seepage Outlet 11	2370	323	1603995.49	528304.49	Information purposes only. No recommendation/repair necessary
61	Seepage Outlet 12	2371	324	1603507.28	527671.39	Information purposes only. No recommendation/repair necessary.
62	Seepage Outlet 13	2372	325	1603021.34	527040.14	Information purposes only. No recommendation/repair necessary
63	Seepage Outlet 14	2374	326	1602353.14	526727.19	Information purposes only. No recommendation/repair necessary.
64	Seepage Outlet 15	2375	327	1601935.24	526739.00	Information purposes only. No recommendation/repair necessary.
65	End construction - West side north of GSSP	167	328	1601851.96	526673.15	Information purposes only. No recommendation/repair necessary
66	Buttress beginning - Southeast slope	147	2001	1601008.86	528327.90	Information purposes only. No recommendation/repair necessary
67	Buttress end - Southeast slope	149	2002	1600720.04	528013.65	Information purposes only. No recommendation/repair necessary
68	Buttress beginning - Southwest Slope	155	2003	1600296.07	527269.41	Information purposes only. No recommendation/repair necessary
69	Buttress end - Southwest Slope	159	2004	1600625.07	527039.71	Information purposes only. No recommendation/repair necessary
70	Rutting/cracks on 626 bench	100	NA[1]	1600991.20	523658.21	Repair in accordance with General Guideline
71	Corroded pipe/undercutting at toe of slope (58-2nd Quarter 2011)	2579	NA[2]	1597863.25	522726.91	URS recommedation to remove abandoned pipe in a memo dated April 11, 2011.
72	(2) corroded pipes/erosion under road (48-2nd Quarter 2011)	1974-1977	NA[2]	1598349.58	522848.67	URS recommedation to remove abandoned pipe in a memo dated April 11, 2011.
73	Abandoned pipe (49-2nd Quarter 2011)	1978	NA[2]	1597530.05	522549.37	URS recommedation to remove abandoned pipe in a memo dated April 11, 2011.
74	Abandoned pipe located next to Red Water Seep (50-2nd Quarter 2011)	1979-1980	NA[2]	1597258.49	522478.94	URS recommedation to remove abandoned pipe in a memo dated April 11, 2011
75	Lack of vegetation (begin)	7634-7639	NA[1]	1596877.91	524129.14	Repair in accordance with General Guidelines
76	Lack of vegetation (end)	7634-7639	NA[1]	1597608.76	525085.17	Repair in accordance with General Guidelines
77	Lack of vegetation/exposed ash due to erosion	7628	NA[1]	1597380.58	524944.86	Repair erosion in accordance with General Guidelines. Reseed to establish vegetation
78	Erosion rills/sediment buildup in ditch below erosion	351	NA[1]	1596836.40	524328.65	Repair in accordance with General Guidelines. Remove sediment from ditch
79	Crushed stone placed on slope near sluice lines	133	NA[1]	1596598.61	522968.97	Information purposes only. No recommendation/repair necessary
80	Active placement of Bottom Ash	7777	NA[1]	1597675.38	523667.79	Information purposes only. No recommendation/repair necessary
81	Erosion/lack of vegetation on lower perimeter road	7573	NA[1]	1598423.48	527559.36	Repair in accordance with General Guidelines
82	Erosion rills/lack of vegetation	339	NA[1]	1597375.89	526162.19	Repair in accordance with General Guidelines
83	Erosion rills/lack of vegetation	7565	NA[1]	1598991.93	526983.78	Repair in accordance with General Guidelines
84	Erosion around manhole	7569	NA[1]	1598661.13	527225.49	Repair in accordance with General Guidelines
85	Small trees growing near discharge pipe.	7554	NA[1]	1600600.69	526293.60	Remove trees in accordance with General Guidelines.

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ITEM NO.	DESCRIPTION	PICTURE NO.	POINT NO.	NORTHING	EASTING	RECOMMENDATION
85	Pipe from 636 perimeter ditch to exterior slope.	345	NA[1]	1599238.84	525789.17	Remove pipe. Place crushed stone on roadway to fill void.
87	Recently installed water drop at Dredge Cell	341	NA[1]	1598097.50	527478.04	Information purposes only. No recommendation/repair necessary.
88	Construction material stockpile.	84-85	NA[1]	1599753.91	525462.21	Information purposes only. No recommendation/repair necessary.
89	New culvert installed at Dredge Cell.	7692, 7696	NA[1]	1598230.48	525032.79	Information purposes only. No recommendation/repair necessary.
90	Bottom Ash placement on top of dredge cell (beginning)		NA[1]	1598325.26	525318.93	Information purposes only. No recommendation/repair necessary.
91	Tree growing in spillway/skimmer	306	NA[1]	1597238.34	525816.54	Remove tree in accordance with General Guidelines.
92	Small tree along waterline	308,318	NA[1]	1597351.42	525949.48	Remove tree in accordance with General Guidelines.
93	Small tree along waterline	308,318	NA[1]	1597529.84	525732.44	Remove tree in accordance with General Guidelines.
94	Erosion rills	315	NA[1]	1597835.42	525453.48	Repair in accordance with General Guidelines.
95	Erosion on interior slope	300	NA[1]	1597554.67	525266.36	Repair in accordance with General Guidelines.
96	New siphon system	7621, 7702	NA[1]	1598093.27	524983.96	Information purposes only. No recommendation/repair necessary.
97	Repaired hand rail.	297	NA[1]	1597674.24	525411.83	Information purposes only. No recommendation/repair necessary.
98	Cluster of trees in southeast corner.	333	NA[1]	1597205.58	526038.19	Remove trees in accordance with General Guidelines.
99	Erosion rills (typical)	7596	NA[1]	1597227.93	525902.37	Repair in accordance with General Guidelines.
100	Erosion near sluice lines	7597	NA[1]	1597131.89	526014.60	Repair in accordance with General Guidelines.
101	Slough (previously identified)	332	NA[1]	1597293.03	525937.95	Engineering design required for slough repair.
102	Culvert from the wetlands to pump pond with the board over the opening	328	NA[1]	1597138.52	525846.10	Engineering analysis of wetlands required.
103	Pipe with blind flange running through southeast dike near pump station (beginning)	336	NA[1]	1597117.55	525959.22	Remove pipe. Engineering letter required for pipe removal
104	Pipe with blind flange running through southeast dike near pump station (end)	335	NA[1]	1597051.61	526066.19	Remove pipe. Engineering letter required for pipe removal.
105	Animal burrows (2)	7798 7799	NA[1]	1596978.04	523236.09	Repair in accordance with General Guidelines.
106	Drainage basin for Chemical Ponds.	150, 161	NA[1]	1596994.53	523808.38	Information purposes only. No recommendation/repair necessary.
107	Small trees along interior slope (typical)	74	NA[1]	1601115.34	523541.09	Remove trees in accordance with General Guidelines
108	Rock buttress/seepage blanket (middle - begin Red Water Pond)	104	NA[1]	1601073.38	523642.72	Information purposes only. No recommendation/repair necessary
109	Rock buttress/seepage blanket (end)	105	NA[1]	1599329.27	523100.72	Information purposes only. No recommendation/repair necessary
110	Temporary pump location	94-96	NA[1]	1601126.22	523571.72	Information purposes only. No recommendation/repair necessary
111	New access ramp	89	NA[1]	1601129.24	523706.74	Information purposes only. No recommendation/repair necessary
112	Erosion along interior of access road	4	NA[1]	1601939.85	528583.72	Repair erosion. Improve roadside ditch - line with riprap.
113	Erosion repair on southeast exterior slope below 655 bench	20	NA[1]	1600560.60	527710.44	Information purposes only. No recommendation/repair necessary.
114	Erosion/lack of vegetation on interior slope (typical)	7755	NA[1]	1601088.48	526661.56	Repair in accordance with General Guidelines.
115	Erosion (beginning) (Item 10 - 2nd Quarter 2011)	839	NA[2]	1601747.54	526928.36	Repair in accordance with General Guidelines.
116	Erosion (end) (Item 11 - 2nd Quarter 2011)	-	NA[2]	1601806.25	526930.18	Repair in accordance with General Guidelines.
117	Slough repair on south interior slope	7754	2005	1600667.57	526853.70	Information purposes only. No recommendation/repair necessary.
118	Stone placed on interior slope to prevent sloughing (600 feet)	7751	NA[1]	1600868.20	526929.94	Information purposes only. No recommendation/repair necessary.
119	Rock buttress on northeast slope (begin)	68	NA[1]	1600864.26	526285.35	Information purposes only. No recommendation/repair necessary.
120	Rock buttress on northeast slope (end)	83	NA[1]	1601854.40	525261.94	Information purposes only. No recommendation/repair necessary.
121	Seepage blanket on northwest slope (begin)	85	NA[1]	1601851.01	524637.65	Information purposes only. No recommendation/repair necessary.
122	Riprap placed on interior slope of ash pond (begin)	183	2009	1600201.56	525751.692	Information purposes only. No recommendation/repair necessary.
123	Riprap placed on interior slope of ash pond (end)	184	2010	1600146.87	525691.865	Information purposes only. No recommendation/repair necessary.
124	Rock check dam installed across ash pond rim ditch	182	NA[1]	1600348.06	523588.68	Information purposes only. No recommendation/repair necessary

1. Location not surveyed. Coordinates were determined based on approximate location on aerial photograph.
2. Location surveyed in a previous quarterly inspection conducted by TVA.


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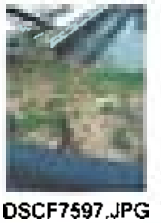

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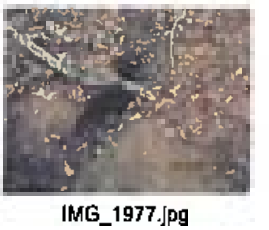

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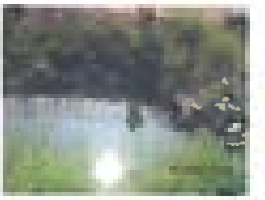

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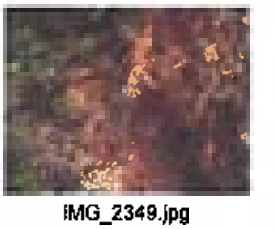

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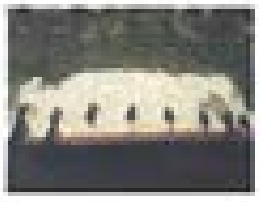

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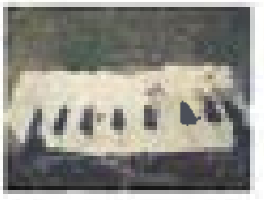

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SANY0332.JPG


SANY0333.JPG


SANY0335.JPG

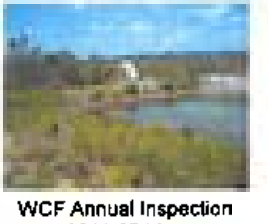

SANY0336.JPG


SANY0339.JPG


SANY0341.JPG


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SANY0351.JPG


WCF Annual Inspection 2011 074.jpg


WCF Annual Inspection 2011 084.jpg



WCF Annual Inspection
2011 085.jpg



Plan View - Item Locations

WCF 2011 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama

Stantec

1 OF 2

Stantec

Plan View - Berm Locations

WCF 2017 Annual Inspection
Widows Creek Fossil Plant
Tennessee Valley Authority
Stevenson, Jackson County, Alabama

SHEET
2 OF 2

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APPENDIX A

Document 8

TVA Widows Creek Fossil Plan NPDES Permit Number AL0003875

ONIS "TREY" GLENN, III
DIRECTOR

ADEM
Alabama Department of Environmental Management
adem.state.al.us
1400 Coliseum Blvd. 36110-2059 • Post Office Box 301463
Montgomery, Alabama 36130-1463
(334) 271-7700
FAX (334) 271-7950

BOB RILEY
GOVERNOR

JANUARY 7, 2008

MR GERALD HEMMER
PLANT MANAGER
TVA WIDOWS CREEK FOSSIL PLANT
P O BOX 2000
STEVENSON AL 35772

RE: Final NPDES Permit Modification
Permit Number: **AL0003875**

Dear Mr. Hemmer:

Attached is the issued copy of the above referenced permit.

We will look forward to receiving monitoring data in accordance with the conditions of your permit. Please see PART I.C., Page 3 for your reporting requirements. In order to minimize the paperwork burden on both of us, we ask that when submitting the required Discharge Monitoring Reports (DMR's), please **do not submit** lab worksheets, logs, reports or other paperwork, not specifically required by the permit unless requested to do so by ADEM staff.

If there are questions or comments in reference to the permit or related monitoring requirements, please contact Scott Ramsey of this office (334) 274-4220.

Sincerely,

Eric Sanderson, Chief
Industrial Section
Water Division

Enclosure: Final Permit

pc. EPA Region IV: Final Permit
Mike McCary, P & S: Final Permit
Montgomery Field Office: Final Permit

Birmingham Branch
110 Vulcan Road
Birmingham, AL 35209-4702
(205) 942-6168
(205) 941-1603 (Fax)

Decatur Branch
2715 Sandlin Road, S.W.
Decatur, AL 35603-1333
(256) 353-1713
(256) 340-9359 (Fax)

Mobile Branch
2204 Perimeter Road
Mobile, AL 36615-1131
(251) 450-3400
(251) 479-2593 (Fax)

Mobile - Coastal
4171 Commanders Drive
Mobile, AL 36615-1421
(251) 432-6533
(251) 432-6598 (Fax)

ADEM
Alabama Department of Environmental Management

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM PERMIT

PERMITTEE:	TENNESSEE VALLEY AUTHORITY WIDOWS CREEK FOSSIL PLANT
FACILITY LOCATION:	STEAM PLANT ROAD STEVENSON, AL
PERMIT NUMBER:	AL 0003875
RECEIVING WATERS:	DSN001 – 006, 008 – 012, 015 – 019: TENNESSEE RIVER DSN013: WIDOWS CREEK

In accordance with and subject to the provisions of the Federal Water Pollution Control Act, as amended, 33 U.S.C. §§1251-1378 (the "FWPCA"), the Alabama Water Pollution Control Act, as amended, Code of Alabama 1975, §§ 22-22-1 to 22-22-14 (the "AWPCA"), the Alabama Environmental Management Act, as amended, Code of Alabama 1975, §§22-22A-1 to 22-22A-15, and rules and regulations adopted thereunder, and subject further to the terms and conditions set forth in this permit, the Permittee is hereby authorized to discharge into the above-named receiving waters.

ISSUANCE DATE:	MARCH 8, 2005
EFFECTIVE DATE:	APRIL 1, 2005
EXPIRATION DATE:	MARCH 31, 2010
MODIFICATION ISSUANCE DATE:	**JANUARY 7, 2008**
MODIFICATION EFFECTIVE DATE:	**JANUARY 7, 2008**

James E. McAdams
Alabama Department of Environmental Management

US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 9

Stantec Consulting Services Memorandum September 28, 2010, Reference: "Hydrologic and Hydraulic Calculations Summary, Spillway Improvement Project, Widows Creek Fossil Plant (WCF), Main Ash Pond"

Memo

Stantec

To:	Scott Turnbow Chattanooga, TN	From:	Matthew Hoy, PE St. Louis, MO
File:	175609015	Date:	September 28, 2010

Reference: Hydrologic and Hydraulic Calculations Summary
Spillway Improvement Project
Widows Creek Fossil Plant (WCF)
Main Ash Pond

The purpose of this memorandum is to summarize the hydrologic and hydraulic calculations at the WCF Main Ash Pond. Detailed design calculations and descriptions will be provided with the final spillway design report and calculation package to be submitted by URS.

BACKGROUND

This pond complex consists of the Main Ash Pond. These ponds act as settling basins for the ash slurry that is discharged from the plant as well as stormwater detention for runoff from the old dredge cell. Five existing spillway risers are located in the Ash Pond and direct flow to the upper and lower Stilling Ponds which eventually flow to the pump pond and out to the Tennessee River.

WATERSHED & PROCESS FLOW

The area draining to the Ash Pond Complex includes direct runoff from 323 acres including 212 acres in the Ash Pond Complex (148 acres water surface, 64 acres upland area) and the former dredge cell area (111 acres). The daily plant process flow averages roughly 50 million gallons per day (MGD) as documented through correspondence with plant personnel.

OUTLET DESCRIPTION

Flow discharges from the ash pond through five (5) 48-inch diameter concrete riser structures connected to 36-inch diameter concrete outlet pipes through the dike. These structures are roughly 35 feet tall and are laterally unsupported. There are currently no other defined emergency spillways or overflow paths.

FREEBOARD

TVA's Master Programmatic Document requires 5 ft of operating freeboard for ash pond facilities. The perimeter dike crest elevation is 635.8 ft, and the Ash Pond water surface elevation is currently maintained at roughly 632.4 ft, resulting in an operating freeboard of 3.4 ft. This facility currently does not meet freeboard requirements.

US EPA ARCHIVE DOCUMENT

Reference: Hydrologic and Hydraulic Calculations Summary
Spillway Improvement Project
Widows Creek Fossil Plant (WCF)
Main Ash Pond

METHODOLOGY SUMMARY

The 6-hour PMP rainfall depth of 36.3 inches for Jackson County, Alabama was determined and the SCS 6-hour rainfall distribution was applied to this depth. This depth was estimated from a map on page 46 of *Hydrometeorological Report No. 56; Probable Maximum and TVA Precipitation Estimates With Areal Distribution for Tennessee River Drainage Less Than 3,000 Mi^2 in Area* by US Department of Commerce. The 24-hour Type II SCS distribution was applied to 100-yr, 24-hr rainfall depth of 7.5 inches obtained from *Technical Paper No. 40; Rainfall Frequency Atlas of the United States* by US Department of Commerce. The SCS curve number method was used to convert this rainfall into runoff. Composite curve numbers of 94 and 97 were assigned to the former dredge cell (composite including rim ditch area) and main ash pond, respectively. Lag time assumptions were based on topographic mapping and delineated longest flow paths and ranged from 27 minutes for the former dredge cell area to 45 minutes for the main ash pond complex. Stage-storage relationships for the main ash pond complex were developed using contour data and hydrographic survey data provided by TVA. This data was input into a Hydrologic Engineering Center's - Hydrologic Modeling System (HEC-HMS) version 3.4 model of the watershed to develop an inflow hydrograph to the ash stilling pond. Standard hydraulic equations were used to develop a rating curve for the existing spillways and level-pool routing methodology was used to route the design storm through the outlets

DESIGN STORM PERFORMANCE

TVA's Master Programmatic Document requires ash pond facilities to convey the 6-hour PMP event without overtopping. Results of the H & H analysis of the ash pond complex indicate that the existing spillways are unable to convey the 6-hr PMP event without overtopping the perimeter dike. Results of storm routings are summarized in Table 1. Supporting data are provided as Attachment A and Attachment B to this memorandum.

Table 1 – Freeboard and Routing Summary

	100-yr, 24-hr Storm	6-hr PMP
Drainage Area (ac)	323	323
Crest of Dam (ft)	635.8	635.8
Normal Pool Elevation (ft)	632.4	632.4
Normal Operating Freeboard (ft)	3.4	3.4
Normal Operation Flow (MGD)	50	50
Precipitation Depth (in)	7.5	36.3
Max. water surface elevation (ft)	633.1	Overtopping
Peak Inflow (cfs)	760	4740
Peak Outflow (cfs)	226	639

US EPA ARCHIVE DOCUMENT

Reference: Hydrologic and Hydraulic Calculations Summary
Spillway Improvement Project
Widows Creek Fossil Plant (WCF)
Main Ash Pond

FUTURE MODIFICATIONS

Because of concerns with stability of the tall, unsupported riser structures, a spillway improvement project is currently under design by URS to replace the current structures. Detailed and potentially refined calculations will be included in the supporting design documentation and will incorporate design requirements from TVA's Master Programmatic Document.

STANTEC CONSULTING SERVICES INC.



Matthew Hoy, PE
Senior Project Engineer
Matthew.Hoy@stantec.com

Attachment A: Watershed Map

Attachment B: Hydrologic and Hydraulic Model Input / Output

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Attachment A - WCF Main Ash Pond
Watersheds and Longest Flow Path

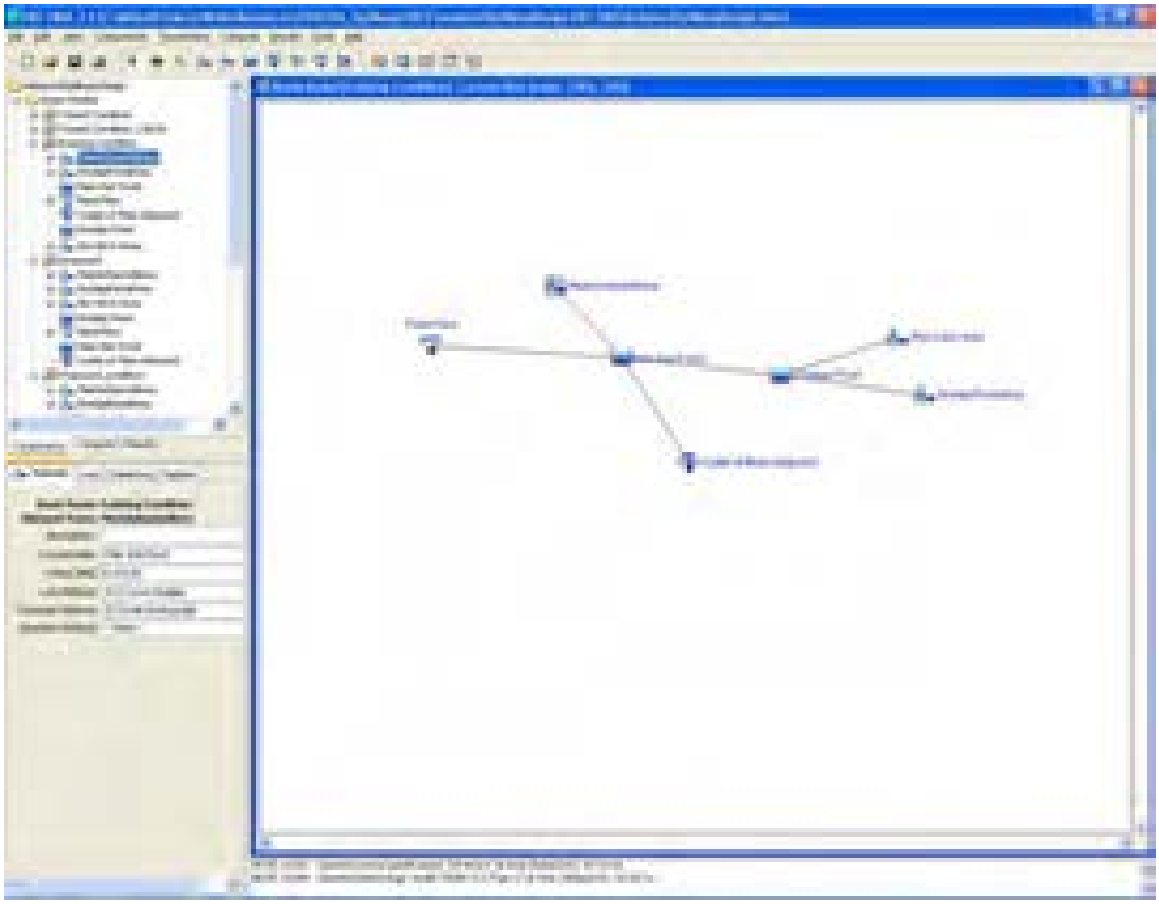


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ATTACHMENT B

HEC-HMS Input Files



Basin Models

Basin: Existing Condition
    Description: Existing Condition
    Last Modified Date: 19 November 2009
    Last Modified Time: 21:40:53
    Version: 3.4
    Unit System: English
    Missing Flow To Zero: No
    Enable Flow Ratio: No
    Allow Blending: No
    Compute Local Flow At Junctions: No

    Enable Sediment Routing: No

    Enable Quality Routing: No
End:

Subbasin: MainAshpondArea
Canvas X: -5474.857667287581
Canvas Y: 3205.859789316968
Area: 0.33125
Downstream: Main Ash Pond

Canopy: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 96.8

Transform: SCS
Lag: 45
Unitgraph Type: STANDARD

Baseflow: None
End:

Subbasin: DredgePondArea
Canvas X: 1477.5181831434966
Canvas Y: 1147.7238924466665
Area: 0.15625
Downstream: Dredge Pond

Canopy: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 97.9

Transform: SCS
Lag: 6
Unitgraph Type: STANDARD

Baseflow: None
End:

Reservoir: Main Ash Pond
Canvas X: -4227.95236191036
Canvas Y: 1865.5088965018294
Label X: -5.0
Label Y: -2.0
Downstream: Outlet of Main Ashpond

Route: Modified Puls
Routing Curve: Elevation-Area-Outflow

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Initial Elevation: 632
Elevation-Area Table: EX MainAshPond Z-A
Elevation-Outflow Table: Ex MainAshPond Z-Q
Primary Table: Elevation-Outflow
End:

Source: Plant Flow
Canvas X: -7816.877382186172
Canvas Y: 2086.3658208264947
Label X: -42.0
Label Y: 23.0
Downstream: Main Ash Pond

Flow Method: CONSTANT_FLOW
Constant Flow: 77.36
End Flow Method:
End:

Junction: Outlet of Main Ashpond
Canvas X: -2939.6203033498123
Canvas Y: -140.60816611388145
End:

Reservoir: Dredge Pond
Canvas X: -1208.9281226972507
Canvas Y: 1519.0782323798448
Downstream: Main Ash Pond

Route: Modified Puls
Routing Curve: Elevation-Area-Outflow
Initial Elevation: 641
Elevation-Area Table: EX DredgePond Z-A
Elevation-Outflow Table: Ex DredgePond_Z-Q
Primary Table: Elevation-Outflow
End:

Subbasin: Rim Ditch Area
Canvas X: 967.8483577788847
Canvas Y: 2241.192187776144
Area: 0.0171875
Downstream: Dredge Pond

Canopy: None

Surface: None

LossRate: SCS
Percent Impervious Area: 0.0
Curve Number: 84

Transform: SCS

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Lag: 27
Unitgraph Type: STANDARD

Baseflow: None
End:

Basin Schematic Properties:
Last View N: 3889.877175358116
Last View S: -4318.331457135656
Last View W: -11056.112272281263
Last View E: 5397.728589906307
Maximum View N: 3889.877175358116
Maximum View S: -4318.331457135656
Maximum View W: -11056.112272281263
Maximum View E: 5397.728589906307
Extent Method: Elements
Buffer: 20
Draw Icons: Yes
Draw Icon Labels: Yes
Draw Map Objects: No
Draw Gridlines: No
Draw Flow Direction: No
Fix Element Locations: No
Fix Hydrologic Order: No
End:

Meteorologic Models

Meteorology: 100yr SCS Type 2
Last Modified Date: 19 November 2009
Last Modified Time: 00:01:27
Version: 3.3
Unit System: English
Precipitation Method: SCS Storm
Snowmelt Method: None
Use Basin Model: Existing Cond-Watersheds
End:

Precip Method Parameters: SCS Storm
Storm Depth: 7.5
Storm Type: Type II
End:

Subbasin: MainAshpondArea
End:

Subbasin: PumpPondArea
End:

Subbasin: DredgePondPerimeterSBr

End:

Subbasin: DredgePondArea
End:

Subbasin: DredgePondPerimeterNBr
End:

Subbasin: StillingPondArea
End:

Meteorology: 6hr PMP
    Last Modified Date: 11 December 2009
    Last Modified Time: 21:39:48
    Version: 3.3
    Unit System: English
    Precipitation Method: Specified Average
    Snowmelt Method: None
    Use Basin Model: Closed Condition
    Use Basin Model: Closed Condition_CN100
    Use Basin Model: Existing Condition
    Use Basin Model: Proposed_condition
End:

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

Subbasin: MainAshpondArea
    Gage: SCS 6-Hr, PMP
End:

Subbasin: DredgePondArea
    Gage: SCS 6-Hr, PMP
End:

Subbasin: Rim Ditch Area
    Gage: SCS 6-Hr, PMP
End:Subbasin: Subbasin-6
    Gage: PMP
End:

Subbasin: Subbasin-7
    Gage: PMP
End:

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Control Specifications

Control: 24-Hours
Last Modified Date: 3 February 2010
Last Modified Time: 18:37:13
Start Date: 31 December 2008
Start Time: 24:00
End Date: 2 January 2009
End Time: 00:01
Time Interval: 1
End:

Paired Data

Stage-storage – Main Ash Pond
Main Ashpond Data

Stage - Storage

Elevation	Area (acres)	Incremental Storage (ac-ft)	Cumulative Storage (ac-ft)
626	47.22	0	0.00
627	50.43	48.82	48.82
628	53.86	52.14	100.95
629	55.26	54.56	155.51
630	61.92	58.56	214.07
631	69.48	65.66	279.73
632	71.37	70.42	350.15
633	105.70	87.97	438.13
634	126.38	115.88	554.01
635	147.19	136.65	690.66
635.8	147.96	118.06	808.72
636	148.15	29.61	838.33
637	149.12	148.64	986.97
638	150.08	149.60	1136.57

Interpolated or Extrapolated Value

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Stage - Discharge

Elevation	Discharge to Stilling Pond							Composite Discharge		
	Risers			Steel Barrels		60' Embankment Overflow				
	Head above Riser Rim	Weir $Q = 3.27LH^{1.5}$	Orifice $Q=0.65A(2gH)^{0.5}$	Head above Barrel Invert	Q (Hydrocalc)	Head above Top of Embankment	Weir $Q=3.087LH^{1.5}$	Spillway (cfs)	Embankment (cfs)	Total Discharge (cfs)
632	0.0	0	0	37.0	714			0		0
633	1.0	205	322	38.0	731			205		205
634	2.0	581	456	39.0	748			456		456
635	3.0	1067	559	40.0	764			559		559
635.8	3.8	1521	629	40.8	777	0.0	0	629		629
636	4.0	1643	645	41.0	780	0.2	17	645		645
637	5.0	2296	721	42.0	796	1.2	243	721	243	965
638	6.0	3018	790	43.0	811	2.2	604	790	604	1394

Time-Series Data

Precipitation Gages: 6-Hour PMP

Time (ddmmmyyyy, mm:hh)	Depth (in)
01Jan2000, 00:00	0.0
01Jan2000, 00:05	0.11727
01Jan2000, 00:10	0.25114
01Jan2000, 00:15	0.38643
01Jan2000, 00:20	0.52488
01Jan2000, 00:25	0.66841
01Jan2000, 00:30	0.81378
01Jan2000, 00:35	0.96291
01Jan2000, 00:40	1.1179
01Jan2000, 00:45	1.2753
01Jan2000, 00:50	1.4372
01Jan2000, 00:55	1.6061
01Jan2000, 01:00	1.7781
01Jan2000, 01:05	1.9557
01Jan2000, 01:10	2.1417
01Jan2000, 01:15	2.3320
01Jan2000, 01:20	2.5294
01Jan2000, 01:25	2.7371
01Jan2000, 01:30	2.9510
01Jan2000, 01:35	3.1742
01Jan2000, 01:40	3.4108
01Jan2000, 01:45	3.6562
01Jan2000, 01:50	3.9148
01Jan2000, 01:55	4.1916
01Jan2000, 02:00	4.4823
01Jan2000, 02:05	4.7927
01Jan2000, 02:10	5.1304
01Jan2000, 02:15	5.4918
01Jan2000, 02:20	5.8869
01Jan2000, 02:25	6.3295
01Jan2000, 02:30	6.8209
01Jan2000, 02:35	7.3847
01Jan2000, 02:40	8.0599
01Jan2000, 02:45	8.8871
01Jan2000, 02:50	9.9984
01Jan2000, 02:55	11.811
01Jan2000, 03:00	18.138
01Jan2000, 03:05	24.465

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01Jan2000, 03:10	26.277
01Jan2000, 03:15	27.389
01Jan2000, 03:20	28.216
01Jan2000, 03:25	28.891
01Jan2000, 03:30	29.455
01Jan2000, 03:35	29.946
01Jan2000, 03:40	30.389
01Jan2000, 03:45	30.784
01Jan2000, 03:50	31.145
01Jan2000, 03:55	31.483
01Jan2000, 04:00	31.793
01Jan2000, 04:05	31.793
01Jan2000, 04:10	32.361
01Jan2000, 04:15	32.620
01Jan2000, 04:20	32.620
01Jan2000, 04:25	33.102
01Jan2000, 04:30	33.325
01Jan2000, 04:35	33.325
01Jan2000, 04:40	33.746
01Jan2000, 04:45	33.944
01Jan2000, 04:50	33.944
01Jan2000, 04:55	34.320
01Jan2000, 05:00	34.498
01Jan2000, 05:05	34.498
01Jan2000, 05:10	34.838
01Jan2000, 05:15	35.000
01Jan2000, 05:20	35.000
01Jan2000, 05:25	35.313
01Jan2000, 05:30	35.462
01Jan2000, 05:35	35.462
01Jan2000, 05:40	35.751
01Jan2000, 05:45	35.889
01Jan2000, 05:50	35.889
01Jan2000, 05:55	36.158
01Jan2000, 06:00	36.288

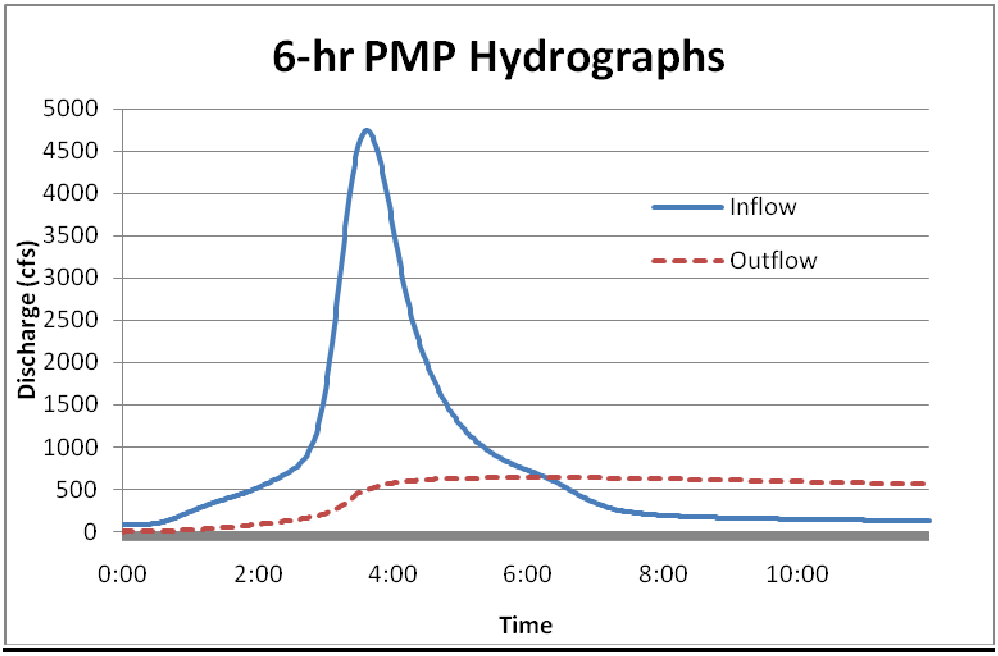
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HEC-HMS Output Files

Main Ash Pond Routing

6-hr PMP



Date	Time	Inflow (cfs)	Storage (ac-ft)	Elevation (ft)	Outflow (cfs)
1/1/2009	12:00:00 AM	77.4	0	632	0
1/1/2009	12:10:00 AM	77.8	1	632	2.4
1/1/2009	12:20:00 AM	82.9	2.1	632	4.9
1/1/2009	12:30:00 AM	99.3	3.3	632	7.6
1/1/2009	12:40:00 AM	132.9	4.7	632.1	11
1/1/2009	12:50:00 AM	181.3	6.7	632.1	15.5
1/1/2009	1:00:00 AM	236.5	9.3	632.1	21.7
1/1/2009	1:10:00 AM	291.1	12.6	632.1	29.3
1/1/2009	1:20:00 AM	339.7	16.5	632.2	38.4
1/1/2009	1:30:00 AM	382.3	20.8	632.2	48.6
1/1/2009	1:40:00 AM	423.2	25.6	632.3	59.8
1/1/2009	1:50:00 AM	467.4	30.9	632.4	71.9
1/1/2009	2:00:00 AM	518.7	36.6	632.4	85.2
1/1/2009	2:10:00 AM	579	42.8	632.5	99.8

1/1/2009	2:20:00 AM	647.5	49.8	632.6	116
1/1/2009	2:30:00 AM	724.2	57.5	632.7	134
1/1/2009	2:40:00 AM	838.8	66.2	632.8	154.3
1/1/2009	2:50:00 AM	1081	76.9	632.9	179.2
1/1/2009	3:00:00 AM	1671.2	92.6	633	215.1
1/1/2009	3:10:00 AM	2658.5	118.8	633.3	271.8
1/1/2009	3:20:00 AM	3810.3	159.1	633.6	359.1
1/1/2009	3:30:00 AM	4590.9	212	634.1	462.1
1/1/2009	3:40:00 AM	4709.9	270.1	634.5	505.9
1/1/2009	3:50:00 AM	4303.4	325.4	634.9	547.6
1/1/2009	4:00:00 AM	3626.5	372.4	635.2	574.1
1/1/2009	4:10:00 AM	2915.7	409.3	635.5	591.6
1/1/2009	4:20:00 AM	2384.5	437.4	635.7	604.9
1/1/2009	4:30:00 AM	1994.8	459	635.8	615.2
1/1/2009	4:40:00 AM	1688.9	475.7	635.9	623.1
1/1/2009	4:50:00 AM	1450.2	488.7	636	629.1
1/1/2009	5:00:00 AM	1264.1	498.6	636.1	631.8
1/1/2009	5:10:00 AM	1118	506.3	636.1	633.9
1/1/2009	5:20:00 AM	1004	512.1	636.2	635.4
1/1/2009	5:30:00 AM	912.6	516.5	636.2	636.6
1/1/2009	5:40:00 AM	838.1	519.8	636.2	637.5
1/1/2009	5:50:00 AM	776.5	522.1	636.2	638.1
1/1/2009	6:00:00 AM	725.2	523.6	636.2	638.6
1/1/2009	6:10:00 AM	675	524.5	636.2	638.8
1/1/2009	6:20:00 AM	616.4	524.6	636.2	638.8
1/1/2009	6:30:00 AM	547.9	523.8	636.2	638.6
1/1/2009	6:40:00 AM	472.4	522.1	636.2	638.1
1/1/2009	6:50:00 AM	401.4	519.3	636.2	637.4
1/1/2009	7:00:00 AM	339.6	515.6	636.2	636.4
1/1/2009	7:10:00 AM	289.8	511.2	636.2	635.2
1/1/2009	7:20:00 AM	253.7	506.2	636.1	633.8
1/1/2009	7:30:00 AM	231.2	500.8	636.1	632.4
1/1/2009	7:40:00 AM	214.4	495.1	636	630.9
1/1/2009	7:50:00 AM	201.8	489.3	636	629.3
1/1/2009	8:00:00 AM	192.2	483.4	636	626.7
1/1/2009	8:10:00 AM	184.6	477.4	635.9	623.9
1/1/2009	8:20:00 AM	178.6	471.3	635.9	621
1/1/2009	8:30:00 AM	173.6	465.2	635.8	618.1
1/1/2009	8:40:00 AM	169.4	459.1	635.8	615.2
1/1/2009	8:50:00 AM	165.7	452.9	635.8	612.3
1/1/2009	9:00:00 AM	162.5	446.8	635.7	609.4

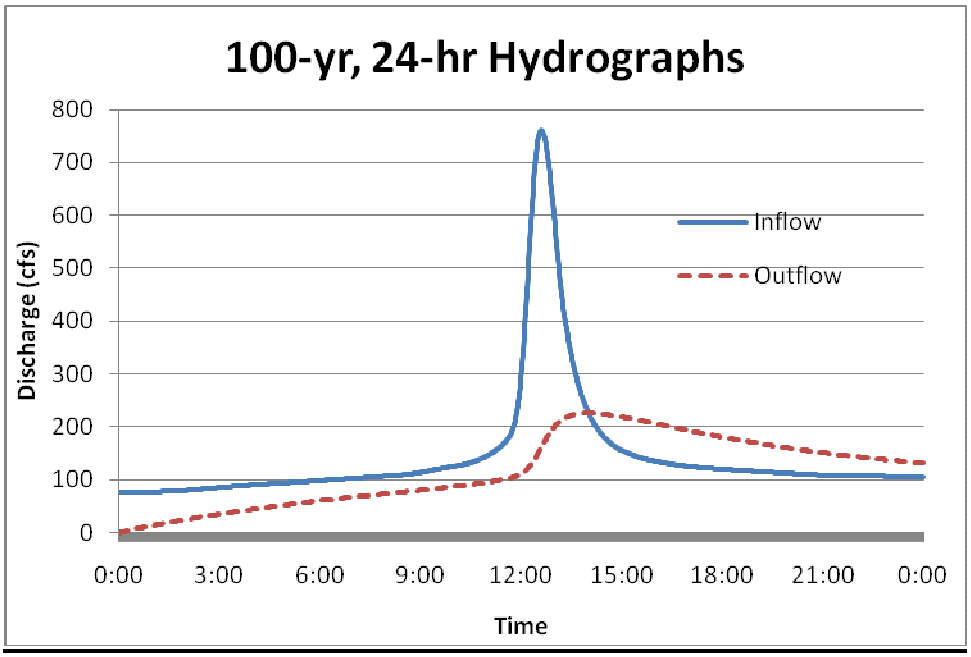
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1/1/2009	9:10:00 AM	159.5	440.6	635.7	606.4
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1/1/2009	9:30:00 AM	154.2	428.3	635.6	600.6
1/1/2009	9:40:00 AM	151.8	422.1	635.6	597.7
1/1/2009	9:50:00 AM	149.6	416	635.5	594.8
1/1/2009	10:00:00 AM	147.5	409.9	635.5	591.9
1/1/2009	10:10:00 AM	145.5	403.8	635.4	589
1/1/2009	10:20:00 AM	143.5	397.7	635.4	586.1
1/1/2009	10:30:00 AM	141.5	391.6	635.3	583.2
1/1/2009	10:40:00 AM	139.6	385.5	635.3	580.3
1/1/2009	10:50:00 AM	137.8	379.4	635.3	577.4
1/1/2009	11:00:00 AM	136	373.4	635.2	574.6
1/1/2009	11:10:00 AM	134.3	367.4	635.2	571.7
1/1/2009	11:20:00 AM	132.7	361.3	635.1	568.9
1/1/2009	11:30:00 AM	131	355.3	635.1	566
1/1/2009	11:40:00 AM	129.5	349.4	635.1	563.2
1/1/2009	11:50:00 AM	127.9	343.4	635	560.4
1/1/2009	12:00:00 PM	127.3	337.4	635	556.7
1/1/2009	12:10:00 PM	127.2	331.6	634.9	552.3
1/1/2009	12:20:00 PM	127.2	325.7	634.9	547.9

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100-yr, 24-hr

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Date	Time	Inflow (cfs)	Storage (ac-ft)	Elevation (ft)	Outflow (cfs)
1/1/2009	12:00:00 AM	77.4	0	632	0
1/1/2009	12:20:00 AM	77.4	2.1	632	4.8
1/1/2009	12:40:00 AM	77.4	4	632	9.3
1/1/2009	1:00:00 AM	77.4	5.8	632.1	13.6
1/1/2009	1:20:00 AM	77.5	7.5	632.1	17.5
1/1/2009	1:40:00 AM	78.3	9.1	632.1	21.3
1/1/2009	2:00:00 AM	79.8	10.7	632.1	24.9
1/1/2009	2:20:00 AM	81.6	12.2	632.1	28.3
1/1/2009	2:40:00 AM	83.5	13.6	632.2	31.7
1/1/2009	3:00:00 AM	85.4	15	632.2	35
1/1/2009	3:20:00 AM	87.1	16.4	632.2	38.2
1/1/2009	3:40:00 AM	88.6	17.7	632.2	41.3
1/1/2009	4:00:00 AM	90.1	19	632.2	44.3
1/1/2009	4:20:00 AM	91.5	20.2	632.2	47.1
1/1/2009	4:40:00 AM	92.8	21.4	632.2	49.9
1/1/2009	5:00:00 AM	94.2	22.6	632.3	52.7
1/1/2009	5:20:00 AM	95.6	23.7	632.3	55.3
1/1/2009	5:40:00 AM	97	24.8	632.3	57.8
1/1/2009	6:00:00 AM	98.5	25.9	632.3	60.3
1/1/2009	6:20:00 AM	100	26.9	632.3	62.7
1/1/2009	6:40:00 AM	101.4	27.9	632.3	65.1
1/1/2009	7:00:00 AM	102.9	28.9	632.3	67.4

1/1/2009	7:20:00 AM	104.2	29.9	632.3	69.7
1/1/2009	7:40:00 AM	105.7	30.8	632.4	71.8
1/1/2009	8:00:00 AM	107.1	31.8	632.4	74
1/1/2009	8:20:00 AM	108.4	32.7	632.4	76.1
1/1/2009	8:40:00 AM	110.4	33.5	632.4	78.2
1/1/2009	9:00:00 AM	113.6	34.4	632.4	80.3
1/1/2009	9:20:00 AM	117.8	35.4	632.4	82.5
1/1/2009	9:40:00 AM	121.9	36.4	632.4	84.8
1/1/2009	10:00:00 AM	125.3	37.4	632.4	87.2
1/1/2009	10:20:00 AM	129.5	38.5	632.4	89.7
1/1/2009	10:40:00 AM	136.2	39.6	632.5	92.4
1/1/2009	11:00:00 AM	146	40.9	632.5	95.4
1/1/2009	11:20:00 AM	160.1	42.5	632.5	98.9
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1/1/2009	12:20:00 PM	625	56.7	632.6	132.2
1/1/2009	12:40:00 PM	755	72.5	632.8	169
1/1/2009	1:00:00 PM	586.1	86.3	633	201
1/1/2009	1:20:00 PM	394.1	93.7	633	217.4
1/1/2009	1:40:00 PM	289.1	96.9	633.1	224.3
1/1/2009	2:00:00 PM	227.9	97.7	633.1	226.1
1/1/2009	2:20:00 PM	191.5	97.2	633.1	225.1
1/1/2009	2:40:00 PM	169.1	96	633.1	222.4
1/1/2009	3:00:00 PM	155.6	94.4	633.1	218.9
1/1/2009	3:20:00 PM	146.5	92.6	633	215
1/1/2009	3:40:00 PM	139.6	90.6	633	210.8
1/1/2009	4:00:00 PM	135	88.7	633	206.5
1/1/2009	4:20:00 PM	131.1	86.7	633	202.1
1/1/2009	4:40:00 PM	127.7	84.8	633	197.5
1/1/2009	5:00:00 PM	125	82.9	632.9	193.1
1/1/2009	5:20:00 PM	123	81	632.9	188.8
1/1/2009	5:40:00 PM	121.4	79.2	632.9	184.7
1/1/2009	6:00:00 PM	119.9	77.5	632.9	180.7
1/1/2009	6:20:00 PM	118.5	75.9	632.9	176.9
1/1/2009	6:40:00 PM	117.2	74.3	632.8	173.2
1/1/2009	7:00:00 PM	115.9	72.8	632.8	169.7
1/1/2009	7:20:00 PM	114.6	71.4	632.8	166.3
1/1/2009	7:40:00 PM	113.3	70	632.8	163
1/1/2009	8:00:00 PM	111.9	68.6	632.8	159.9
1/1/2009	8:20:00 PM	110.6	67.3	632.8	156.9
1/1/2009	8:40:00 PM	109.4	66.1	632.8	154

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1/1/2009	9:00:00 PM	108.6	64.9	632.7	151.2
1/1/2009	9:20:00 PM	108	63.7	632.7	148.5
1/1/2009	9:40:00 PM	107.6	62.6	632.7	146
1/1/2009	10:00:00 PM	107.3	61.6	632.7	143.6
1/1/2009	10:20:00 PM	107	60.6	632.7	141.3
1/1/2009	10:40:00 PM	106.7	59.7	632.7	139.2
1/1/2009	11:00:00 PM	106.4	58.9	632.7	137.1
1/1/2009	11:20:00 PM	106.1	58	632.7	135.2
1/1/2009	11:40:00 PM	105.9	57.2	632.7	133.4
1/2/2009	12:00:00 AM	105.6	56.5	632.6	131.7

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APPENDIX A

Document 10

Stantec Consulting Services Memorandum September 30, 2010, Reference: "Hydrologic and Hydraulic Calculations Summary, Spillway Improvement Project, Widows Creek Fossil Plant (WCF), Gypsum Stack Stilling Pond"

Memo

Stantec

To:	Michael S. Turnbow	From:	Erman Caudill, PE, CFM
			Brad Allgeier
	Chattanooga, TN		Lexington, KY
File:	175660008	Date:	September 30, 2010

Reference: Hydrologic and Hydraulic Calculations Summary
Widows Creek Fossil Plant (WCF)
Gypsum Stack Stilling Pond

The purpose of this memorandum is to summarize the hydrologic and hydraulic calculations preformed to check the freeboard and storm surge capacity for the Gypsum Stack Stilling Pond at the Widows Creek Fossil Plant. Stantec was first involved in assessing this capacity in the Spring of 2009 and subsequent design work involved both Stantec and URS. The conditions described herein represent the conditions at the time of assessment and subsequent construction modifications may have changed those conditions.

BACKGROUND

During the initial reconfiguration of the Gypsum Stack embankment and spillway system, Stantec performed a hydrologic and hydraulic analysis of the Gypsum Stack Stilling Pond to check available freeboard and storm surge capacity during the 100-year 24-hour and 6-hour Probable Maximum Precipitation (PMP) storm event.

WATERSHED & PROCESS FLOW

The area draining to the Gypsum Stack Stilling Pond includes direct runoff from approximately 143 acres and the daily plant process flow. The daily plant process flow from the slurry lines averages roughly 8.2 million gallons per day (MGD).

OUTLET DESCRIPTION

Flow discharges from the Gypsum Stack through a spillway system that was reconstructed as a part of the Stantec design efforts. The spillway system dumps into the Stilling Pond which then discharges through a 30-inch diameter steel outlet pipe (invert elevation approximately 612 ft) through the embankment to a channel that leads to the Tennessee River. Aside from the primary spillway, there is one defined emergency spillway at the Stilling Pond that empties to Widows Creek. The crest of the

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spillway was initially set at 619.6 ft and was 150 ft long. A rating curve was developed for the combination of the outlet structures, and is shown in Attachment B.

During our assessment efforts TVA implemented changes to this spillway that raised the height of the embankment and created a larger spillway. Stantec never received as-built survey data for this embankment and the performance has not been re-assessed. The conditions described herein are based on the elevations and geometry at the time of analysis.

FREEBOARD

TVA's Master Programmatic Document requires 5 ft of operating freeboard for the stilling pond. The perimeter dike crest elevation was found to be approximately 620 ft. The stilling pond water surface elevation (WSE) was currently maintained at approximately 613.7 ft during maximum process water discharges, resulting in an operating freeboard of approximately 6 ft. Assuming this freeboard has been maintained since that time, this facility MEETS freeboard requirements.

METHODOLOGY SUMMARY

Stantec also assessed the storm surge capacity of the Gypsum Stack and Stilling Pond. A depth of 7.4-inches was used for the 100 year 24 hour event. This depth was estimated from the map on page 56 of *Technical Paper No. 40; Rainfall Frequency Atlas of The United States* by US Department of Commerce. A depth of 36.3-inches was used for the PMP 6 hour event. This depth was estimated from a map on page 46 of *Hydrometeorological Report No. 56; Probable Maximum and TVA Precipitation Estimates With Areal Distribution for Tennessee River Drainage Less Than 3,000 Mi^2 in Area* by US Department of Commerce.

Hydrologic Engineering Center's - Hydrologic Modeling System (HEC-HMS) version 3.3 was used to check the freeboard of the Gypsum Stack Stilling Pond. The SCS curve number method was used to convert this rainfall into runoff. Composite curve numbers were developed for the contributing drainage areas based off of the landcover and hydrologic soil group (HSG). Lag time assumptions were based on topographic mapping and delineated longest flow paths. Stage-storage relationships for the Gypsum Stack Stilling Pond were developed using contour data and hydrographic survey data provided by TVA.

DESIGN STORM PERFORMANCE

TVA's Master Programmatic Document requires ash pond facilities to convey the 6-hour PMP event without overtopping. Results of the H & H analysis indicate the Stilling Pond

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could pass the 100-year 24-hour event without overtopping the embankment, however it could not pass the 6-hour PMP event.

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

Table 1 – Freeboard and Routing Summary

	100 yr – 24 hr	PMP 6 hr
Drainage Area (ac)	143	143
Crest of Dam (ft)	620	620
Normal Pool Elevation (ft)	613.7	613.7
Normal Operating Freeboard (ft)	8	8
Normal Operation Flow (MGD)	8.2	8.2
Precipitation Depth (in)	7.4	36.3
Max. water surface elevation (ft)	615.7	620.6 Overtopping
Peak Outflow (cfs)	31.7	686.8
Peak Inflow (cfs)	143.1	767.4

CONCLUSIONS AND RECOMMENDATIONS

At the time, Stantec recommended TVA re-establish the emergency spillway in the Gypsum Stack Stilling Pond to the design elevation of 619.0' and consider raising the surrounding embankment to provide adequate storage to safely pass the PMP event when the stack was in a closure scenario. As we understand, TVA has implemented some of the proposed modifications, and the changes should have resulted in the pond being able to safely pass the PMP event. However, an as-built survey has not been provided to Stantec to verify the current conditions and performance for the PMP event has not been verified. Assuming the spillway is at least as large as the previous opening, performance during the 100-yr 24-hour event should be available. URS has since been assisting TVA with portions of this design.

It is recommended that the Gypsum Stack Emergency Spillway be surveyed and re-evaluated as the Gypsum Stack closure design progresses.

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Reference: Hydrologic and Hydraulic Calculations Summary
Widows Creek Fossil Plant (WCF)
Gypsum Stack Stilling Pond

Please let us know if you have any questions or comments concerning this memo or these recommendations.

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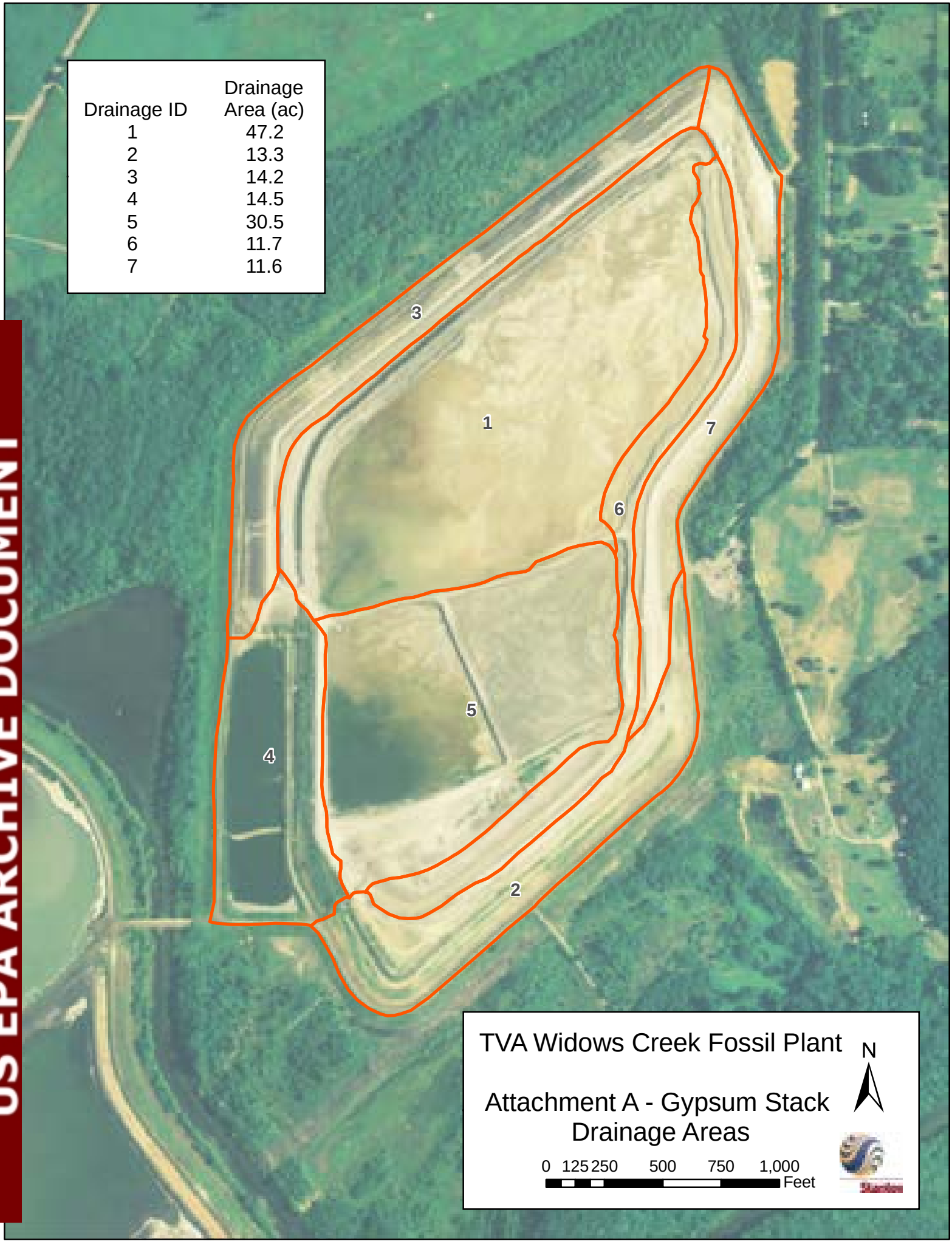
Erman Caudill, PE, CFM
Project Manager & Associate
Erman.caudill@stantec.com

Attachment A: Watershed Map

Attachment B: Hydrologic and Hydraulic Model Input / Output

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Attachment B –

Hydrologic and Hydraulic Model Input / Output

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Legend

Subbasins

CN

- 58
- 71
- 78
- 85
- 86
- 89
- 91
- 94
- 98

TVA Widows Creek Fossil Plant

Curve Number Map

N

0 125 250 500 750 1,000 Feet

Landcover and HSG Table

Landcover	HSG	CN
Pond Area	B	98
Pond Area	D	98
Pond Area	C	98
Gravel Road	B	85
Gravel Road	D	91
Gravel Road	C	89
Covered Landfill	C	71
Covered Landfill	B	58
Covered Landfill	D	78
Active Landfill	B	86
Active Landfill	D	94
Active Landfill	C	91

Composite CN Table

Subbasin	Composite CN
1	96.0
2	76.6
3	72.6
4	86.8
5	96.2
6	90.3
7	74.1

Lag Time Table

Subbasin	Lag Time (min)
1	853
2	33
3	38
4	11
5	492
6	127
7	26

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Distribution of 6-hour PMP event

Time (hh:mm)	Precipitation (in)
00:00	0.00
00:15	0.52
00:30	1.16
00:45	1.92
01:00	2.80
01:15	3.86
01:30	5.08
01:45	6.44
02:00	9.39
02:15	15.15
02:30	21.45
02:45	23.88
03:00	25.41
03:15	26.94
03:30	28.21
03:45	29.42
04:00	30.52
04:15	31.40
04:30	32.31
04:45	32.91
05:00	33.61
05:15	34.41
05:30	35.12
05:45	35.74
06:00	36.30

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Stage - Storage

Elevation	Area (acres)	Cumulative Storage (ac-ft)
612	5.60	0.00
613	5.80	5.70
614	6.00	11.60
615	6.20	17.70
616	6.30	23.95
617	6.50	30.35
618	6.70	36.95
619	6.90	43.75
619.6	6.96	47.91
620	7.10	50.74
621	7.10	57.78

Interpolated or Extrapolated Value

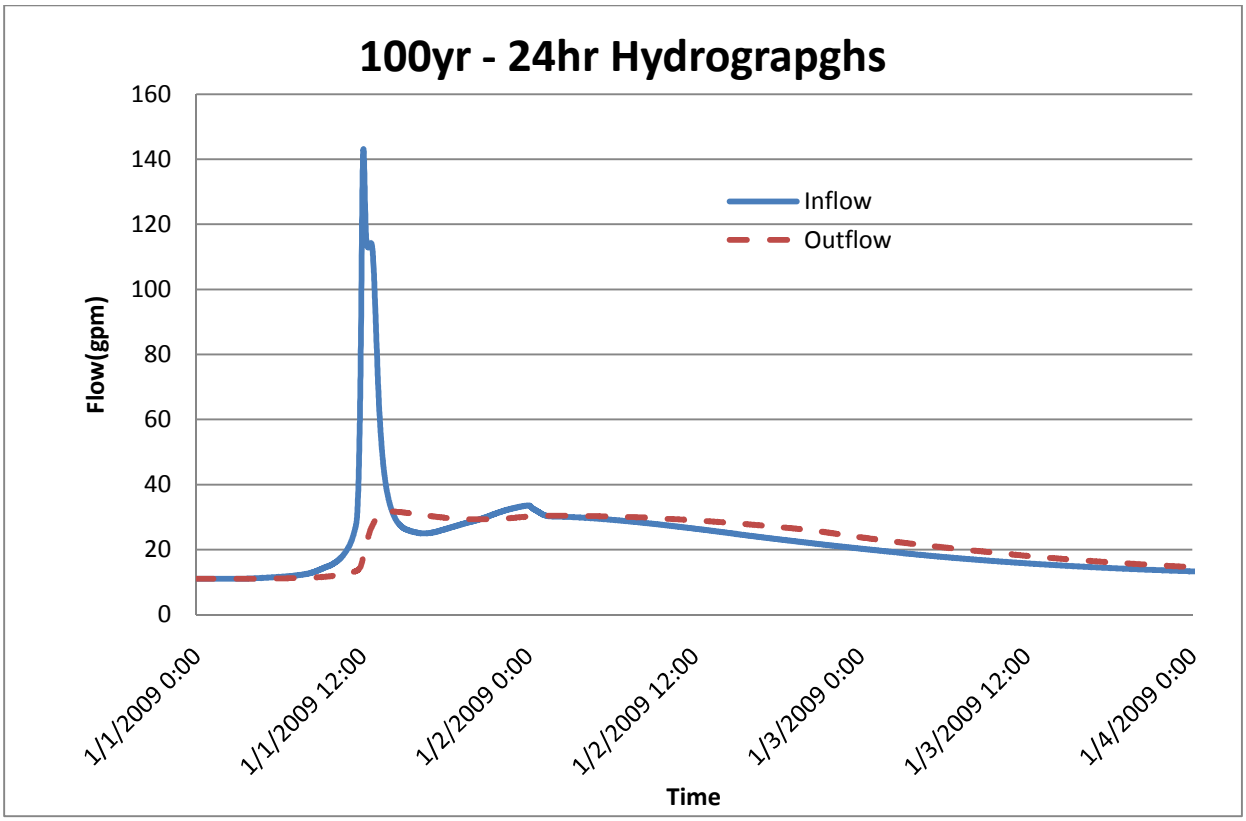
Rating Curve

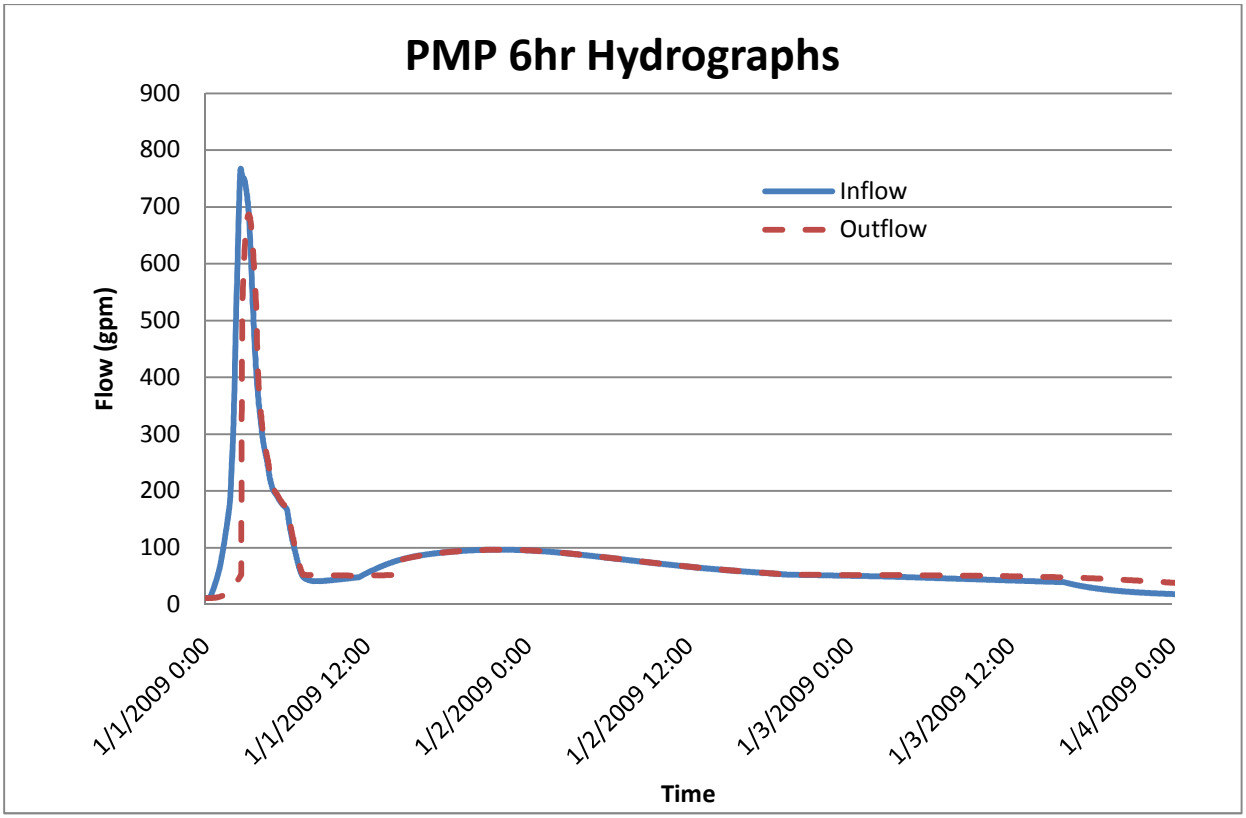
Elevation	Discharge Pipe 30" Stl. Barrel @ 0.5%		Overflow to Creek 150' Emb. Overflow		Composite Discharge		
	H	Q (Hydrocalc)	H	Weir $Q=3.087LH^{1.5}$	Principal Spillway (cfs)	Overflow (cfs)	Total Discharge (cfs)
612	0	0			0		0
613	1	5			5		5
614	2	14			14		14
615	3	26			26		26
616	4	34			34		34
617	5	40			40		40
618	6	45			45		45
619	7	50			50		50
619.6	7.6	52	0	0	52	0	52
620	8	54	1	463	54	463	517
621	9	58	1.4	767	58	767	825

Controlling, component of composite discharge

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APPENDIX A

Document 11

Stantec Consulting Services Correspondence September 29, 2011, Reference: "Results of Seismic Slope Stability Analysis, Active CCP Disposal Facilities, Widows Creek Fossil Plant"

Stantec Consulting Services Inc.
10509 Timberwood Circle Suite 100
Louisville, KY 40223-5301
Tel: (502) 212-5000
Fax: (502) 212-5055

September 29, 2011 ltr_005_175551015

Mr. Michael S. Turnbow
Tennessee Valley Authority
1101 Market Street, LP 2G-C
Chattanooga, Tennessee 37402-2801

Re: Results of Seismic Slope Stability Analysis
Active CCP Disposal Facilities
Widows Creek Fossil Plant

Dear Mr. Turnbow:

As requested, Stantec Consulting Services Inc. (Stantec) has conducted seismic slope stability analyses to support the U.S. Environmental Protection Agency's assessment of TVA's CCP disposal facilities. The results for Widows Creek Fossil Plant (WCF) are presented in this letter.

1. Introduction

The U.S. Environmental Protection Agency is undertaking a nationwide effort to assess coal combustion product (CCP) disposal facilities. These assessments are now underway for facilities at TVA's fossil plants. To support TVA, Stantec has conducted seismic stability analyses for WCF's active disposal facilities, which include the Gypsum Stack, Dredge Cell (Old Scrubber Sludge Pond), Bottom Ash Stack, and the Main Ash Pond.

The seismic slope stability analyses results presented in this letter employ a pseudostatic approach and are representative of current conditions. For seismic assessment in upcoming closure design of these facilities, TVA will undertake a comprehensive risk/consequences-based approach, with design and mitigation decisions being based on the likelihood and consequences of failure. This approach is described in the document presented in Enclosure A. For WCF, the following closure time frames are currently planned: Gypsum Stack – 2017, Dredge Cell – 2019, Bottom Ash Stack – 2021, Main Ash Pond – 2021.

2. Seismic Stability Analysis Approach

Seismic slope stability has been performed for current conditions using pseudostatic stability methods, where the added inertial load from an earthquake is represented by a simple horizontal pseudostatic coefficient which provides an approximate representation of the dynamic loads imposed by an earthquake. Specifics related to the analyses/approach are as follows:

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- Subsurface data was obtained from the following Stantec geotechnical reports:
    - Report of Phase 2 Geotechnical Exploration; Ash Pond Complex; Widows Creek Fossil Plant; Stevenson, Alabama; February 4, 2010.
    - Report of Phase 2 Geotechnical Exploration; Gypsum Stack; Widows Creek Fossil Plant; Stevenson, Alabama; February 5, 2010.
- SLOPE/W software (from GEO-SLOPE International, Inc.) was used to perform the calculations.
- One existing SLOPE/W cross-section model per disposal facility was selected for analysis. The selected sections are representative of the facility's lowest current static (long-term) factor of safety, with consideration given to proper representation of a release/breach. The selected SLOPE/W models were updated to reflect any significant mitigations or operational changes that have occurred since completion of Stantec's geotechnical studies.
- Undrained shear strength parameters were used.
- Ground motion level corresponding to a return period of 500 years (or approximate exceedance probability of 10% in 50 years) was used for selection of horizontal seismic coefficient. This return period is consistent with seismic stability analysis guidance provided by Tennessee's dam safety regulations Chapter 1200-5-7, "Rules and Regulations Applied to the Safe Dams Act of 1973". Tennessee dam safety regulations were selected for reference since Alabama does not have such regulations. The peak ground acceleration (or seismic coefficient) for a 500 year return period was selected from Table 23 of TVA's March 28, 2011 region-specific seismic hazard study performed by AMEC Geomatrix, Inc.
- A target factor of safety (FS) of 1.0 was considered for comparing results.

3. Results

The results of the pseudostatic stability analyses indicate the following factors of safety:

- Gypsum Stack – 1.9
- Dredge Cell – 1.5
- Bottom Ash Stack – 1.9
- Main Ash Pond – 1.8

These are greater than the target value of 1.0.

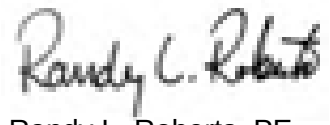
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Enclosure B contains a summary spreadsheet, SLOPE/W cross-sections, and plan views showing cross-section locations.

Stantec appreciates the opportunity to provide these services. If you have questions, or if we can provide additional information, please let us know.

Sincerely,

STANTEC CONSULTING SERVICES INC.



Randy L. Roberts, PE
Principal

Enclosures

/cdm

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Enclosure A

White Paper - Seismic Risk Assessment Closed CCP Storage Facilities

Seismic Risk Assessment
Closed CCP Storage Facilities
Tennessee Valley Authority Fossil Plants

Prepared by:

Alan F. Rauch, PhD, P.E.
Senior Associate
Stantec Consulting Services, Inc.

Wayne Quong, M.A.Sc., P.Eng.
Senior Associate
Stantec Consulting Services, Inc.

Jeffrey S. Dingrando, P.G., P.E.
Associate
Stantec Consulting Services, Inc.

Reviewed and Approved by:

Barry S. Snider, P.E.
TVA-CCP Interim Engineering Manager
Tennessee Valley Authority

William H. Walton, P.E.
Vice President, Senior Principal Engineer
AECOM

Thomas L. Cooling, P.E.
Vice President, Geotechnical Services
URS Corporation

Gonzalo Castro
Independent Consultant

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Seismic Risk Assessment
Closed CCP Storage Facilities
Tennessee Valley Authority Fossil Plants

This document outlines proposed engineering analyses to estimate seismic failure risks at wet storage facilities for coal combustion products, following closure, at various TVA fossil power plants. The specific details outlined in this document are subject to future discussion and modification by the project team.

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OVERVIEW

Tennessee Valley Authority (TVA) operates storage facilities for coal combustion products (CCPs) at eleven fossil power generating stations. As TVA transitions to dry systems for handling these materials, 18 to 25 wet storage facilities (CCP ponds, impoundments, dredge cells, etc.) will be closed (drained and capped). The CCP storage facilities are currently operated in accordance with state and federal regulations, but previously issued permits have not required evaluations for seismic performance. Moreover, the existing permits do not require seismic qualification for the storage facilities in their closed configurations.

TVA recognizes there is a potential for strong earthquakes to occur within the region, and there is a tangible risk for seismic failure at each closed CCP facility. These risks, including both the likelihood of failure and the consequences, must be understood to effectively manage TVA's portfolio of byproduct storage sites. This white paper summarizes the methodology that will be used to estimate these risks at the CCP storage facilities following closure.

Seismicity in the TVA service area is attributed to the New Madrid fault and smaller, less concentrated crustal faults. These two earthquake scenarios generate significantly different seismic hazards at each locality and will be considered independently within the risk assessment. At each closed byproduct facility, potential seismic failure modes will be evaluated in sequence. Instability due to soil liquefaction, slope instability due to inertial loading, and other potential failure mechanisms will be addressed. Seismic performance will be evaluated for differing earthquake return periods until a limiting (lowest return period) event that would cause failure is obtained. The probability of seismic failure will then correspond to the probability of this limiting earthquake event. The assessment of risk will also include estimates of potential consequences, as well as costs to mitigate the risks, that reflects the unique setting of the individual storage facilities after closure.

Following the same general methodology, seismic risks will be estimated in two phases. The near-term "Portfolio Seismic Assessment" will provide a rough estimate of seismic risks. The likely performance of each facility will be evaluated using simplified analyses, empirical methods, and the judgment of experienced engineers. The results will establish a ranking of the relative risks across the closure portfolio and also provide a preliminary picture of overall seismic risk. For the subsequent "Facility Seismic Assessments", seismic performance will be judged on the basis of site-specific data and detailed engineering analyses, which will be completed during the closure design process for individual facilities.

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SEISMIC RISKS

This white paper provides an overview of the engineering methods proposed by Stantec for estimating seismic risks at TVA's closed byproduct storage sites. For each facility, four specific questions must be answered quantitatively:

(1) What is the approximate probability that a strong earthquake will occur?

Several seismic source zones could produce earthquakes large enough to impact these TVA sites. Very large magnitude earthquakes have occurred within the New Madrid seismic zone, which is located along the western boundaries of Tennessee and Kentucky. Because of their observed large magnitude and frequency of occurrence, New Madrid events contribute substantially to the seismic risks at all TVA sites. Ground motions from a New Madrid earthquake would attenuate with distance toward the east, such that local area sources also contribute significantly to site-specific seismic hazards.

Seismicity across the Tennessee Valley was previously characterized by AMEC/Geomatrix (2004), in a probabilistic study that focused on TVA dam sites. The same seismogenic model can be applied in evaluating earthquakes that would impact other TVA sites. Accordingly, probabilistic seismic hazards obtained from the 2004 AMEC/Geomatrix model will be used in the seismic risk assessment of the closed CCP storage facilities.

(2) Will a given earthquake cause failure in the closed facility?

Many of the TVA byproduct storage facilities are underlain by a substantial thickness of loose, saturated, alluvial soils (silts and sands). Some facilities will have layers of ash or other uncemented CCPs that remain saturated following closure. These materials, especially sluiced fly ash, are prone to liquefaction in a strong earthquake, as cyclic motions cause a build up of pore water pressure and a consequent loss of effective stress and shearing resistance. Extensive liquefaction in a foundation or CCP deposit under a storage facility would be expected, in most cases, to result in lateral spreading and massive slope movements (failure). Even without liquefaction, large slope deformations or failures may be triggered by lateral inertial loads during an earthquake. Liquefaction and dynamic loading of slopes are the most likely failure mechanisms, but other seismic failure modes, which may be unique to a particular closed storage facility, must also be evaluated.

(3) What are the potential consequences of a failure?

In addition to understanding the probability of failure, a risk assessment should consider the potential consequences. A failure is likely to have economic costs associated with clean-up and restoration of the site. Depending on the local site conditions, failure of a closed CCP facility may or may not cause significant impacts on the environment, waterways, transportation routes, buried or overhead utilities, or other infrastructure. Substantial economic costs would result if power generation is interrupted. Failure consequences may also include the potential loss of human life at some sites.

In this proposed seismic risk assessment, the definition of "failure" will be constrained to

mean the displacement of stored materials to a distance beyond the permitted boundary of the facility. While smaller deformations in a closed storage facility could cause economic damages, the resulting consequences for TVA should be manageable. Hence, this risk assessment will focus on potential "failures" where stored materials could move past the permitted boundary.

(4) What are the approximate costs to mitigate the risks of a seismic failure?

With an understanding of the probability and consequences of failure, the potential risks can be quantified and understood, possibly leading to decisions to mitigate seismic risks in the closure of certain facilities. Mitigation measures might include ground improvement to reduce liquefaction potential (stone columns, deep soil mixing, jet grouting, or other appropriate technology), stabilization of slopes by flattening or buttressing, enhanced drainage features, or some other engineered solution. The potential cost of these risk mitigation strategies are needed to make appropriate management decisions.

PORTFOLIO AND FACILITY ASSESSMENTS

Seismic evaluations will be completed for each of the CCP storage facilities that TVA has slated for closure; a tentative list is given in Table 1. The assessment of seismic risks will be accomplished in two phases:

A. Portfolio Seismic Assessment

In this first phase, the seismic risk assessment will be carried out using general site information, simplified analyses, empirical methods, and the judgment of experienced engineers. A team of four to five engineers will complete this evaluation for the entire portfolio, with assistance from the engineering teams currently working on each facility. After the probabilistic seismic hazards are defined, this phase of the work can be completed in a relatively short timeframe.

Given the level of effort and the simplified engineering analyses to be employed, the seismic risk estimates from the Phase A assessment will be approximate. Rather than attempting to compute precise risk numbers, Phase A will focus on capturing the relative risks between the different closed facilities. The key to successfully meeting this objective will be the consistent application of the assessment process across the portfolio.

This effort will result in a ranked list of sites that can be used to illustrate where seismic risks are greatest within the portfolio. The results will also provide some insight for understanding and communicating the magnitude of potential risks associated with seismic loading of the closed CCP facilities.

As a secondary objective, the Phase A assessment team will also consider the potential for failure of the active storage facilities, due to an earthquake occurring prior to closure. The seismic risks associated with the operating facility will not be estimated, but the Phase A assessment process provides an opportunity to identify potential failure mechanisms that should be addressed in the short term. This information may suggest the need to re-prioritize the closure schedule. Prior to closure, many of the wet CCP storage facilities retain large pools of water and are thus more susceptible to uncontrolled

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releases in an earthquake. TVA has already made the decision to close these wet storage facilities to manage these risks, so the effort in Phase A will focus on identifying sites that may have unusually high seismic risks and deserve more study or higher priority in the closure program.

B. Facility Seismic Assessment

In this subsequent phase of work, more detailed engineering analyses will be carried out using site-specific geometry, subsurface conditions, material parameters, and results from static slope stability analyses. Simplified, state-of-the-practice methods of engineering analysis will be used; more complex analytical methods will be generally impractical for this risk assessment.

This phase of the work will be accomplished for individual facilities as part of the closure design, after the completion of other engineering analyses. The risks will be quantified by the design team, with assistance from the portfolio seismic assessment team. Significant, detailed effort will be required to assess each closed facility.

Compared to Phase A, the risk estimates obtained at this stage will be more reliable and better represent the actual risks for seismic failure. While it will be impossible to know how accurately the risks have been characterized at the completion of Phase B, the objective is to obtain results that are within perhaps ± 30% of the "actual" risk numbers. TVA expects to use the Phase B results to decide if the risks are acceptable, or if the closure design should be modified to mitigate risks for a seismic failure.

The engineering methodology (described below) to be followed in the Phase A and B evaluations will not characterize all of the uncertainties with respect to seismic performance. The uncertainties in the soil parameters and in the liquefaction, stability, and deformation analyses will not be quantified and carried through the risk assessment. Consequently, the estimated risk numbers will be approximate, but the results will be sufficiently accurate to support TVA decisions regarding prioritization for closure or the need for seismic mitigation. At most sites, the risks are expected to be high enough or low enough that further refinement in the risk numbers would not change these decisions. More detailed analysis beyond Phase B would be unjustified in these cases.

This assessment plan does not preclude the possibility that more detailed risk evaluations could be undertaken in subsequent phases of work. The Phase B results might reveal a subset of closed facilities with marginal risks, where a more rigorous and complete calculation of the risks would be needed to support a management decision. Hence, at the conclusion of the Phase B assessments, a "Phase C" evaluation may be needed for select sites and facilities, wherein uncertainties in the soil parameters and performance analyses would be quantified and carried through the risk assessment.

RESULTS AND APPLICATION

The results from the Phase A Portfolio Assessment will be presented in a table, like Table 1. For each facility evaluated, the estimated annual probability of failure due to a seismic event, the expected consequences (economic costs and potential loss of life), and the mitigation costs (design features to reduce risks) will be tabulated. The same parameters, but more

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accurate numbers, will be reported from the more in-depth Phase B assessments. A qualitative description of the data quality (based on the number of borings, test data on key soil properties, etc.) will also be included, to indicate how well the site conditions were characterized at the time of the Phase A or B assessment.

In both Phase A and B, the evaluation teams will prepare a discussion of significant issues driving the seismic risks at each site. This summary will include knowledge gaps, likely failure mechanisms, unique consequences, suggested approaches for risk mitigation, and other key information. The Phase A evaluation of a facility may point out the need for additional data to support later seismic analyses in Phase B; needed field or laboratory testing could then be accomplished and documented as part of the facility closure design effort.

In the short term, TVA will utilize the Phase A results to better plan budgets and schedules for managing the closure process over the next several years. The Phase A assessment will also be used as an opportunity to identify operating facilities with especially high seismic risks. While these risks will not be quantified for conditions prior to closure, the consideration of potential seismic failure modes may prompt additional study and reconsideration of priorities. Where justified, the priorities for closure may be changed to more quickly address sites with higher seismic risks.

More accurate risk estimates will be obtained from the Phase B assessments, which will be completed as part of the closure design process. Those results will be used, within TVA's existing decision making framework, to judge if seismic mitigation is needed. For context, the criteria in Tables 2 and 3 represent the risk-based framework TVA uses to guide enterprise-level decisions. This framework relies upon broad, qualitative scoring of consequences and risks for the organization. For managing the seismic risks at the closed CCP facilities, complete probabilistic calculations of risk are not needed; approximate estimates of seismic risk will be sufficient to support TVA decisions.

The risks computed in Phase A and B will not be compared to a prescribed threshold or design risk level. Criteria for tolerable seismic risk in these closed CCP storage facilities has not been defined in the existing permits, in TVA policy, or in TVA design guidance.

METHODOLOGY

The same general methodology, outlined in ten steps below and in Figures 1 through 4, will be used to evaluate seismic risk in both the Phase A Portfolio Assessments and the Phase B Facility Assessments. While advanced engineering analyses may be required to demonstrate acceptable seismic performance in a design situation, simplified analyses will be used here, consistent with the goal of estimating the probability of failure.

In Step 1, seismic hazard parameters will be defined for each site; the results will be used as inputs for both the Phase A and Phase B assessments. Then, the evaluation of a particular facility will begin with a review of existing site information (Step 2), followed by engineering analyses for seismic performance. As described in Steps 3 through 7 below, the engineering analyses in Phase B will be more detailed than the simplified estimates in Phase A. The analyses will commence with an initial selection of an earthquake return period and evaluation for seismic performance. Steps 3 through 7 will be repeated until the limiting (lowest) earthquake return period expected to cause failure is obtained. Flowcharts

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summarizing Steps 1 through 7 in the Phase A and B seismic performance assessments are given in Figures 3 and 4, respectively. The earthquake event with the lowest return period that causes failure will then be used to compute the probability of failure in Step 8. The potential consequences and mitigation costs will be estimated in Steps 9 and 10.

Step 1 – Define Seismic Input Parameters

Seismic hazards at TVA dam sites were quantified in a 2004 study by AMEC/Geomatrix. The New Madrid fault zone and several area source zones contribute to the seismicity of the region, as represented schematically in Figure 1. The New Madrid seismic zone is characterized by a large linear, combined reverse/strike-slip fault. Earthquakes in the area source zones are more diffuse (less concentrated in clusters) and tend to occur in zones of weakness of large crustal extent rather than along narrow, well-defined faults. Earthquakes occurring within the New Madrid Seismic Zone and in area sources outside of it will be considered in developing seismic input parameters for each CCP facility. However, only seismic source zones that contribute significantly to the ground motion hazard at a particular site will be used to develop seismic input parameters.

The national USGS seismic hazard model will not be used in these seismic risk assessments; instead, TVA will ask AMEC/Geomatrix to compute the site-specific seismic hazards for each closed CCP facility. The needed information can be obtained from the existing seismogenic model, but will need to separately consider the hazards associated with the New Madrid events and all other seismic sources (Figure 2), hereafter referred to in this white paper as the “earthquake scenarios”. The following parameters are needed for each earthquake scenario:

- Uniform hazard spectra for frequencies from 0.25 to 100 Hz (100 Hz value is equivalent to peak ground acceleration, PGA) at the top of rock for a range of return periods from 100 to 2,500 years.
- De-aggregation for relevant ground motion frequencies (one or more of the following: 0.5, 1.0, 2.5, 5.0, and 100 Hz) at each return period. The de-aggregation results will be used to select appropriate, representative earthquake parameters (magnitude and distance from the site), from which inputs needed for liquefaction analyses can be developed.

In the Phase A effort, the project team (including seismologists designated by TVA) will meet to consider the earthquake hazard data produced by the AMEC/Geomatrix model for each site. The team will reach consensus on the appropriate parameters (return period, earthquake magnitude, and peak ground acceleration) to be used in evaluating each facility, before proceeding with work on subsequent steps of the analysis. The seismic parameters to be tabulated (Table 4) will then be used in both the Phase A and Phase B assessments.

Ground motion time histories will be needed for the detailed Phase B calculations, and TVA will need to ask AMEC/Geomatrix to provide:

- Representative acceleration time histories (two orthogonal components), representing ground motions at the top of the rock profile for the specified earthquake return periods.

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Given the results of the Phase A assessment, the Phase B analyses will focus on a narrower range of possible earthquakes. Hence, acceleration time histories will not be needed for every seismic event listed in Table 4.

Step 2 – Review Site and Facility Information

To meet the requirements for closure of TVA ash storage facilities, the closed condition may involve placement of compacted ash behind a strengthened dike, drainage of pond water to the levels of the surrounding groundwater table, and capping of the area with native soils. The collection of available site information for each facility will be reviewed from a seismic performance perspective. For the Phase B assessment, this information will be augmented with new data that becomes available during the closure design process.

The project information needed for each storage facility includes:

- Planned geometry of the closed storage facility, as needed to meet current design criteria and regulatory requirements.
- Geologic mapping and related information about the site geology.
- Historical records and other information related to site development.
- Boring logs, SPT data, CPT data, shear wave velocities, etc. from field explorations.
- Laboratory data from testing of site materials, including classification, Atterberg limits, moisture content, particle size, specific gravity, unit weight, compaction tests, and other relevant test data.
- Laboratory data on measured strength properties, for both drained and undrained conditions.
- Previously completed slope stability analyses, where available, will be modified for calculations in the risk assessments.

Step 3 - Evaluate Potential for Soil Liquefaction

The potential for soil liquefaction may be the greatest contributor to failure risk at many of the TVA storage sites. Liquefaction will thus be considered first in the assessment of seismic performance at each closed facility (Figures 3 and 4).

The Phase A assessment will utilize empirical charts and back-of-the-envelope calculations to judge if liquefaction would be likely for a given earthquake scenario. For example, Ambraseys (1988) compiled magnitude, epicentral distance, and whether or not liquefaction was observed in past earthquakes, and then suggested a threshold boundary (in terms of magnitude and epicentral distance) where liquefaction might occur in natural soil deposits. Selected, parametric calculations with the simplified procedure outlined by Youd et al (2001) will also be useful in judging what earthquakes would cause liquefaction in the Phase A Portfolio Assessments. These empirical methods may be unconservative for evaluating saturated CCPs, which are often more prone to liquefaction than a sandy soil, but the results will still provide useful guidance in the Phase A assessment.

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For the Phase B liquefaction evaluations, detailed engineering analyses will be undertaken to obtain estimates of cyclic loading, soil resistance, and factor of safety as described below. Potentially liquefiable soils include saturated alluvial soils, loose granular fills, and sluiced ash. The detailed analyses will focus on critical cross sections of the closed facilities; liquefaction safety factors will not be computed for all boring locations at a site.

(a) Soil Loading from Earthquake Motions

The magnitude of the cyclic shear stresses induced by an earthquake are represented by the cyclic stress ratio (CSR). The simplified method proposed by Seed and Idriss (1971) will be used to estimate CSR in the Phase A parametric analyses (ground response analyses will not be completed in Phase A).

In Phase B, the CSR at specific locations (borings and depths where in situ penetration resistance are measured) will be computed using one-dimensional, equivalent-linear elastic methods as implemented in the ProSHAKE software. Using an acceleration time history at the top of rock (obtained from the seismic hazards study in Step 1), the computer program will model the upward propagation of the ground motions through a one-dimensional soil profile. For cases where the one-dimensional assumption is inadequate, the calculations can be accomplished using QUAKE, a two-dimensional finite element program that implements the same dynamic modulus reduction curves and damping relationships as used in ProSHAKE.

The cyclic stresses imparted to the soil will be estimated from the earthquake parameters described in Step 1, representing earthquakes on the New Madrid fault and local crustal events.

(b) Soil Resistance from Correlations with Penetration Resistance

The resistance to soil liquefaction, expressed in terms of the cyclic resistance ratio (CRR), will be assessed using the NCEER empirical methodology (Youd et al. 2001). Updates to the procedure from recently published research will be used where warranted. The analyses will be based on the blowcount value (N) measured in the Standard Penetration Test (SPT) or the tip resistance (q_c) measured in the Cone Penetration Test (CPT). In Phase A, typical or representative values will be used in parametric hand calculations; detailed data from site-specific explorations will be analyzed in Phase B.

The NCEER procedure involves a large number of correction factors. Based on the site-specific conditions and soil characteristics, engineering judgment will be used to select appropriate correction factors consistent with the consensus recommendations of the NCEER panel (Youd et al. 2001). To avoid inappropriately inflating the CRR, the NCEER fines content adjustment will not be applied where zero blowcounts (“weight of hammer” or “weight of rod”) are recorded. The magnitude scaling factor (MSF) is used in the empirical liquefaction procedure to normalize the representative earthquake magnitude to a baseline 7.5M earthquake. The earthquake magnitude (M) considered to be most representative of the liquefaction risk will be determined by applying the MSF to the de-aggregation data (from Step 1) for each selected earthquake return period.

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Saturated fly ash, where it remains following closure, is likely to be more susceptible to liquefaction than indicated by these empirical methods. Values of CRR determined via the NCEER procedure are related to the observation of liquefaction in natural soils, mostly silty sands. Given the spherical particle shape and uniform, small grain size of fly ash, the NCEER procedure may give CRR values that are too high for saturated fly ash.

Lacking better methods of analysis, the lower-bound, "clean sand" base curve (Youd et al. 2001) will be assumed to apply for fly ash in the Phase A assessment. Within the liquefaction calculations, this will be accomplished for these materials by neglecting the fines content adjustment to the normalized penetration resistance. For Phase B, published and unpublished data from cyclic laboratory testing on similar materials will be sought to augment the indications of liquefaction resistance obtained from in situ penetration tests.

(c) Factor of Safety Against Liquefaction

The factor of safety against liquefaction (FS_{liq}) is defined as the ratio of the liquefaction resistance (CRR) over the earthquake load (CSR). Following TVA design guidance and the precedent set by Seed and Harder (1990), FS_{liq} is interpreted as follows:

- Soil will liquefy where $FS_{liq} \leq 1.1$.
- Expect substantial soil softening where $1.1 < FS_{liq} \leq 1.4$.
- Soil does not liquefy where $FS_{liq} > 1.4$.

Using this criteria for guidance, values of FS_{liq} computed throughout a soil deposit or cross section (at specific CPT-q_c and SPT-N locations) will be reviewed in aggregate. Occasional pockets of liquefied material in isolated locations are unlikely to induce a larger failure, and are typically considered tolerable. Instead, problems associated with soil liquefaction are indicated where continuous zones of significant lateral extent exhibit low values of FS_{liq}. Engineering judgment, including consideration for the likely performance in critical areas, will be used for the overall assessment of each facility. A determination of "extensive" or "insignificant" liquefaction will then lead to the appropriate stability analyses in the next stage of the evaluation, as indicated in Figures 3 and 4.

Step 4 – Characterize Post-Earthquake Soil Strengths

The post-earthquake shearing resistance of each soil and CCP will be estimated, with consideration for the specific characteristics of that material. The full, static shear strength will be assigned to unsaturated soils. Excess pore pressures will not develop in an unsaturated soil during seismic loading, so drained strength parameters can be used. The undrained strengths of saturated soils will be decreased to account for the softening effects of pore pressure buildup during the earthquake. Specifically:

- In saturated clays and soils with $FS_{liq} > 1.4$, 80% of the static undrained strength will be assumed.
- In saturated, low-plasticity, granular soils with $1.1 < FS_{liq} \leq 1.4$, a reduced strength will be assigned, based on the excess pore pressure ratio, r_u (Seed and Harder 1990).

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Typical relationships between FS_{liq} and r_u have been published by Marcuson and Hynes (1989).

- In saturated, low-plasticity, granular soils with $FS_{liq} \leq 1.1$, a residual (steady state) strength (S_{us}) will be estimated for the liquefied soil. Values of S_{us} can be obtained from the empirical correlations published by Seed and Harder (1990), Castro (1995), Olson and Stark (2002), Seed et al. (2003), and Idriss and Boulanger (2008).

Subsequent stability and deformation analyses will be accomplished using these reduced strength parameters. No attempt will be made to model the cyclic reduction in soil shear strength during an earthquake. In the deformation analyses, the fully reduced strengths will be assumed at the start of cyclic loading, which will yield conservative estimates of slope displacements.

Step 5 – Analyze Slope Stability

The next step in the performance evaluation (Figures 3 and 4) will consider slope stability, for conditions with or without significant liquefaction. Slope stability will be evaluated using two-dimensional, limit equilibrium, slope stability methods. Reduced soil strengths (from Step 4), conservatively representing the loss of shearing resistance due to cyclic pore pressure generation during the earthquake, will be used in the stability calculations. The analyses will be accomplished using Spencer's method of analysis, as implemented in the SLOPE/W software, considering both circular and translational slip mechanisms.

Input files for static stability calculations, where previously completed for a particular facility, will be updated to represent seismic conditions. These stability analyses may be not available, or the closure geometry may be undefined, for the Phase A assessment of some sites. In those cases, simplified or approximate geometries will be developed for approximate analysis in Phase A. Engineering experience will also be useful in judging likely seismic stability. For example, a complete failure is likely if liquefaction undermines the foundation of the outslope. In the absence of liquefaction, a slope that exhibits adequate safety factors under static conditions is unlikely to fail in an earthquake. Back-of-the-envelope hand calculations can be useful in assessing stability where extensive liquefaction occurs in the saturated materials within or below CCPs retained by a stable perimeter dike. Detailed slope stability calculations, which accurately represent the planned closure geometry, will be used in the Phase B facility assessments.

(a) Slope Stability if Extensive Liquefaction

If extensive liquefaction is indicated, stability will be evaluated for the static conditions immediately following the cessation of the earthquake motions. Residual or steady state strengths will be assigned in zones of liquefied soil, with reduced strengths that account for cyclic softening and pore pressure build up assumed in non-liquefied soil. In both Phase A and B, complete failure (large, unacceptable displacements) will be assumed if the safety factor (FS_{slope}) computed in this step is less than one (Figures 3 and 4).

For slopes where the post-earthquake $FS_{slope} \geq 1$, deformations will be estimated in the Phase B assessment (Step 6 and Figure 4). Slope deformations will not be estimated in the Phase A portfolio assessment, where ground motion time histories will not be available. In Phase A, slopes exhibiting $FS_{slope} \geq 1$ with liquefaction will be assumed

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stable with tolerable deformations; this condition may exist, for example, where liquefied ash at the base of a closed storage facility is contained within a stable perimeter dike.

Note that pseudostatic stability analyses are not useful for evaluating a factor of safety where extensive liquefaction is expected, because appropriate pseudostatic coefficients can not be defined.

(b) Slope Stability if No Significant Liquefaction

If no significant liquefaction is expected, seismic stability will be analyzed in Phase A using approximate, pseudostatic stability methods (Figure 3). The added inertial loads from the earthquake will be represented with a simple, horizontal pseudostatic coefficient (k_h), which provides an approximate representation of the dynamic loads imposed by an earthquake. The horizontal pseudostatic coefficient will be set to one-tenth of the peak ground acceleration in rock ($k_h = 0.1 \cdot PGA_{rock}$). In Phase A, tolerable deformations (less than about 5 meters) will be assumed if the pseudostatic $FS_{slope} \geq 1$, and failure will be assumed if the pseudostatic $FS_{slope} < 1$.

This approach and criteria are based on the work of Hynes-Griffin and Franklin (1984). They performed Newmark deformation analyses, integrated over 350 ground motion time histories, used an amplification factor of three to represent peak accelerations at the base of an earth embankment, and assumed a displacement of 1 meter would be tolerable for an embankment dam. For a typical CCP facility, assuming no pool is retained following closure, “failure” would imply displacements significantly greater than 1 meter. A tolerable displacement of about 5 meters will be assumed here, for the Phase A risk assessments. From the upper bound curve plotted by Hynes-Griffin and Franklin (1984), a displacement of 5 meters would correspond to a yield acceleration of about 0.03 times the peak acceleration along the slip surface. Then, assuming an amplification factor of 3 for the ground motions at the base of the embankment, this suggests $k_h = 0.1 \cdot PGA_{rock}$ can be used conservatively in the pseudostatic analysis to judge failure, as described above.

Pseudostatic factors of safety will not be computed in the Phase B assessment. Instead, where a liquefaction failure is not predicted, potential slope displacements will be computed as described in Step 6.

Step 6 – Predict Deformations

In the Phase A Portfolio Assessment, closed facilities that are expected to remain stable (pseudostatic $FS_{slope} \geq 1$ with no liquefaction, or post-earthquake $FS_{slope} \geq 1$ with liquefaction) will be assumed to have tolerable displacements. Dynamic slope deformations are difficult to estimate without detailed analysis; the available empirical or approximate methods do not represent the conditions of interest, or the level of effort is not consistent with the goals of the first phase of risk assessments. In addition, earthquake ground motion time histories will not be available for the Phase A analyses.

In the Phase B Facility Assessments, the potential deformation of stable slopes will be evaluated as indicated in Figure 4. Conventional methods of analysis will be implemented to estimate potential slope displacements that accumulate during earthquake shaking; movements are assumed to stop when the earthquake ends, consistent with a post-

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earthquake safety factor greater than one. The acceleration time histories obtained from the ground response analyses in Step 3a will be used as inputs for computing deformations with one of the following simplified methods:

- Newmark's (1965) method involves double integration of accelerations greater than the yield acceleration (k_y), which will be determined from a succession of pseudostatic slope stability analyses in which k_h is varied. The value of k_h where the pseudostatic FS_{slope} = 1.0 corresponds to the yield acceleration.
- The Makdisi-Seed (1978, 1979) procedure, which better accounts for the dynamic response of embankments. This procedure was developed based on parametric numerical simulations for earthen dams. The procedure is iterative, considers the fundamental periods of the embankment response, and can be completed in steps using published charts. Results from QUAKE can also be used as input in this procedure.

The slope deformations predicted in Phase B will be conservative, because the yield acceleration will be computed based on reduced, post-earthquake soil strengths. In reality, the yield acceleration declines in successive cycles of seismic loading, as pore pressures accumulate and saturated soils become weaker. The analysis outlined in Figure 4 assumes reduced strengths and, where liquefaction is predicted, residual strengths at the start of the earthquake. Detailed numerical simulations can be used to track the progressive softening and liquefaction of soil within an embankment during an earthquake; such analyses are expensive and time consuming. Rigorous analyses of this type will not be justified except in a "Phase C" analysis, or where performance in a given seismic design event must be demonstrated. Note that the logic in Figure 4 might appear to assume a slope will be stable if there is no significant liquefaction; however, the deformation analysis will indicate unlimited deformations and certain failure if $FS_{slope} < 1$ for static, post-earthquake conditions.

Step 7 – Consider Other Potential Failure Modes

For most of the closed facilities, soil liquefaction, slope instability, and slope deformations will be the most likely seismic failure modes. However, depending on the unique configuration of each CCP facility, other potential failure modes may contribute significantly to the seismic risks. For example, the loss of critical drainage structures or retaining walls could lead to a failure condition. Other potential failure modes will be identified and evaluated quantitatively in this step.

As a secondary objective of the Phase A effort, the assessment team will consider the potential for failure of the active storage facilities, due to an earthquake occurring prior to closure. Many of the wet CCP storage facilities retain large pools of water, so this assessment will need to consider additional failure modes such as seepage and embankment cracking. The objective here will be to identify operating facilities that may have unusually high seismic risks, and might deserve more study or higher priority in the closure program.

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Step 8 – Estimate Annual Probability of Seismic Failure

As indicated in the flowcharts in Figures 3 and 4, the assessments of seismic performance (in both the Phase A and Phase B efforts) will consider a range of potential earthquakes with differing return periods. The analyses will be repeated until the limiting (lowest) earthquake return period (from the candidate events defined in Step 1) that predicts failure of a particular CCP storage facility is obtained. Interpolation may be used, as appropriate, to narrow the definition of the limiting earthquake.

The return period for each earthquake scenario (Table 4) represents the annual probability of exceedance for the associated ground motion parameter. Hence, for each earthquake scenario, the event with the smallest return period that causes failure represents a limiting case, where all events having longer return periods would also cause failure. The inverse of the limiting return period thus represents the annual probability of seismic failure due to that earthquake scenario.

Step 9 – Estimate Potential Consequences of Failure

The potential consequences of a failure at each closed facility will be estimated in this step. The potential consequences will be unique to each site, but may include any of the following:

- restoration of the site and storage facility,
- clean-up to address environmental impacts,
- off-site disposal of released materials,
- damages and loss of use for transportation routes, including buried or overhead utilities,
- damages to buildings and other infrastructure,
- economic losses from the possible shutdown of power generation, and
- loss of human life (expected to be unlikely at most sites following closure).

Except for the potential loss of life, the failure consequences will be expressed in terms of present day costs. Detailed cost estimates of the potential consequences of failure will not be attempted in the Phase A assessments; instead, the potential magnitude of total consequence costs will be estimated using broad categories (< $100K, < $500K, < $1M, < $5M, < $10M, < $50M, < $100M). Cost estimates that better reflect the local site conditions will be produced by the closure design teams during the Phase B assessments.

Step 10 – Estimate Possible Mitigation Costs

The final step in the process will involve estimating the costs to mitigate seismic risks, perhaps by altering the closure design to withstand stronger earthquakes. Examples of possible mitigation measures include:

- ground improvements to reduce liquefaction potential (stone columns, deep soil mixing, jet grouting, or other appropriate technology),
- altering the geometry of outslopes (setbacks, benches, or flatter slopes) to improve

stability,

- adding buttresses or other supporting structures at the toe of slopes,
- enhanced drainage features, and
- relocation of infrastructure or people away from potential impact zones.

These mitigation approaches generally involve higher construction costs, which can be quantified in terms of present dollars. As with the consequence costs, detailed estimates of mitigation costs will not be attempted in the Phase A assessments. The potential magnitude of mitigation will be estimated in categories (< $100K, < $500K, < $1M, < $5M, < $10M, < $50M, < $100M). Mitigation cost estimates that better reflect the local conditions and facility layout will be developed by the closure design teams during the Phase B assessments.

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Table 1. Expected Results from the Phase A and B Seismic Risk Assessments

TVA Facility	Prob. Failure	Econ. Costs	Loss of Life	Mitigat. Costs	Data Quality
ALF East Ash Disposal					
ALF East Stilling Pond					
BRF Dry Fly Ash Disposal					
BRF Fly Ash Pond And Stilling Basin Area 2					
BRF Bottom Ash Disposal Area 1					
BRF Gypsum Disposal Area 2a					
COF Disposal Area 5					
COF Ash Pond 4					
CUF Dry Ash Stack					
CUF Ash Pond					
CUF Gypsum Storage Area					
GAF Fly Ash Pond E					
GAF Bottom Ash Pond A					
GAF Stilling Pond B, C & D					
JSF Dry Fly Ash Stack					
JSF Bottom Ash Disposal Area 2					
JOF Ash Disposal Area 2					
KIF Dike C					
PAF Scrubber Sludge Complex					
PAF Peabody Ash Pond					
PAF Slag Areas 2a & 2b					
SHF Consolidated Waste Dry Stack					
SHF Ash Pond					
WCF Ash Pond Complex					
WCF Gypsum Stack					

Prob Failure = Annual probability of failure due to earthquakes
Econ. Costs = Economic costs resulting from a failure
Loss of Life = Potential loss of life resulting from a failure
Mitigat. Costs = Costs to mitigate seismic risks in closure design
Data Quality = Qualitative indication of how well conditions in the facility are characterized

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Table 2. Risk Severity Scoring (Draft) used by TVA

as of 4/22/2009

TVA Risk Event Consequence Rating Scale (Work-In-Progress)						
Strategic Objective	Success Factor	5 Worst Case	4 Severe	3 Major	2 Moderate	1 Minor
Customer	Public Image	International media attention; nearly unanimous public criticism	National media attention; federal, state officials, and customers publicly critical	Regional / local media attention; customers voice concern	Minimal media attention; letters / emails to executive leadership voicing concern	No media attention; sparse criticism
	Rate Impact	Average total retail rate increases by 15%, relative to peers	Average total retail rate increases by 10%-15%, relative to peers	Average total retail rate increases by 5%-10%, relative to peers	Average total retail rate increases by 2%-5%, relative to peers	Average total retail rate increases by 0-2%, relative to peers
People	Safety	Fatalities	Wide spread injuries	Major injuries	Significant injuries	Minor injuries
	Employee Confidence	Widespread departures of key staff with scarce skills or knowledge	Sharp, sustained drop in CHI results; departures of key staff with scarce skills or knowledge	Sharp decline in CHI results	Modest decline in CHI results	No effect on CHI results
Financial	Cash Flow Impact	>$500M	$100M - $500M	$25M - $100M	$5M - $25M	<$5M
	Credit Worthiness	Credit rating downgrade to below investment grade	Credit Rating Downgrade	TVA put on credit watch	TVA put on negative outlook	Credit rating agencies and bondholders express concern
Assets and Operations	LNS (Load not served)*	10% of System Daily Sales (48,000 MWhrs)	1% of System Daily Sales (4,800 MWhrs)	0.1% of System Daily Sales (480 MWhrs)	0.05% of System Daily Sales (240 MWhrs)	140 MWhrs
	CPI (Connection Point Interruptions)	10% of CPs are down simultaneously	5% of CPs are down simultaneously	CPI totaling 10% of current CP count (124 for FY09)	CPI totaling 7.5% of current CP count (93 for FY09)	CPI totaling 5% of current CP count (62 for FY09)
	Duration (in Hours) of Service Interruption	3,000 cumulative hours for CPs	1,000 cumulative hours for CPs	500 cumulative hours for CPs	150 cumulative hours for CPs	50 cumulative hours for CPs
	Delivered Cost of Power	Sustained increase in delivered cost of power >1 year	Increase in delivered cost of power <1 year	Increase in delivered cost of power <1 month	Increase in delivered cost of power <1 week	Delivered cost of power not effected
	Damage to environment; type and magnitude of contamination / discharge	Major coal, nuclear plant accident or dam failure	Significant hazardous waste discharged; nuclear plant accident; dam integrity failure resulting in drawdown of pool elevation	Hazardous materials / waste discharge; clean up / remediation time takes approximately two weeks	Localized environmental damage, no impact to wildlife, clean up / remediation time less than two weeks	Minimal environmental damage, no hazardous discharge; clean up time takes a few days

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Table 3. Risk Likelihood Scoring used by TVA

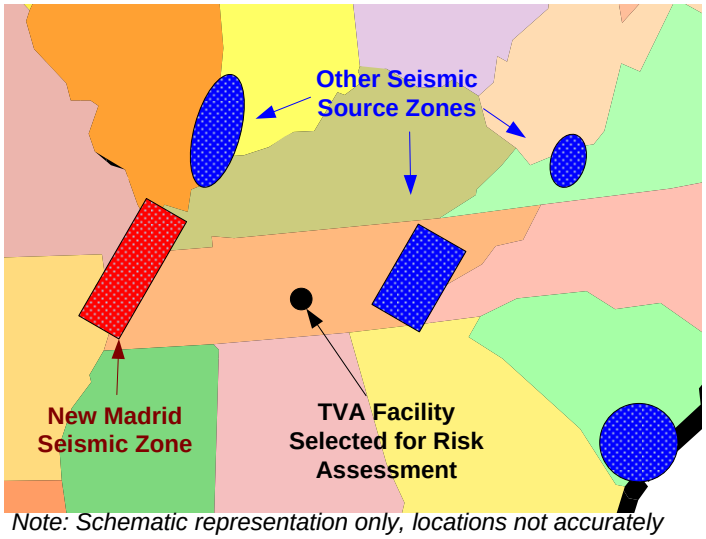
TVA Risk Event Probability Rating Scale		
Score	Rating	Description
5	Virtually Certain	95% probability that the event will occur in the next 3 years /10 years
4	Very Likely	75% probability that the event will occur in the next 3 years/10 years
3	Even Odds	50% probability that the event will occur in the next 3 years/10 years
2	Unlikely	25% probability that the event will occur in the next 3 years/10 years
1	Remote	5% probability that the event will occur in the next 3 years/10 years

- *The 3-year timeframe will be the primary focus for the business unit risk maps*
- *The 10-year risks will be collected by the ERM organization and charted separately for the enterprise*

Table 4. Seismic Hazard Input Data for Probabilistic Assessment of TVA Facilities

Seismic Sources	Return Period (years)	Annual Probability of Exceedance	Peak Ground Acceleration (g)	Earthquake Magnitude
New Madrid Seismic Zone	2,500 1,000 500 250 100	0.0004 0.001 0.002 0.004 0.01	*Values to be determined from the seismic hazard curves*	*Values to be determined from the hazard de-aggregation data**
All Other Seismic Sources	2,500 1,000 500 250 100	0.0004 0.001 0.002 0.004 0.01		

** Representative magnitude corresponding to the maximum contribution to the seismic hazard for liquefaction, as determined from the de-aggregation data weighted by the magnitude scaling factor (maximum PGA / MSF)*



Note: Schematic representation only, locations not accurately depicted, some sources omitted.

Figure 1. Schematic Representation of Seismic Source Model for TVA Facilities

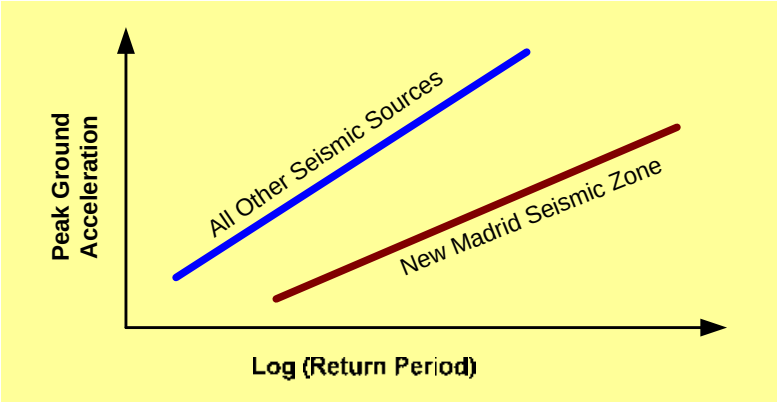


Figure 2. Typical Seismic Hazard Curves for Proposed Probabilistic Assessment of TVA Facilities

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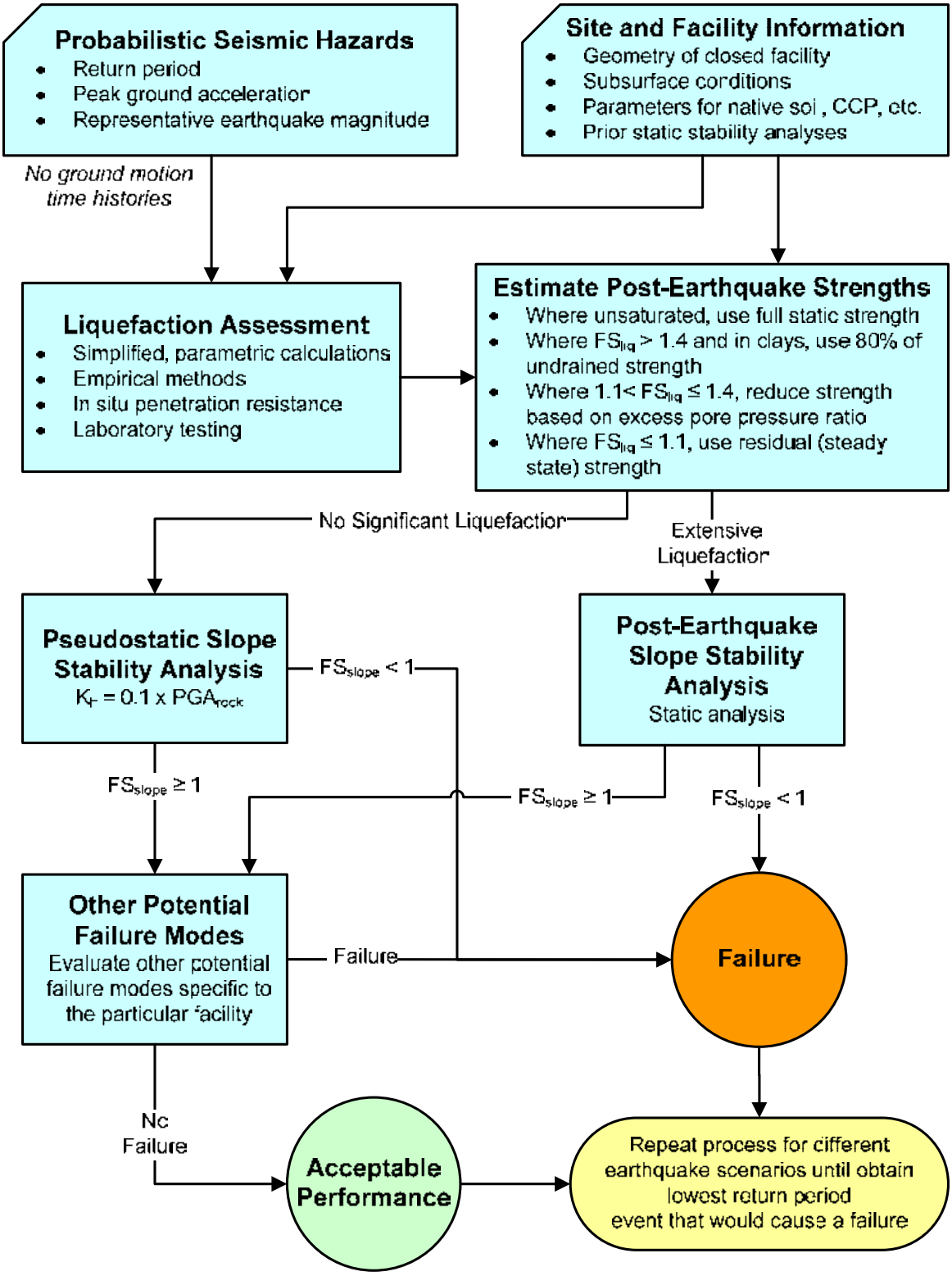


Figure 3. Simplified Flowchart for Assessing Facility Performance During a Probabilistic Seismic Event in Phase A

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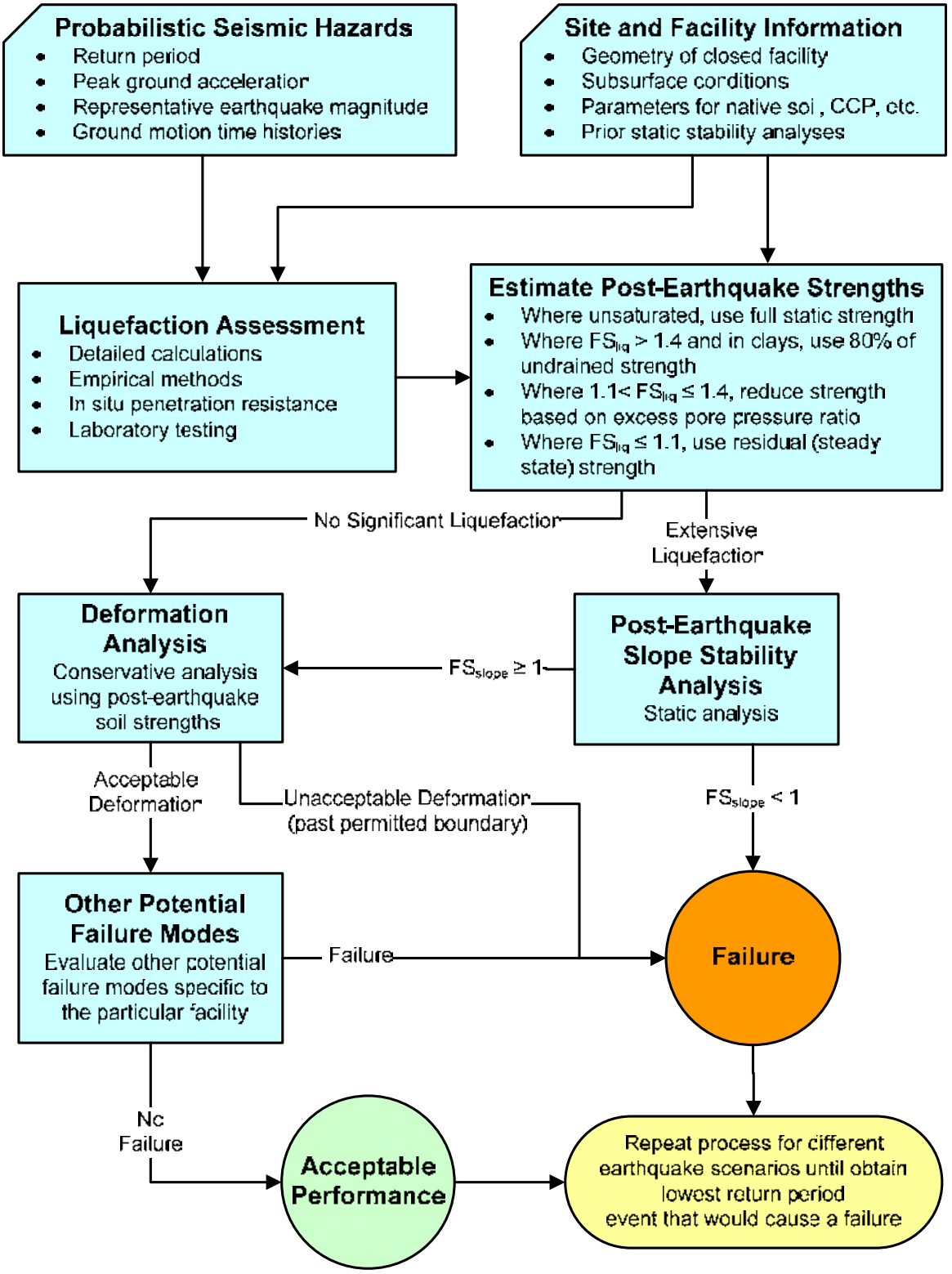


Figure 4. Simplified Flowchart for Assessing Facility Performance During a Probabilistic Seismic Event in Phase B

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Enclosure B

Pseudostatic Analysis Results

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Widows Creek Fossil Plant - Pseudostatic Stability Analysis Summary

CCP Disposal Facility		Cross-Section Information		500 yr Return		Mitigation and Improvement Activities Since January 2009 As-Found Conditions
Name	Type	Section Analyzed	Section Location	PGA (g)	Factor of Safety	
Gypsum Stack	Wet Stack	F	Northeast Side	0.038	1.9	No mitigation activities were necessary along this segment of the Gypsum Stack. As-found static FS was sufficient. Section F represents current and as-found conditions.
Dredge Cell (Old Scrubber Sludge Pond)	Impoundment	D	East Corner		1.5	Design of Immediate Risk Reduction Plans (IRRP #3) are currently on-going to improve static stability, stormwater drainage, and seepage conditions. Section D represents as-found conditions.
Bottom Ash Stack	Stack	M	West Side		1.9	Design of Immediate Risk Reduction Plans (IRRP #4) are currently on-going to improve static stability, stormwater drainage, and seepage conditions. Section M represents as-found conditions.
Main Ash Pond	Impoundment	J	Northeast Side		1.8	Construction activities were completed in 2011 to improve static stability, stormwater drainage, and seepage conditions. This includes installing graded filter toe buttress and lowering the Main Ash Pond pool level. Section J represents these mitigations.

Notes:

1) Acceleration is from March 28, 2011 TVA region-specific seismic hazard study performed by AMEC Geomatrix, Inc. (total hazard).
2) Refer to layout plan for locations of cross-sections.
3) Stability models reflect current ground lines and conditions.
4) Liquefaction was not considered in this analysis.

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

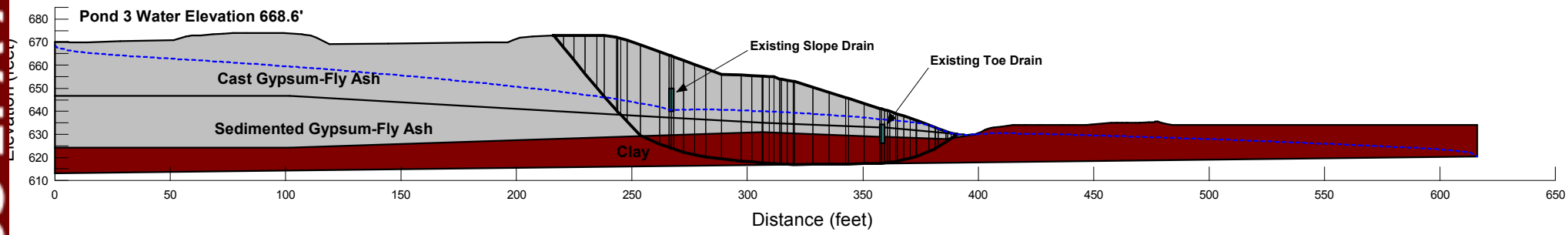
Section F - Gypsum Stack
Widows Creek Fossil Plant
Stevenson, Alabama

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Material Type	Unit Weight	Cohesion	Friction Angle
Cast Gypsum-Fly Ash	113 pcf	0 psf	40 °
Sedimented Gypsum-Fly Ash	112 pcf	0 psf	41 °
Sand Drains	110 pcf	0 psf	33 °
Clay	123 pcf	650 psf	15.7 °

Factor of Safety: 1.91

Horizontal Sesmic Coefficient Kh = 0.038 g
500 year Return Period Event



te of Assessment - 09/09/2011

Project No. 175551015

US EPA ARCHIVE DOCUMENT

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

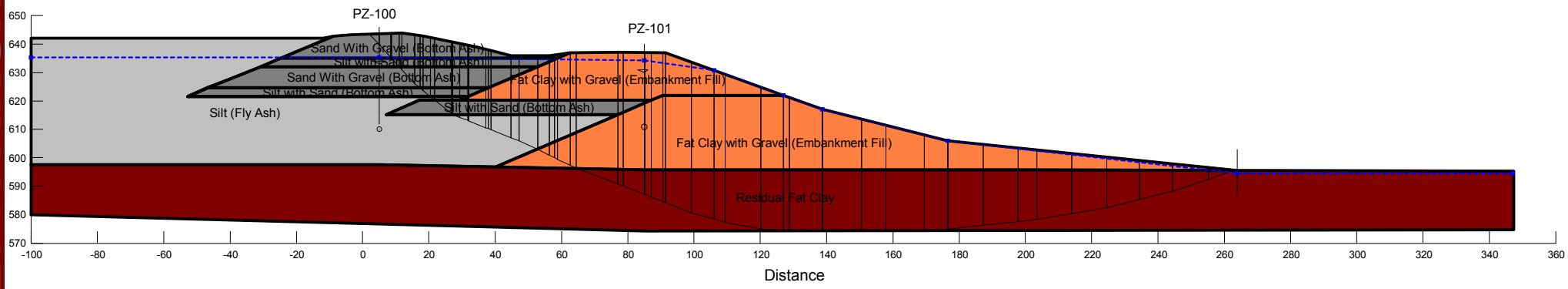
Section D - Dredge Cell (Old Scrubber Sludge Pond)
Widows Creek Fossil Plant
Stevenson, Alabama

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Material Type	Unit Weight	Cohesion	Friction Angle
Residual Fat Clay	125 pcf	650 psf	15.7 °
Silt with Sand (Bottom Ash)	112 pcf	0 psf	33 °
Silt (Fly Ash)	112 pcf	0 psf	21.8 °
Sand With Gravel (Bottom Ash)	119 pcf	0 psf	33 °
Fat Clay with Gravel (Embankment Fill)	125 pcf	1375 psf	14.2 °

Factor of Safety: 1.46

Horizontal Sesmic Coefficient Kh = 0.038 g
500 year Return Period Event



Date of Assessment - 09/09/2011

Project No. 175551015

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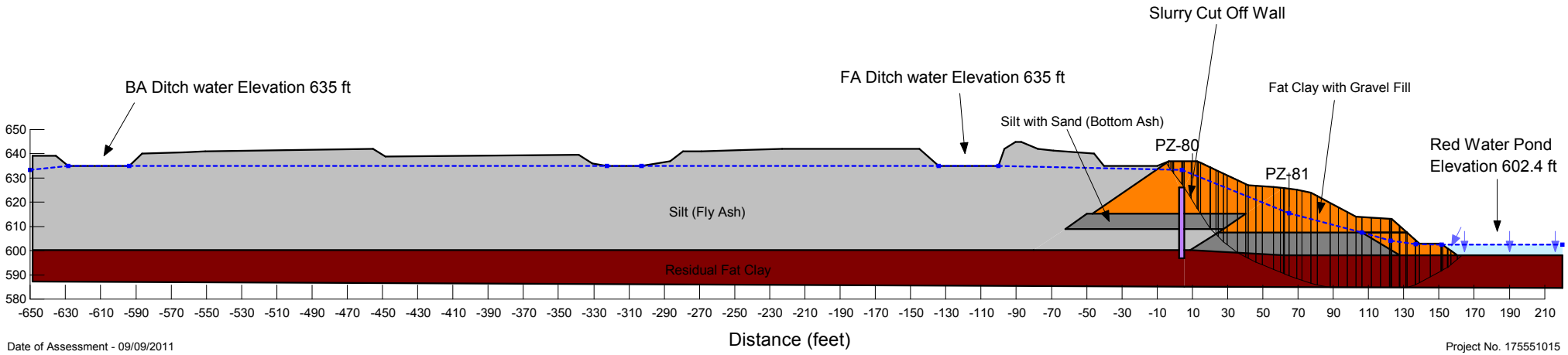
Pseudostatic Slope Stability Analysis
CCP Storage Facilities - Existing Conditions
Tennessee Valley Authority Fossil Plants

Section M - Bottom Ash Stack
Widows Creek Fossil Plant
Stevenson, Alabama

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Material Type	Unit Weight	Cohesion	Friction Angle
Sand with Gravel (Bottom Ash)	119 pcf	0 psf	33 °
Silt with Sand (Bottom Ash)	112 pcf	0 psf	33 °
Silt (Fly Ash)	112 pcf	0 psf	21.8 °
Residual Fat Clay	125 pcf	650 psf	15.7 °
Fat Clay with Gravel (Embankment Fill)	125 pcf	1375 psf	14.2 °

Factor of Safety: 1.93
Horizontal Sesmic Coefficient Kh = 0.038 g
500 year Return Period Event



Date of Assessment - 09/09/2011

Project No. 175551015

US EPA ARCHIVE DOCUMENT

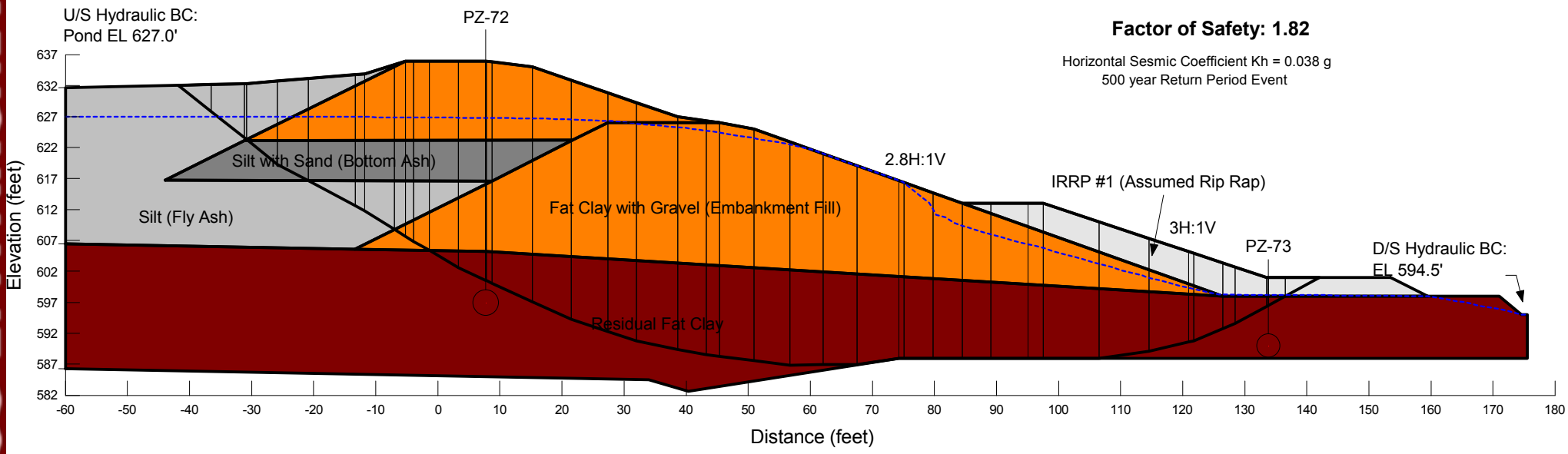
Pseudostatic Slope Stability Analysis
CCP Storage Facilities - Existing Conditions
Tennessee Valley Authority Fossil Plants

Section J - Main Ash Pond
Widows Creek Fossil Plant
Stevenson, Alabama

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Additional remediation measures taken from URS plans dated 8/14/2010.

Material Type	Unit Weight	Cohesion	Friction Angle
Sand with Gravel (Bottom Ash)	119 pcf	0 psf	33 °
Rip Rap	115 pcf	0 psf	40 °
Silt (Fly Ash)	112 pcf	0 psf	21.8 °
Residual Fat Clay	125 pcf	650 psf	15.7 °
Fat Clay with Gravel (Embankment Fill)	125 pcf	1375 psf	14.2 °



Assessment - 09/09/2011

Project No. 175551015

BORING LOCATION TABLE			
Boring	Northing	Easting	Elev. (ft.)
***STN-28	1,600,650.1	527,741.0	651.31
***STN-29	1,600,590.1	527,790.4	623.86
***STN-31	1,601,984.7	527,265.3	672.62
***STN-32	1,602,037.1	528,528.5	656.24
***STN-33	1,602,037.0	528,597.9	640.20
***STN-34	1,603,809.7	528,724.1	674.11
***STN-35	1,603,831.0	528,927.7	656.13
***STN-36	1,603,833.8	529,007.1	631.94
***STN-37	1,603,319.9	527,464.9	626.95
***STN-38	1,602,372.3	526,925.2	675.50
***STN-39	1,602,374.5	526,839.7	655.19
***STN-40	1,602,379.2	526,750.0	621.38
***STN-41	1,601,812.5	527,159.8	657.76
***STN-42	1,601,813.4	527,067.0	659.04
***STN-43	1,601,979.2	527,240.0	672.69
***STN-44	1,601,640.7	527,161.8	655.78
***STN-45	1,601,630.6	527,063.0	655.20
***STN-46	1,601,239.7	527,133.1	654.70
***STN-47	1,601,297.4	527,118.9	655.53
***STN-48	1,601,416.0	527,060.8	654.92
***STN-49	1,601,258.4	527,058.8	655.03
***STN-50	1,601,244.1	527,059.0	654.94
***STN-51	1,601,651.7	526,940.3	621.13
***STN-52	1,601,757.5	526,939.5	621.65
***CPT-10	1,601,250.6	526,991.4	629.0
***CPT-11	1,601,253.2	527,098.3	655.0
***CPT-12	1,601,629.8	526,995.7	629.0
***CPT-13	1,601,650.8	527,105.0	655.0
***V-9	1,601,630.2	527,103.0	655.0
***V-10	1,601,235.3	527,092.3	655.0
*TH-1	1,600,610.0	527,279.3	643.43
*TH-2	1,600,584.4	527,243.0	650.48
*TH-3	1,601,322.1	527,292.9	643.55
*TH-4	1,601,303.2	527,810.5	639.89
*TH-5	1,601,646.1	528,409.3	642.09
*TH-6	1,600,568.4	527,615.1	648.88
*TH-7	1,601,553.5	528,589.9	638.64
**SPT-1	1,601,472.4	527,068.5	649.90
**SPT-2	1,602,279.9	527,011.4	661.89
**SPT-3	1,603,928.6	528,728.1	659.79
**SPT-4	1,602,691.0	528,555.7	663.34
**SPT-5	1,603,183.6	527,640.9	660.18
**SPT-6	1,601,092.3	527,910.1	660.50

* Boring drilled in 1991.
** Boring drilled in 2004.
*** Boring drilled in 2009.

NOTES:

1. Topographic mapping was developed from a composite of a LIDAR survey which was originally performed in NAD 83 and later shifted to NAD 27 Datum by Tuck Mapping Solutions, Inc. on January 9, 2009 (post-event) and surveys performed by TVA on October 30, 2006, January 14, 2009 and February 23, 2009.
2. The geotechnical information and data furnished herein are not intended as representation or warranties but are furnished for information only. It shall be distinctly understood that the Owner or Engineer will not be responsible for any deduction, interpretation or conclusion drawn therefrom. The information is made available in order that the Contractor may have ready access to the same information available to the Owner and the Engineer and is not part of this contract.
3. See sheet 2 for baseline curve data and table of baseline coordinates.

RECORD DRAWING

FOR INFORMATION ONLY
This Record Drawing which has been previously submitted to TVA is provided for Information Only.

Widows Creek Fossil Plant, Gypsum Stack. Cross Section F used to perform pseudostatic slope stability analysis.

STILLING POND
POND 3
POND 2B
POND 2A
POND 1
Project Baseline
GRAPHIC SCALE: 1" = 100'
CONTOUR INTERVAL = 2'

LEGEND

- STN-47 Soil Boring with Undisturbed (Shelby) Tube Samples and Rock Core
- STN-52 Rock Sounding
- STN-34 Soil Boring with Standard Penetration Tests and/or Undisturbed (Shelby) Tube Samples
- CPT-10 Cone Penetration Test with Piezometer
- V-10 Vane Shear Test Location
- *TH-7 Existing Test Boring, 1991
- **SPT-3 Existing Test Boring, 2004

For Supporting Design Calculations see FPGWCFFESCDX00000020100005

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST
R 0	02/05/10	GKA	PJB	RDF	HRA	JSM	JSM	TJ		
ISSUED FOR REVIEW										

SCALE: 1"=100' EXCEPT AS NOTED

YARD
–

GYPSUM STACK GEOTECHNICAL EXPLORATION BORING LAYOUT

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
K. ANDERSON	P. BOND	R. FULLER	H. APARICIO	J. MONTGOMERY	J. MONTGOMERY	T. JOHNSON

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

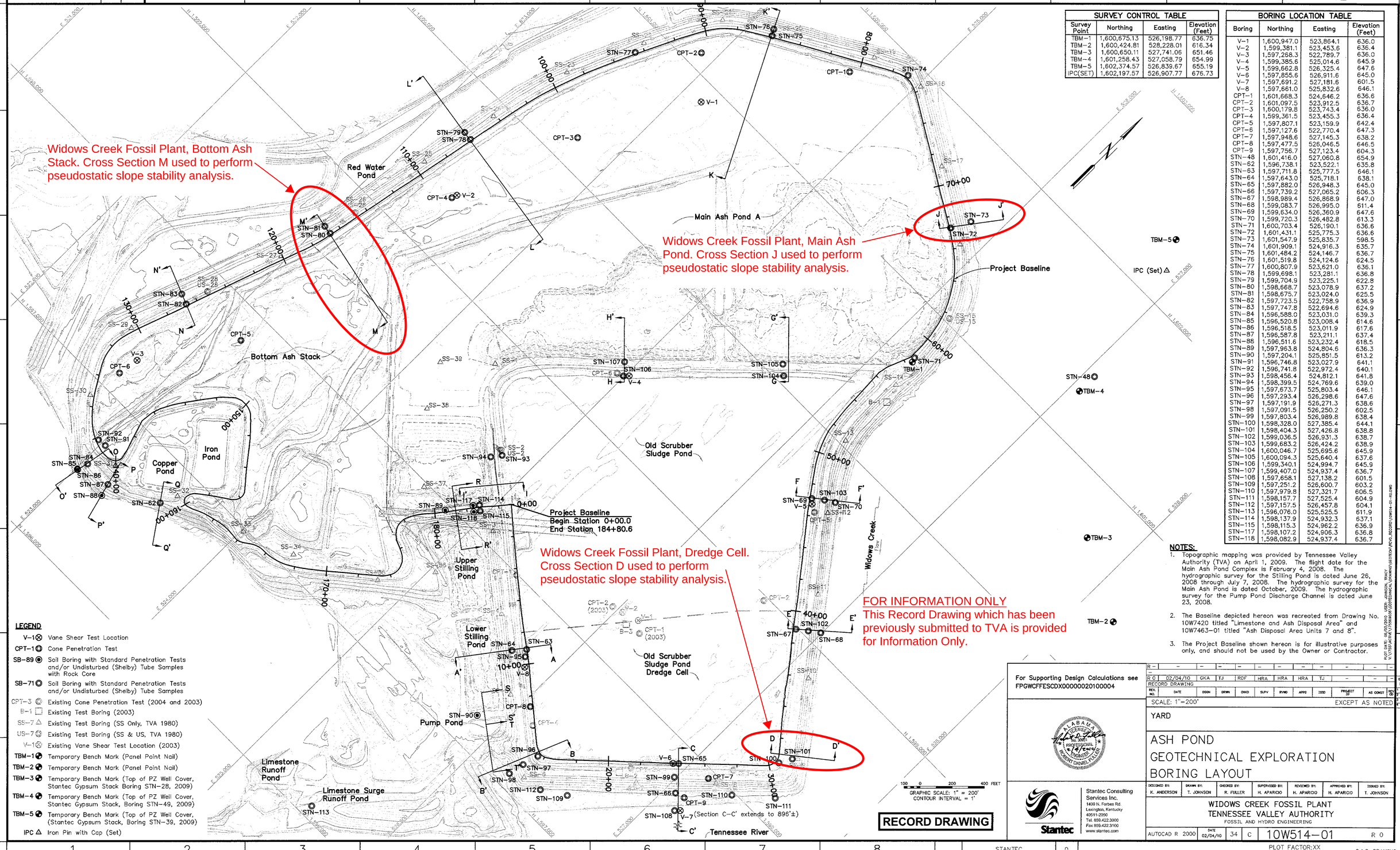
Stantec Consulting Services Inc.
1409 N. Forbes Rd.
Lexington, Kentucky
40511-2050
Tel. 859.422.3000
Fax 859.422.3100
www.stantec.com

AUTOCAD R 2000 | DATE 02/05/10 | 34 | C | 10W515-04 | R 0

STANTEC | 0
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:100
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY



SURVEY CONTROL TABLE

Survey Point	Northing	Easting	Elevation (Feet)
TBM-1	1,600,675.13	526,198.77	636.75
TBM-2	1,600,424.81	528,228.01	616.34
TBM-3	1,600,650.11	527,741.06	651.46
TBM-4	1,601,258.43	527,058.79	654.99
TBM-5	1,602,374.57	526,839.67	655.19
IPC(SET)	1,602,197.57	526,907.77	676.73

BORING LOCATION TABLE

Boring	Northing	Easting	Elevation (Feet)
V-1	1,600,947.0	523,864.1	636.0
V-2	1,599,381.1	523,453.6	636.4
V-3	1,597,268.3	522,789.7	636.0
V-4	1,599,385.6	525,014.6	645.9
V-5	1,599,662.8	526,325.4	647.6
V-6	1,597,855.6	526,911.6	645.0
V-7	1,597,691.2	527,181.6	601.5
V-8	1,597,661.0	525,832.6	646.1
CPT-1	1,601,668.3	524,646.2	636.6
CPT-2	1,601,097.5	523,912.5	636.7
CPT-3	1,600,179.8	523,743.4	636.0
CPT-4	1,599,361.5	523,455.3	636.4
CPT-5	1,597,807.1	523,159.9	642.4
CPT-6	1,597,127.6	522,770.4	647.3
CPT-7	1,597,948.6	527,145.3	638.2
CPT-8	1,597,477.5	526,046.5	646.5
CPT-9	1,597,756.7	527,123.4	604.3
STN-48	1,601,416.0	527,060.8	654.9
STN-62	1,596,738.1	523,522.1	635.8
STN-63	1,597,711.8	525,777.5	646.1
STN-64	1,597,643.0	525,718.1	638.1
STN-65	1,597,882.0	526,948.3	645.0
STN-66	1,597,739.2	527,065.2	606.3
STN-67	1,598,989.4	526,868.9	647.0
STN-68	1,599,083.7	526,995.0	611.4
STN-69	1,599,634.0	526,360.9	647.6
STN-70	1,599,720.3	526,482.8	613.3
STN-71	1,600,703.4	526,190.1	636.6
STN-72	1,601,431.1	525,775.3	636.6
STN-73	1,601,547.9	525,835.7	598.5
STN-74	1,601,909.1	524,916.3	635.7
STN-75	1,601,484.2	524,146.7	636.7
STN-76	1,601,519.8	524,124.6	624.5
STN-77	1,600,807.9	523,621.0	636.1
STN-78	1,599,698.1	523,281.1	636.8
STN-79	1,599,704.9	523,225.1	622.8
STN-80	1,598,668.7	523,078.9	637.2
STN-81	1,598,675.7	523,024.0	625.5
STN-82	1,597,723.5	522,758.9	636.9
STN-83	1,597,747.8	522,694.6	624.9
STN-84	1,596,588.0	523,031.0	639.3
STN-85	1,596,520.8	523,008.4	614.6
STN-86	1,596,516.5	523,011.9	617.6
STN-87	1,596,587.8	523,211.1	637.4
STN-88	1,596,511.6	523,232.4	618.5
STN-89	1,597,963.8	524,804.6	636.3
STN-90	1,597,204.1	525,851.5	613.2
STN-91	1,596,746.8	523,027.9	641.1
STN-92	1,596,741.8	522,972.4	640.1
STN-93	1,598,456.4	524,812.1	641.8
STN-94	1,598,399.5	524,769.6	639.0
STN-95	1,597,673.7	525,803.4	646.1
STN-96	1,597,293.4	526,298.6	647.6
STN-97	1,597,191.9	526,271.3	638.6
STN-98	1,597,091.5	526,250.2	602.5
STN-99	1,597,803.4	526,989.8	638.4
STN-100	1,598,328.0	527,385.4	644.1
STN-101	1,598,404.3	527,426.8	638.8
STN-102	1,599,036.5	526,931.3	638.7
STN-103	1,599,683.2	526,424.2	638.9
STN-104	1,600,046.7	525,695.6	645.9
STN-105	1,600,094.3	525,640.4	637.6
STN-106	1,599,340.1	524,994.7	645.9
STN-107	1,599,407.0	524,937.4	636.7
STN-108	1,597,658.1	527,138.2	601.5
STN-109	1,597,251.2	526,600.7	603.2
STN-110	1,597,979.8	527,321.7	606.5
STN-111	1,598,157.7	527,525.4	604.9
STN-112	1,597,157.5	526,457.8	604.1
STN-113	1,596,076.0	525,525.5	611.9
STN-114	1,598,137.9	524,932.3	637.1
STN-115	1,598,115.3	524,962.2	636.9
STN-117	1,598,107.2	524,906.3	636.8
STN-118	1,598,082.9	524,937.4	636.7

NOTES:

1. Topographic mapping was provided by Tennessee Valley Authority (TVA) on April 1, 2009. The flight date for the Main Ash Pond Complex is February 4, 2008. The hydrographic survey for the Stilling Pond is dated June 26, 2008 through July 7, 2008. The hydrographic survey for the Main Ash Pond is dated October, 2009. The hydrographic survey for the Pump Pond Discharge Channel is dated June 23, 2008.
2. The Baseline depicted hereon was recreated from Drawing No. 10W7420 titled "Limestone and Ash Disposal Area" and 10W7463-01 titled "Ash Disposal Area Units 7 and 8".
3. The Project Baseline shown hereon is for illustrative purposes only, and should not be used by the Owner or Contractor.

LEGEND

- V-1⊗ Vane Shear Test Location
- CPT-1 Cone Penetration Test
- SB-89 Soil Boring with Standard Penetration Tests and/or Undisturbed (Shelby) Tube Samples with Rock Core
- SB-71 Soil Boring with Standard Penetration Tests and/or Undisturbed (Shelby) Tube Samples
- CPT-3 Existing Cone Penetration Test (2004 and 2003)
- B-1 Existing Test Boring (2003)
- SS-7 Existing Test Boring (SS Only, TVA 1980)
- US-7 Existing Test Boring (SS & US, TVA 1980)
- V-1⊗ Existing Vane Shear Test Location (2003)
- TBM-1 Temporary Bench Mark (Panel Point Nail)
- TBM-2 Temporary Bench Mark (Panel Point Nail)
- TBM-3 Temporary Bench Mark (Top of PZ Well Cover, Stantec Gypsum Stack Boring STN-28, 2009)
- TBM-4 Temporary Bench Mark (Top of PZ Well Cover, Stantec Gypsum Stack, Boring STN-49, 2009)
- TBM-5 Temporary Bench Mark (Top of PZ Well Cover, (Stantec Gypsum Stack, Boring STN-39, 2009)
- IPC △ Iron Pin with Cap (Set)

FOR INFORMATION ONLY
This Record Drawing which has been previously submitted to TVA is provided for Information Only.

For Supporting Design Calculations see FPGWCFFESCDX00000020100004

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST
R 0	02/04/10	GKA	TJ	RDF	HRA	HRA	HRA	TJ	–	–

RECORD DRAWING

SCALE: 1"=200' EXCEPT AS NOTED

YARD

ASH POND GEOTECHNICAL EXPLORATION BORING LAYOUT

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
K. ANDERSON	T. JOHNSON	R. FULLER	H. APARICIO	H. APARICIO	H. APARICIO	T. JOHNSON

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000 | DATE 02/04/10 | 34 | C | 10W514-01 | R 0

Stantec Consulting Services Inc.
1409 N. Forbes Rd.
Lexington, Kentucky
40511-2050
Tel. 859.422.3000
Fax 859.422.3100
www.stantec.com

STANTEC | 0
TASK COMPLETED BY: | REV. NO.

PLOT FACTOR:XX
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

PLOT DATE: 05/04/2010 USER: JOHNSON, TRACY
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US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 12

Seepage Action Plan (SAP), Widows Creek Fossil Plant, Stevenson, Alabama, Stantec Consulting Services, June 25, 2010

Seepage Action Plan (SAP)

Widows Creek Fossil Plant
Stevenson, Alabama

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www.stantec.com

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

June 25, 2010

Seepage Action Plan (SAP)

Widows Creek Fossil Plant
Stevenson, Alabama

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

June 25, 2010

Seepage Action Plan (SAP)
Widows Creek Fossil Plant
Stevenson, Alabama

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Seepage Action Plan (SAP)
Widows Creek Fossil Plant
Stevenson, Alabama

1. Potential Seepage Areas

For readers not familiar with seepage through dams, refer to Appendix B, "Possible Seepage Problems and Recommendations" for more illustrative details. Seepage through an impoundment dam can typically be found on the lower third of the slope and extending beyond the toe approximately fifty feet. Figure 1 below displays the typical area on a cross section that should be reviewed during the seepage inspection for the Gypsum Stack and Ash Pond Complex. However, other seepage areas may exist, and the field inspector should be familiar with previous inspection reports and observations. Based on geotechnical analysis, plan views illustrating low factors of safety in terms of seepage have been prepared and are included in Appendix A. The areas identified, along with any other area previously identified during inspections, should be reviewed on a regular basis as identified in this document.

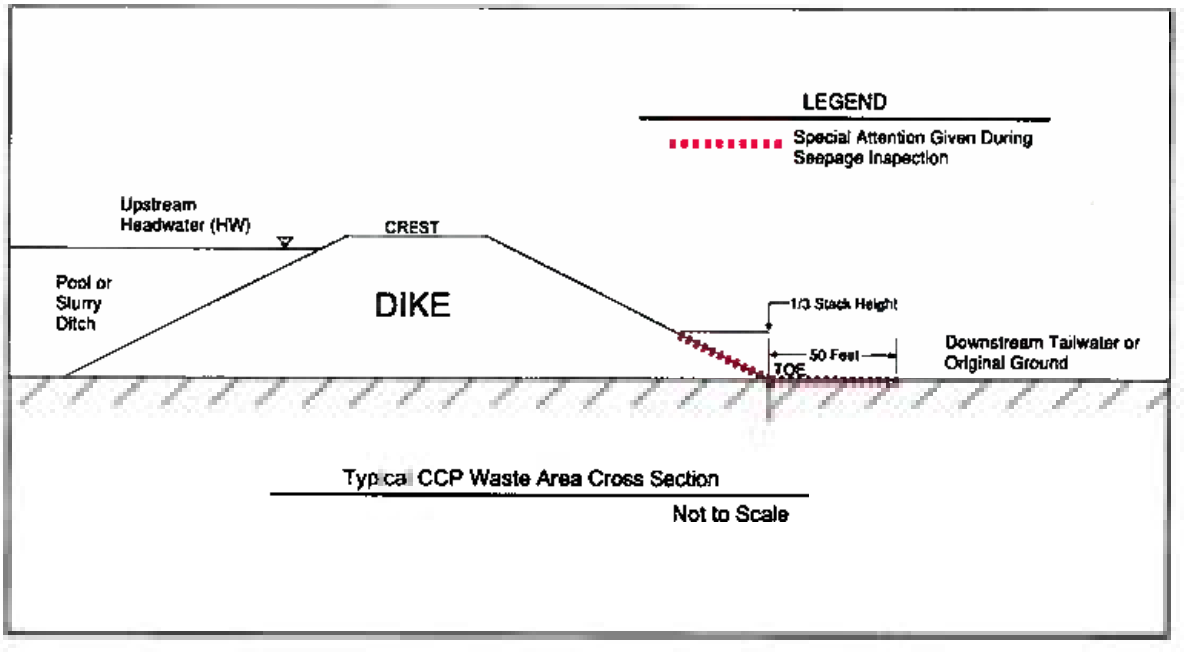


Figure 1. Seepage Inspection Location

2. Basic SAP Data

2.1. Purpose

The purpose of this SAP is to describe potential seepage action levels, and provide seepage short term management measures and actions in the event these action levels are observed.

2.2. Potential Impacted Area

Seepage related issues impact the integrity of earthen embankments. Seepage can lead to internal erosion of the embankment, known as piping, which has been the cause of many catastrophic failures in the past. Piping is a process where soil particles slowly carried out from inside the dam, eventually creating a tunnel or pipe. If the pipe forms all the way to the reservoir, the embankment will fail rapidly. Since the embankments at Widows Creek Fossil Plant serve as an impoundment for ash and gypsum slurry, it is imperative to maintain the embankments and prevent any possible failure from occurring. If a failure were to occur, the ash and gypsum slurry mixture could potentially contaminate Widows Creek and the Tennessee River.

2.3. Primary Responsibility and Frequency of Dike Safety Inspections

1. TVA RHO&M Field Supervisor for Widows Creek Fossil Plant (**Field Supervisor**)
2. TVA RHO&M East Region Construction Manager
3. TVA RHO&M Program Manager for Widows Creek Fossil Plant

Documented inspections should occur at a minimum of once per month. Additionally, there are two criteria which warrant an inspection. A documented inspection should occur following a significant precipitation event (0.5 inches of rain, 4 inches of snow), as well as following a change in the operation of the wet stack, pond, or other CCP wet waste area (switching between east/west ditch, switching ponds, raising pool elevations, etc.). A documented inspection involves inspecting the potential seepage areas noted on the plan views in Appendix A, paying particular attention to areas of concern previously identified. The **Seepage Log** should be updated to include new descriptions and photographs of any new areas of concern or changes to previously identified areas. Random inspections can occur on a more frequent basis if deemed necessary by the **Field Supervisor**.

3. Seepage Action Level Determination

For the purpose of this plan, three seepage action levels have been identified. The levels are based on potential risk associated with progressive erosion due to seepage and resulting breach of the embankment or impoundment.

Action Level 1 – Non-Flowing

- Wet areas
- Ponded Water

Action Level 2 – Flowing Seepage – No Erosion

- Non turbid (clear water) flow

Action Level 3 – Flowing Seepage – Active Erosion

- Turbid Flow
- Deposition of Sediment from Dike or Dam
- Boils (Ground Surface/ Underwater)
- Upstream Collapse or Sinkhole

3.1. Action Level 1 – Non Flowing

Seepage occurs in all earthen dams and dikes. The key is to properly collect and control seepage in a manner that does not cause damage to the embankment. Seepage that is not flowing but is evident by damp areas or ponded water does not generally represent an imminent threat to the embankment in terms of erosion (see Figure 2). However, if left unattended this seepage can lead to slope instabilities. Therefore, this should be noted so that it can be observed for changing conditions both at the downstream observation point and immediately upstream along the interior slopes.

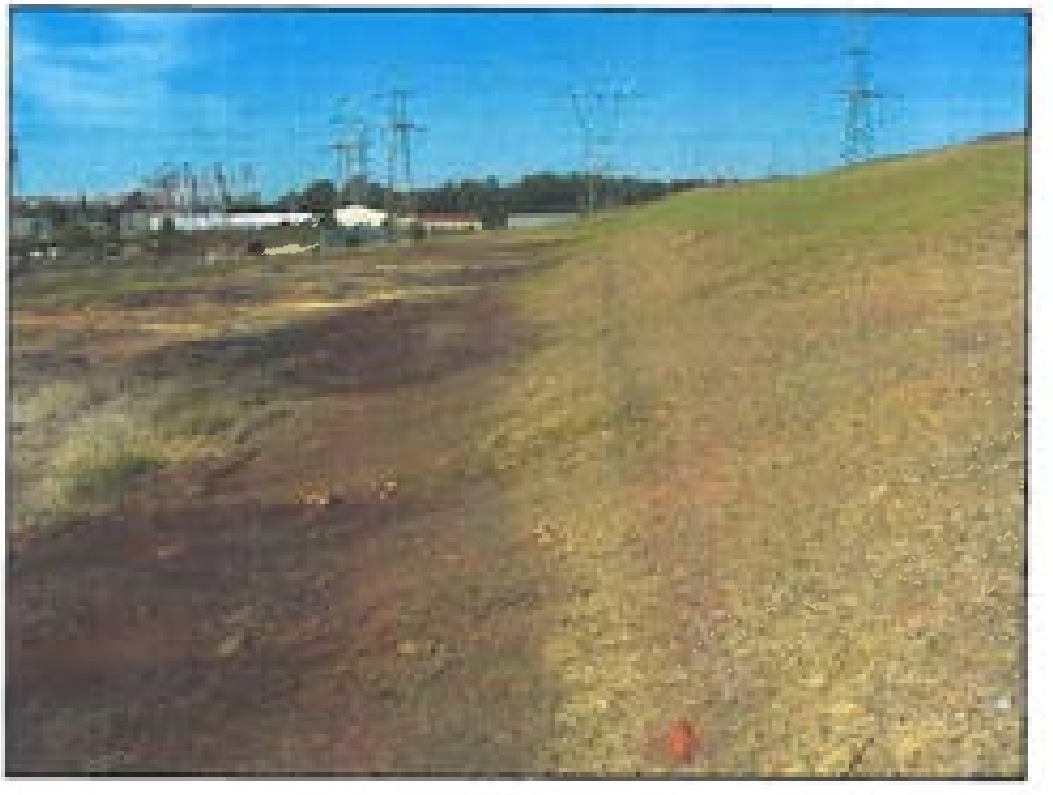


Figure 2. Example of Action Level 1 – Non-Flowing – Wet Area

3.2. Action Level 2 – Flowing Seepage – No Erosion

Action Level 2 involves observations of flowing seepage, but evidence of erosion is not noted. Evidence of erosion can be in the form of turbid (muddy water) flow, sediment deposition, obvious hole or soil "pipe". Evidence of erosion can be subtle and as a result, any flowing seepage should be carefully reviewed and monitored at least monthly. A picture

of flowing seepage water showing no evidence of erosion is depicted in Figure 3. Note that a seep does not need to be continuously turbid for a piping situation to be forming.



Figure 3. Example of Action Level 2 – Clear Flowing – Seepage Boil

3.3. Action Level 3 – Flowing Seepage – Active Erosion

Left unmitigated seepage demonstrating active erosion can lead to progressive failure of the embankment and catastrophic loss of the impoundment. Evidence of erosion can be in the form of turbid flow, sediment deposition, boil, obvious hole or soil "pipe". Evidence of erosion can be subtle and as a result, any flowing seepage should be carefully reviewed and monitored frequently. Careful attention should be given to seepage below water such as a stilling pond, creek or river (see Figure 6). This type of seepage is difficult to observe and determine if soil erosion is occurring. In moving water, evidence of seepage boils conveying embankment soil/ash materials will likely be (partially) washed away. Examples of active erosion are shown in Figures 4 thru 5.



Figure 4. Example of Action Level 3 – Turbid Flowing – Seepage Boil



Figure 5. Example of Action Level 3 – Deposition of Sediment from Dike



Figure 6. Example of Action Level 3 – Underwater Turbid Flowing – Seepage Boil

4. Intermediate Corrective Measures

For each action level a typical corrective measure is listed below.

4.1. Action Level 1 – Non Flowing

- **Field Supervisor** should document the seepage area into the **Seepage Log** (see below).
- All observers should pay particular attention to conduits through the embankments.
- **Field Supervisor** should record the date, time, size of area, location, and photographs in the **Seepage Log**.

The **Seepage Log** should be kept at the Shift Operation Supervisor's (SOS) office such that inspectors (TVA, geotechnical consultant, or others) can document event triggers (date, time, location, pool level, etc.) and the site conditions observed for each seepage event. The **Seepage Log** shall function as a "living document" and be part of an ongoing monitoring program (to be controlled by TVA). As the monitoring program progresses, the **Seepage Log** will allow inspectors to summarize the historical conditions observed and provide a baseline of events to compare with future readings.

4.2. Action Level 2 – Flowing Seepage – No Erosion

- **Field Supervisor** should carefully inspect the area for outflow quantity, any transported material, and take photographs.
- If the seepage involves a conduit penetration associated with a spillway pipeline, storm culvert, or underdrain pipeline, the observer(s) should carefully inspect the

area by probing and /or carefully shoveling to see if the cause can be determined, determine if embankment materials are being transported, evident by turbid or cloudy water, and determine quantity of flow.

- Contact team members in accordance with Figure 8.
- Send photographs to the RHO&M Regional Construction Manager and CCP Program Manager for distribution.
- Geotechnical consultant, with concurrence of the TVA Program Manager and CCP Engineering Manager, should determine a plan of action within four hours of notification
- **Field Supervisor** should record the date, time, size of area, location, and photographs in the **Seepage Log.**

4.3. Action Level 3 – Flowing Seepage – Active Erosion

- **Field Supervisor** should carefully inspect the area for outflow quantity and transported material.
- **Field Supervisor** should determine if piping has occurred and extent by observing locations of seepage exits, take photographs, and contact team members in accordance with Figure 9.
- Geotechnical consultant, TVA Program Manager, and CCP Engineering Manager should determine a plan of action within four hours of notification such as lowering the pool, constructing a reverse graded filter, or sand bagging
- A typical reverse graded filter will consist of the following:
  - One foot of Concrete Sand (ALDOT Concrete Sand)
  - One foot of TDOT No. 89 Stone (ALDOT No. 89 Stone)
  - Two feet of KYTC Class II Channel Lining (TDOT Machine Rip Rap Class A-3, ALDOT Class I Rip Rap)
  - Silt Fence as required by guidance provided in the Best Management Practices for Erosion Prevention and Sediment Control
- An example of sandbagging is provided in Figure 7.
- **Field Supervisor** should record the date, time, size of area, location, and photographs in the **Seepage Log**.



Figure 7. Sand Bag Treatment (Temporary)

5. Materials On-Site

In case an emergency situation is observed during the inspection of the potential seepage areas, it is necessary to have materials readily available on-site to correct the situation. Table 1 below lists the materials to be stockpiled on-site and the quantity of each material.

Table 1. Stockpile Material Quantities

Material	Tons	Cubic Yards
Concrete Sand	90	60
TDOT No. 89 Stone	90	60
Class II Channel Lining	180	120
Sandbags (filled)	300 (total)	NA
30" Diameter HDPE Pipe	100 feet	NA

The amount of materials to be stockpiled is based on a production rate of 60 cubic yards per hour for a 2.5 CY long reach excavator assuming a material unit weight of 110 PCF.

The materials should be stockpiled at the lay down yard near the Gypsum Stack and southwest of the Pump Pond. The following earthwork equipment and qualified operator(s) should be located to place the material in case of an emergency:

- Long Reach Excavator
- Dump Truck
- Compactor, Bulldozer, Bobcat, any other nearby equipment which aids in the emergency

6. The SAP Process

6.1. Step 1 – Dike Observation or Event Detection

This step describes the detection of an unusual observation or emergency event and provides information to assist the Widows Creek RHO&M **Field Supervisor** or appropriate personnel in determining the appropriate emergency level for the observation or event. These observations could be made by inspectors during routine inspections of the embankments, or by everyday personnel.

6.2. Step 2 – Emergency Level Determination

Following an unusual observation or emergency event detection, the **Field Supervisor** is responsible for classifying the event into one of the following three emergency levels:

6.2.1. Action Level 1 – Non Flowing

Observation is routine to other observations and a similar established plan of action for minor repair or continued observation will be required. If a Level 1 Emergency is identified, the following steps should be taken:

- Update maps and **Seepage Log**
- Inform WCF personnel if repairs are needed
- Determine if other work activities need to be made aware of observation.

6.2.2. Action Level 2 – Flowing – No Erosion

A change in condition or a condition that has not been previously identified and discussed with the geotechnical engineers. If a Level 2 Emergency is identified, the following steps should be taken:

- Inform individuals in accordance with the flowchart in Figure 8.
- Update map and **Seepage Log**
- Inform WCF personnel if repairs are needed
- Determine if other work activities need to be made aware of new conditions.

6.2.3. Action Level 3 – Flowing – Active Erosion

A change in condition that is drastic and could rapidly lead to failure of the embankment if not corrected. If a Level 3 Emergency is identified, the following steps should be taken:

- Inform plant SOS, who will initiate TVA plant-specific Emergency Action Plan (see Figure 9).
- Inform geotechnical consultant
- Develop safe plan of action for repair with geotechnical consultants
- Initiate repairs once plan has been approved by site safety and geotechnical consultant
- Update map and **Seepage Log**.

6.3. Step 3 – Notification and Communication

6.3.1. Notification

Following the determination of a possible seepage situation, it is necessary to notify the appropriate personnel discussed below for the required action to occur.

6.3.2. Communication

In case of an Action Level 2 emergency, the flowchart presented in Figure 8 should be followed to ensure the proper personnel are contacted. In an Action Level 3 emergency, the flowchart presented in Figure 9 should be followed.

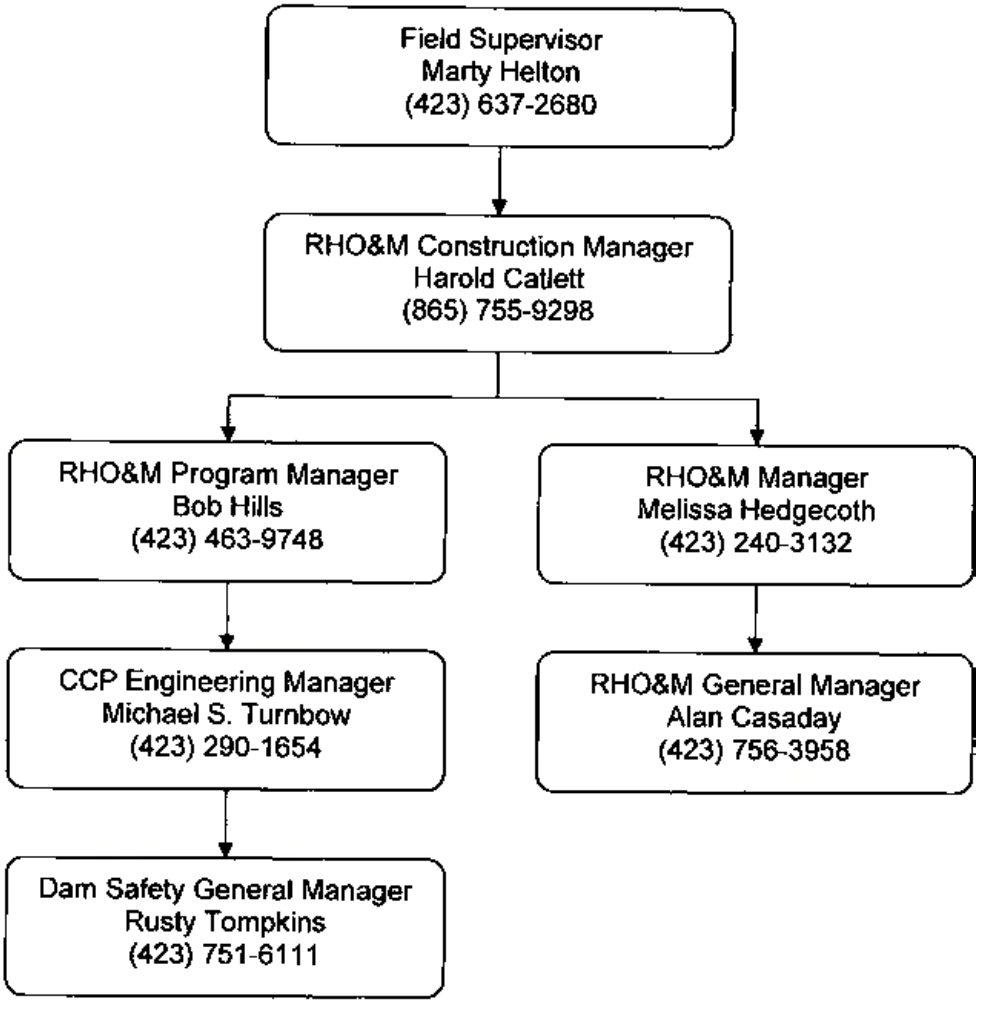


Figure 8. Level 2 Emergency Contact Flowchart

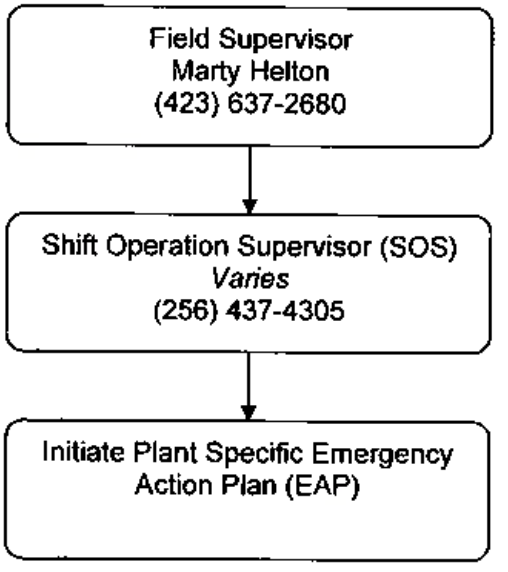
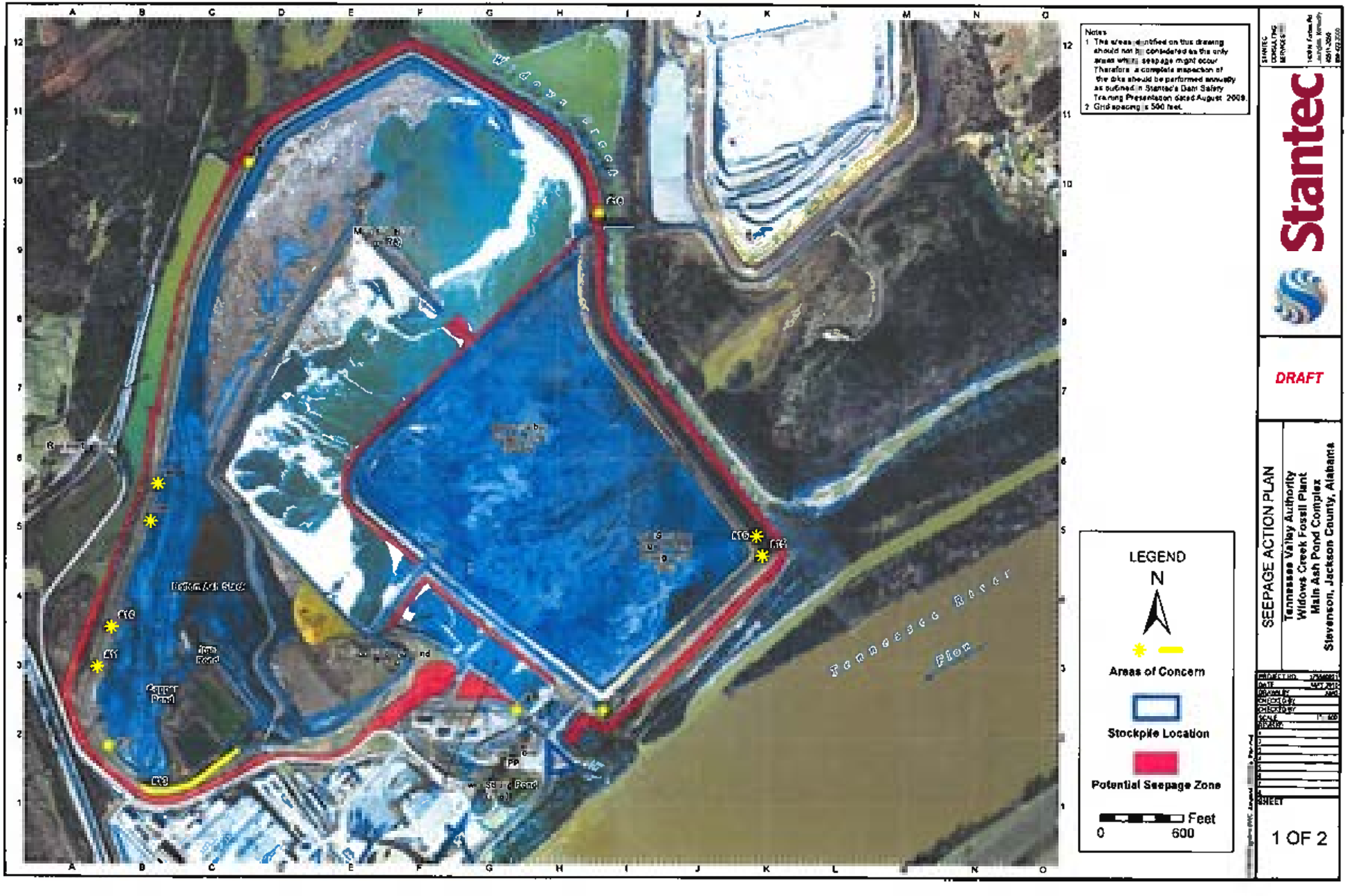


Figure 9. Level 3 Emergency Contact Flowchart

Appendix A

Gypsum Stack and Ash Pond Complex Site Plans


Notes
1. The areas identified on this drawing should not be considered as the only areas where seepage might occur. Therefore, a complete inspection of the dike should be performed annually as outlined in Stantec's Dam Safety Training Presentation dated August 2009.
2. Grid spacing is 500 feet.
LEGEND
N
Areas of Concern
Stockpile Location
Potential Seepage Zone
Feet
0
600
Stantec
DRAFT
SEEPAGE ACTION PLAN
Tennessee Valley Authority
Widows Creek Fossil Plant
Main Ash Pond Complex
Stevenson, Jackson County, Alabama
SHEET
1 OF 2



Notes
1. The areas identified on this drawing should not be considered as the only areas where seepage might occur. Therefore, a complete inspection of the dike should be performed annually as outlined in Stantec's Dam Safety Training Presentation dated August 2009.
2. Grid spacing is 500 feet.

LEGEND

N

Areas of Concern

Stockpile Location

Potential Seepage Zone

Feet
0 200 400

STANTEC CONSULTING SERVICES INC.
Stantec

DRAFT

SEEPAGE ACTION PLAN
Tennessee Valley Authority
Widows Creek Fossil Plant
Gypsum Stack
Stevenson, Jackson County, Alabama

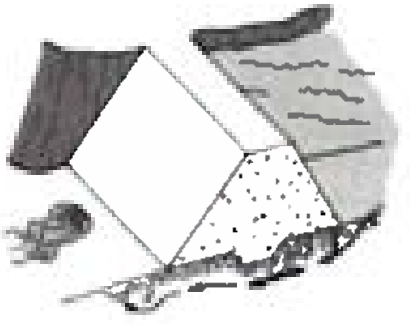
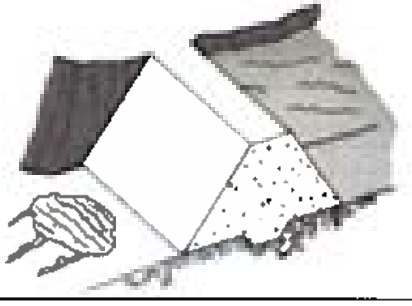
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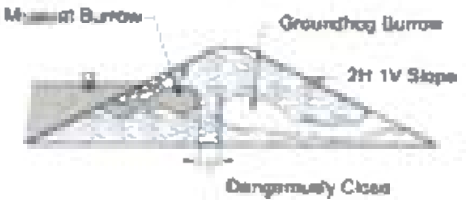
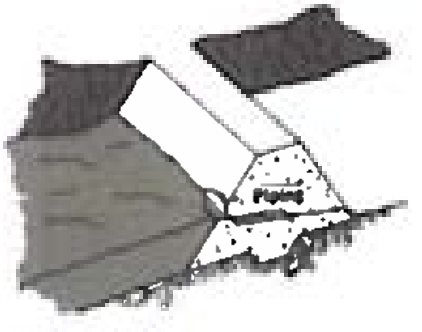
Appendix B

Possible Seepage Problems and Recommendations

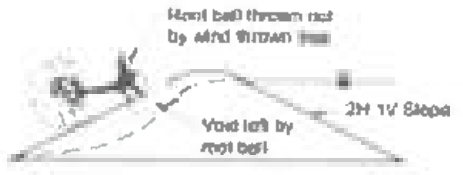
Appendix B – Possible Problems and Recommendations

Seepage Problem	Recommendations
Seepage Water Exiting at Abutment Contact 	Study leakage area to determine quantity of flow and extent of saturation. Stake out the saturated area and monitor for growth or shrinkage. Inspect frequently for slides. Water level in the impoundment may be lowered to increase embankment safety. A QUALIFIED ENGINEER should inspect the conditions and recommend further actions to be taken.
Seepage Water Exiting as a Boil in the Foundation 	Examine boil for transportation of foundation materials, evidenced by discoloration. If soil particles are moving downstream, create a sand bag or earth dike around the boil. This is a temporary control measure. The pressure created by the water level within the dike may control flow velocities and prevent further erosion. If erosion continues, lower the reservoir level. A QUALIFIED ENGINEER should inspect the condition and recommend further actions to be taken.
Spongy Condition at Toe of Dam 	Carefully inspect the area for outflow quantity and any transported material. A QUALIFIED ENGINEER should inspect the condition and recommend further actions to be taken.

Appendix B – Possible Problems and Recommendations

Seepage Problem	Recommendations
Rodent Activity 	Control rodents to prevent more damage. Determine exact location of digging and extent of tunneling. Remove rodents and backfill existing holes.
Seepage Water Exiting from a Point Adjacent to the Outlet 	Investigate the area by probing and/or carefully shoveling to see if the cause can be determined. Determine if leakage water is carrying soil particles evidenced by discoloration. Determine quantity of flow. If flow increases, or is carrying embankment materials, reservoir level should be lowered until leakage stops. A QUALIFIED ENGINEER should inspect the condition and recommend further actions to be taken.
Sinkhole 	Inspect other parts of the dam for seepage or more sinkholes. Identify exact cause of sinkholes. Check seepage and leakage outflows for dirty water. A QUALIFIED ENGINEER should inspect the conditions and recommend further actions to be taken.

Appendix B – Possible Problems and Recommendations

Seepage Problem	Recommendations
Trees and Brush 	Remove all trees and shrubs on and within 25 feet of the embankment. Properly backfill void with compacted material. A QUALIFIED ENGINEER may be required.

Source: Connecticut Department of Environmental Protection, Guidelines for Inspection and Maintenance of Dams, September 2001.

Appendix C

Seepage Log

WCF Seepage Log

Widows Creek Fossil Plant
Stevenson, Alabama
Updated 6/23/2010, Rev. 6

Area of Concern	Coordinate Location (Northing/Easting)		Date Initially Observed	Time	Approximate Size (Linear Feet)	SAP Level	Description	Mitigation Status/ Future Plans
1	Survey Requested		12/5/2007	AM	120	1	Seepage induced slough below the springline-Initially observed by Kelly Evans of TVA on 12/5/2007	Workplan #5 was completed in 08/2008 by TransAsh.No further action needed.
2	Survey Requested		1/10/2009	AM	100	1	Possible slough initially observed 1/10/2009 by Stantec.	Stantec installed slope inclinometers and monitored the readings. No movement was recorded to date. No further action is needed.
3	1603548.56	527808.9	01/2009	AM	150	1	Slough area was initially observed by Stantec in 01/2009.	It was apparent to Senior Geotechnical Engineers at Stantec that the slough area had recently been remediated. This area should be monitored for changes.
4	Survey Requested		2/17/2010	AM	50	1	Minor slough-Initially observed on 2/17/2010 by TransAsh.	Monitoring repair plan to be completed by URS. The area should be monitored in the future for changes.
5	1602625.66	528696.28	2/17/2010	AM	100	1	Minor head scarp-initially discovered on 2/17/2010 by TransAsh after heavy rain.	Monitoring repair plan to be completed by URS. The area should be monitored in the future for changes.
6	Survey Requested		1/8/2010	AM	50	1	Slough -initially observed by Marty Helton,Site Supervisor from TVA on 01/08/2010	The area was repaired soon after and should be monitored for future changes.
7	Survey Requested		1/26/2010	AM	75	1	Red water seep at the toe of red water dike-initially observed on 01/26/2010 by Marty Helton, Site Supervisor from TVA.	Stantec is in the process of completing Geotecchnical Exploration.The area should be monitored for changes until the Geotechnical Report is issued in August.
8	Survey Requested		2/17/2010	AM	200	1	Wet weather rutting on the access road-initially discovered by Stantec on 2/17/2010.	The wet area was observed to be dried up in Aprit-no further action is required.
9	1598472.65	523086.75	2/17/2010	AM	120	1	Slough in bottom ash stack-initially observed by Stantec on 2/17/2010.	The slough area should be monitored for future changes.
10	Survey Requested		2/17/2010	AM	70	1	Wet weather rutting on the access road-initially discovered by Stantec on 2/17/2010.	The wet area was observed to be dried up in April-no further action is required.

WCF Seepage Log

Widows Creek Fossil Plant
Stevenson, Alabama
Updated 6/23/2010, Rev. 6

Area of Concern	Coordinate Location (Northing/Easting)		Date Initially Observed	Time	Approximate Size (Linear Feet)	SAP Level	Description	Mitigation Status/ Future Plans
11	Survey Requested		2/17/2010	AM	50	1	Wet weather rutting on the access road-initially discovered by Stantec on 2/17/2010.	The wet area was observed to be dried up in April-no further action is required.
12	Survey Requested		2/17/2010	AM	50	1	Wet weather rutting on the access road-initially discovered by Stantec on 2/17/2010.	The wet area was observed to be dried up in April-no further action is required.
13	1596509.24	523234.86	1/10/2009	AM	1000	2	Red water seep in bottom ash area causig slough encroaching into plant road-initially observed 01/10/2009 by Stantec.	Workplan #7 issued for review- remediation procedures to be provided by URS. The area should be monitored for future changes.
14	Survey Requested		2/17/2010	AM	300	1	Standing water seepage below lower stilling pond dike-initially observed by Stantec on 2/17/2010.	Remediation plan to be provided by URS. The area should be monitored for future changes.
15	Survey Requested		01/2009	AM	250	1	Rutting & loss of vegetation on dredge cell - initially observed by Stantec in 01/2009.	No remediations necessary. The area should be monitored for future changes.
16	Survey Requested		01/2009	AM	300	1	Seepage through road subgrade killing vegetation on road slope-initially observed by Stantec in 01/2009	No remediations necessary. The area should be monitored for future changes.
17	1603564.25	527792.84	2/26/2010	8:00 AM	10	2	Noted seep/boil area from 2/26/2010 on the northwest slope of the gypsum stack- the seep is in the general location of the AOC 3, identified in January 2009.	A reverse graded filter was installed at the location of the boil/seep on 2/26/2010.
18	Survey Requested		6/3/2010	9:30 AM	130	2	Seep area was observed on 6/3/2010 by Brian Franks of TransAsh,on the northeast slope of Main Ash Pond A, on the upstream side of the "Million Dollar" bridge.	A reverse graded filter blanket was designed and installed on 6/63/2010 to remediate the seep area. Future observations should be completed.

WCF Seepage Log

Widows Creek Fossil Plant
Stevenson, Alabama
Updated 6/23/2010, Rev. 6

Area of Concern	Coordinate Location (Northing/Easting)	Date Initially Observed	Time	Approximate Size (Linear Feet)	SAP Level	Description	Mitigation Status/ Future Plans
[illegible]	Survey Requested	6/15/2010	N/A	N/A	1	Surficial disturbance caused by mowing on steeper slope section on SW corner of Dredge cell about 15' (vertical) above toe, where clay is moistened from seepage. Clayey embankment surface was firm, and I judge this as only surface disturbance caused by combination of slightly wet surface due to small amount of seepage, steep slope, and mower tractor scuffing the surface.	Suggest just smoothing the surface with the back of an excavator bucket (or similar equipment) to prevent water from being trapped in ruts. Continue to monitor and document changes
[illegible]	Survey Requested	6/15/2010	N/A	200	1	Wet weather rutting caused by surficial softening of clay/gypsum near East toe of Gypsum Stack. Seepage flow not evident, but soil is now disturbed enough to require repair via installation of inverted filter.	Surface already releveled using excavator bucket to prevent further pooling of water. Repair area identified via utility probing, and field flagged by URS, TVA and Trans Ash personnel. Area warranting filter installation approx. 35 feet up slope from toe, and about 200 feet along the slope.

Note: Initial Seepage Log was developed based on Stantec's understanding of known issues from Phase 1 and 2 assessments as well as ongoing construction work and knowledge of the site. No field visit was conducted to verify current seepage areas of concern.



Area of Concern 1

12/5/2007
Seepage induced slough below the spring line on the west side of the gypsum stack adjacent to the stilling pond. Remediation included Work Plan 5, which was completed by Trans Ash in August 2009.



Area of Concern 2
6/8/2010
Possible slough initially observed by Stantec. Slope inclinometers were installed by Stantec and monitored regularly. To date, no movement was recorded, and no further action is required.



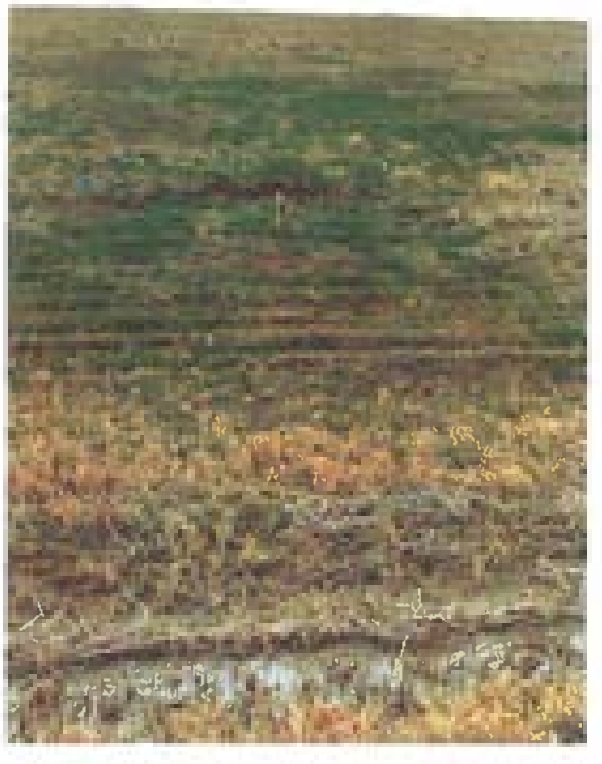
Area of Concern 3

January 2009
Apparent slough area was discovered by Stantec. The area had been previously remediated. Continued monitoring was recommended by Stantec as the course of action.



Area of Concern 4

2/17/2010
Minor slough area initially observed by Trans Ash. Monitoring and repair plan to be implemented by URS. Continued monitoring for changes should be performed.



Area of Concern 5

2/17/2010
Minor head scarp discovered by Trans Ash following a rain event. Monitoring and repair plan to be implemented by URS. Continued monitoring for changes should be performed.



Area of Concern 6

1/8/2010
Slough in the crushed stone embankment was initially observed by TVA. Repairs were immediately made, and area should be monitored regularly for changes.



Area of Concern 7

1/26/2010
Red water seep at the toe of the red water dike, observed by TVA. Stantec is currently performing a geotechnical investigation in the area. Continued monitoring should be performed.



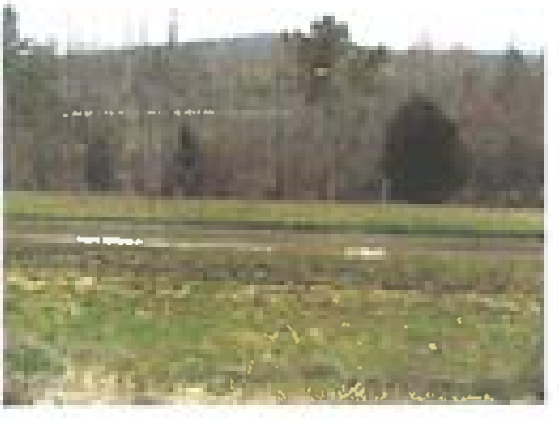
Area of Concern 8

2/17/2010
Wet weather rutting on the access road was observed by Stantec after a rain event. Upon further inspection in April, the area appeared to have been dried up, and no further action was required.



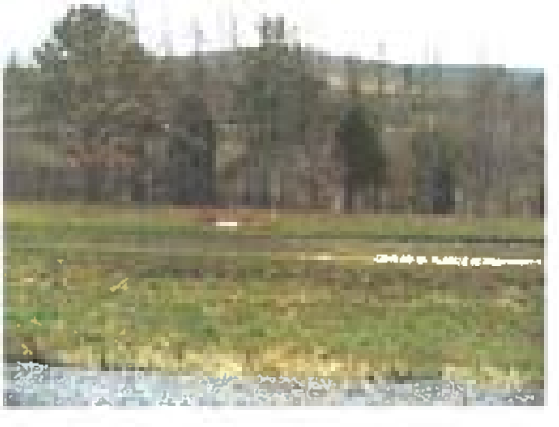
Area of Concern 9

2/17/2010
Slough area observed at the bottom of the ash stack by Stantec. It was determined that no immediate action was necessary and that the area should be monitored regularly for any changes..



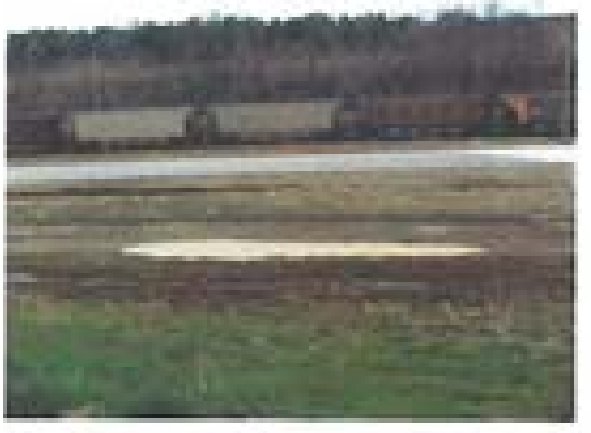
Area of Concern 10

2/17/2010
Wet weather rutting on the access road was observed by Stantec after a rain event. Upon further inspection in April, the area appeared to have been dried up, and no further action was required.



Area of Concern 11

2/17/2010
Wet weather rutting on the access road was observed by Stantec after a rain event. Upon further inspection in April, the area appeared to have been dried up, and no further action was required.



Area of Concern 12

2/17/2010
Wet weather rutting on the access road was observed by Stantec after a rain event. Upon further inspection in April, the area appeared to have been dried up, and no further action was required.



Area of Concern 13

1/15/2009
Red water seep in the bottom ash area that was causing a slough that encroached onto the plant road. First report of excessive seepage in vicinity dates back to 1985. Work plan 7 was proposed and submitted for review by TVA. The area should be monitored for any changes in the interim prior to URS implementing mitigation.



Area of Concern 14

2/17/2010
Standing water seep below the lower stilling pond dike, observed by Stantec. Remediation plan is to be provided by URS, and regular monitoring of the area should be performed.



Area of Concern 15

January 2010
Rutting at the toe of slope and a loss of vegetation on the dredge cell, observed by Stantec. Monitor the area for any changes.



Area of Concern 16

January 2010
Seepage through the road sub-grade resulting in the loss of vegetation down slope. Monitor area regularly for any changes.



Area of Concern 17

2/26/2010
Boil on the northwest slope of the Gypsum Stack, observed by Trans Ash. The boil is in the general location of the AOC 3, identified in January 2009. SAP level 2. (see page g)



Area of Concern 17

2/26/2010
Boil on the northwest slope of the Gypsum Stack.



Area of Concern 17

2/26/2010
Initial layer of sand placed over boil. Note the boil is actively displacing sand in the center of the picture.



Area of Concern 17

2/26/2010
Reverse graded filter constructed over the boil.



Area of Concern 18

6/3/2010
South facing view of seep area along the northeast slope of the main ash pond approximately 130 feet long on the upstream side of million dollar bridge. SAP level 2.



Area of Concern 18

6/3/2010
Seep on the northeast slope of the main ash pond.



Area of Concern 18

6/3/2010
Seep and standing water on the northeast slope of the main ash pond.



Area of Concern 18

6/3/2010
Close-up view of the saturated surface on the northeast slope of the main ash pond.



Area of Concern 18

6/8/2010
Initial excavation of the soft seepage area on the northeast slope of the main ash pond in preparation for the reverse graded filter blanket.



Area of Concern 18

6/8/2010
Placement of stone base and sand begins in the affected area.



Area of Concern 18

6/8/2010
Nearly completed reverse graded filter blanket over the affected seepage area.



Area of Concern 19

6/15/2010
Surficial disturbance caused by mowing on steeper slope section on SW corner of Dredge cell about 15' (vertical) above toe, where clay is moistened from seepage. Clayey embankment surface was firm, and I judge this as only surface disturbance caused by combination of slightly wet surface due to small amount of seepage, steep slope, and mower tractor scuffing the surface.



Area of Concern 19

6/15/2010
Surficial disturbance caused by mowing on steeper slope section on SW corner of Dredge cell about 15' (vertical) above toe, where clay is moistened from seepage. Clayey embankment surface was firm, and I judge this as only surface disturbance caused by combination of slightly wet surface due to small amount of seepage, steep slope, and mower tractor scuffing the surface.



Area of Concern 19

6/15/2010
Surficial disturbance caused by mowing on steeper slope section on SW corner of Dredge cell about 15' (vertical) above toe, where clay is moistened from seepage. Clayey embankment surface was firm, and I judge this as only surface disturbance caused by combination of slightly wet surface due to small amount of seepage, steep slope, and mower tractor scuffing the surface.



Area of Concern 20

6/15/2010
Wet weather rutting caused by surficial softening of clay/gypsum near East toe of Gypsum Stack. Seepage flow not evident, but soil is now disturbed enough to require repair via installation of inverted filter.



Area of Concern 20

6/15/2010
Wet weather rutting caused by surficial softening of clay/gypsum near East toe of Gypsum Stack. Seepage flow not evident, but soil is now disturbed enough to require repair via installation of inverted filter.



Area of Concern 20

6/15/2010
Wet weather rutting caused by surficial softening of clay/gypsum near East toe of Gypsum Stack. Seepage flow not evident, but soil is now disturbed enough to require repair via installation of inverted filter.



Area of Concern 20

6/15/2010
Wet weather rutting caused by surficial softening of clay/gypsum near East toe of Gypsum Stack. Seepage flow not evident, but soil is now disturbed enough to require repair via installation of inverted filter.

Appendix D

WCF CCP Emergency Action Plan

US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 13

Tennessee Valley Authority (TVA) Routine Handling Operations and Maintenance (RHO&M) Operating Support Document, Widows Creek Fossil Plant, July 2011

Tennessee Valley Authority (TVA)

Routine Handling Operations and Maintenance (RHO&M)

Operations Support Document



Widows Creek Fossil Plant
July 2011

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Notes: Each section to be updated as required by RHO&M.

(1) **Red indicates annual updates required**

(2) **Blue indicates the contract term is currently 2 years**

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Notes: Each section to be updated as required by RHO&M.

(1) **Red indicates annual updates required**

(2) **Blue indicates the contract term is currently 2 years**

1.0 OVERVIEW OF OPERATIONS SUPPORT DOCUMENT

1.1 TABLE OF CONTENTS

The table of contents is presented in **Section 100**.

1.2 OVERVIEW

The overview of the OSD is presented herein as **Section 200**.

1.2.1 OVERVIEW AND HISTORY

The Widows Creek Steam Plant is the third steam-electric project built by TVA. The first, Watts Bar, was built as a part of TVA's first emergency program of the World War II period and began commercial operation in February 1942. The second, Johnsonville, began a series of seven large steam-electric generating plants constructed between 1949 and 1959, a building program which resulted principally from increased power demands of the Atomic Energy Commission, Arnold Engineering Development Center, and other Federal defense agencies in the region. The Widows Creek plant was originally planned as a necessary system addition to reduce the transmission of power from the Eastern area of the TVA region to the Chattanooga area by supplying the anticipated demands of the Chattanooga area.

The first Widows Creek unit was placed in commercial operation July 1, 1952. Five units with 125,000 kw capability each were added between that date and July 17, 1954. Two additional units, numbers 7 and 8, each having a capacity and capability of 500,000 kw, were authorized for the Widows Creek Steam Plant November 15, 1957, and October 6, 1960, respectively. Construction of unit 7 began March 3, 1958, and commercial operation was attained February 1, 1961. Unit 8 construction started October 12, 1960, and was placed in commercial operation February 7, 1965.

With the addition of units 7 and 8, Widows Creek Steam Plant became the largest generating plant in the TVA system--a distinction held until unit 3 was added to Paradise Steam Plant in February 1970.

1.2.2 OVERVIEW ASH POND COMPLEX CCP OPERATIONS

Eight structures are defined within the Ash Pond Complex[1];

1. Dredge Cell, 116 acres,
2. Main Ash Pond (consisting of Clarification Ponds 1-3), 156 acres,
3. Upper/Lower Stilling Pond, 4 acres each,
4. Pump Pond, 0.25 acre,
5. Red Water Pond, 17 acres,
6. Bottom Ash Stack, 32 acres.

1.2.2.1 DREDGE CELL

The elevated Dredge Cell has a surface area of approximately 116 acres and is located south of the Main Ash Pond as shown in **Figure 1**. Once TVA added a forced oxidation gypsum scrubber in the mid-1980's, a new gypsum stack was created. After the Gypsum Stack was placed in service beginning in 1986, the dredge cell received dredged material from the Main Ash Pond until 2009[1]. The Dredge Cell remained inactive and was allowed to dewater.

In July 2010 TVA began hydraulically dredging Fly Ash from Clarification Ponds 1 and 2, into the Bottom Ash Stack where it is dried. The dried fly ash is then deposited in the Dredge Cell where it is being placed and graded to facilitate future closure. A closure plan for the Dredge Cell is currently in development.

1.2.2.2 MAIN ASH POND

The Main Ash Pond is northwest of the Dredge Cell and is shown in **Figure 1**. It has a surface area of 156 acres and it receives both fly ash and bottom ash.[1] The fly ash and bottom ash from Units 1-6 are sluiced to one slurry ditch, while the bottom ash from Units 7 and 8 are sluiced to a separate slurry ditch.

According to the current Long Term Disposal Plan (see **Section 1402**) the volume of both ash types, fly and bottom ash, sluiced to the Main Ash Pond is 107,300 cubic yards.

The outfall from the sluice pipe lines for Units 1-6 enter the Bottom Ash Stack just west of the Copper Pond; and the outfall from the sluice pipe lines for Units 7 and 8 enter the Bottom Ash Stack just east of the Iron Pond.

1.2.2.3 UPPER/LOWER STILLING POND

The Upper/Lower Stilling Pond, constructed in 1986, receives discharge from the Main Ash Pond (shown in **Figure 1**) and occupy an area of approximately 4 acres each.[1] The stilling pond receives effluent discharge through five morning glory type spillways in the Main Ash Pond. The northernmost, or Upper Stilling Pond, and Lower Stilling Pond both have a normal operating pool elevation of 611.5 with top and toe of dike at approximately elevation 614.0 and 596.0, respectively[1]. The Upper Stilling Pond and Lower Stilling Pond are hydraulically connected but partially separated by an internal deflector dike approximately 100 feet in length, and 15 to 20 feet in height[1]. Water flows from north to south through the ponds, eventually discharging to the Pump Pond through a series of five spillways.

1.2.2.4 PUMP POND

The Pump Pond (See **Figure 1**) is located southeast of, and adjacent to, the Lower Stilling Pond and has a surface area of approximately 0.25 acres.[1]

The pump pond contains two recirculation pumps that recycle water for use in the Condensate Cooler Water Intake and to the wet gypsum system. As the water level rises above the level of the three overflow spillways in the pond, it can overflow and discharge to the Tennessee River through the NPDES permitted outfall 001.

1.2.2.5 REDWATER POND

The Red Water Pond (see **Figure 1**) serves as a surface water run-off collection pond for the Main Ash Pond and was constructed in 2007. The water surface area measures approximately 17 acres, with the lowest confining dike top elevation of 609 feet.

1.2.2.6 BOTTOM ASH STACK

The Bottom Ash Stack (see **Figure 1**) is located southwest of the Main Ash Pond. It was formerly known as Ash Disposal Area Units 7 and 8. It is comprised of primarily bottom ash deposited by routine wet handling methods from two slurry ditches which traverse the Bottom Ash Stack in a north to south direction. The Bottom Ash Stack has a surface area of approximately 44 acres and a top elevation of approximately 646.0.[1] The toe of the stack is at approximately an elevation of 610.0 with 3H:1V sideslopes.[1]

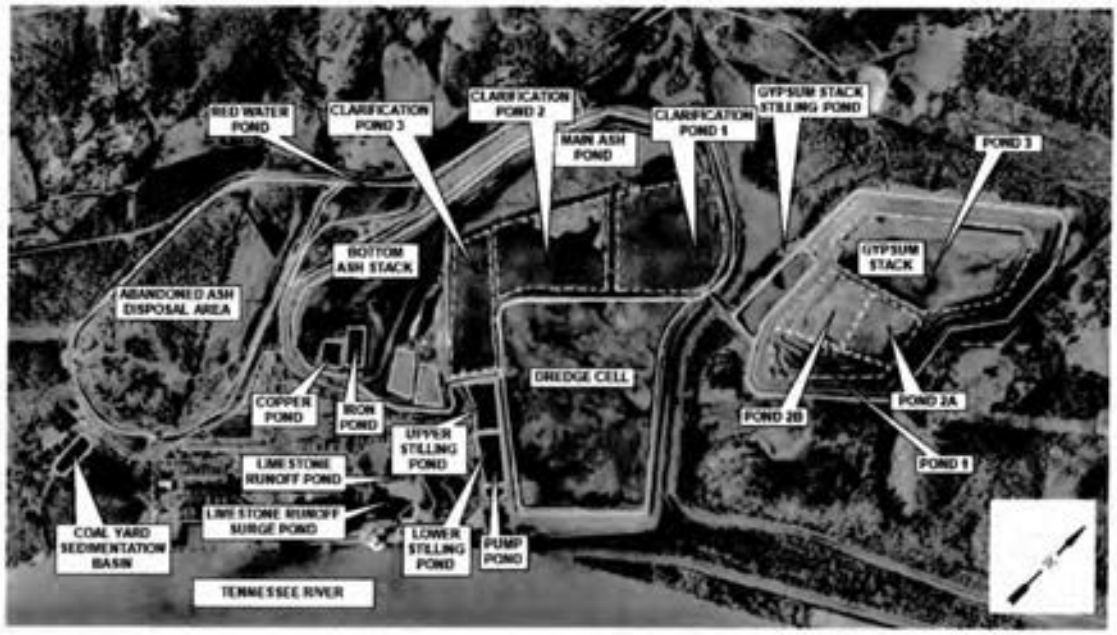


Figure 1 – WCF Plant Aerial Image

1.2.3 GYPSUM STACK

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.[2] The site is accessed from the plant over a one-lane concrete bridge spanning Widows Creek. 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. Since then, the facility has been used for disposing gypsum and fly ash from Units 7 and 8. During the first phase (Phase 1, see **Figure 2**), gypsum was sluiced into Clarification Pond 1.[2] 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, see **Figure 2**) area and converted to wet stacking method which involves mechanically stacking CCP's on top of hydraulically placed gypsum to build perimeter dikes.[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.

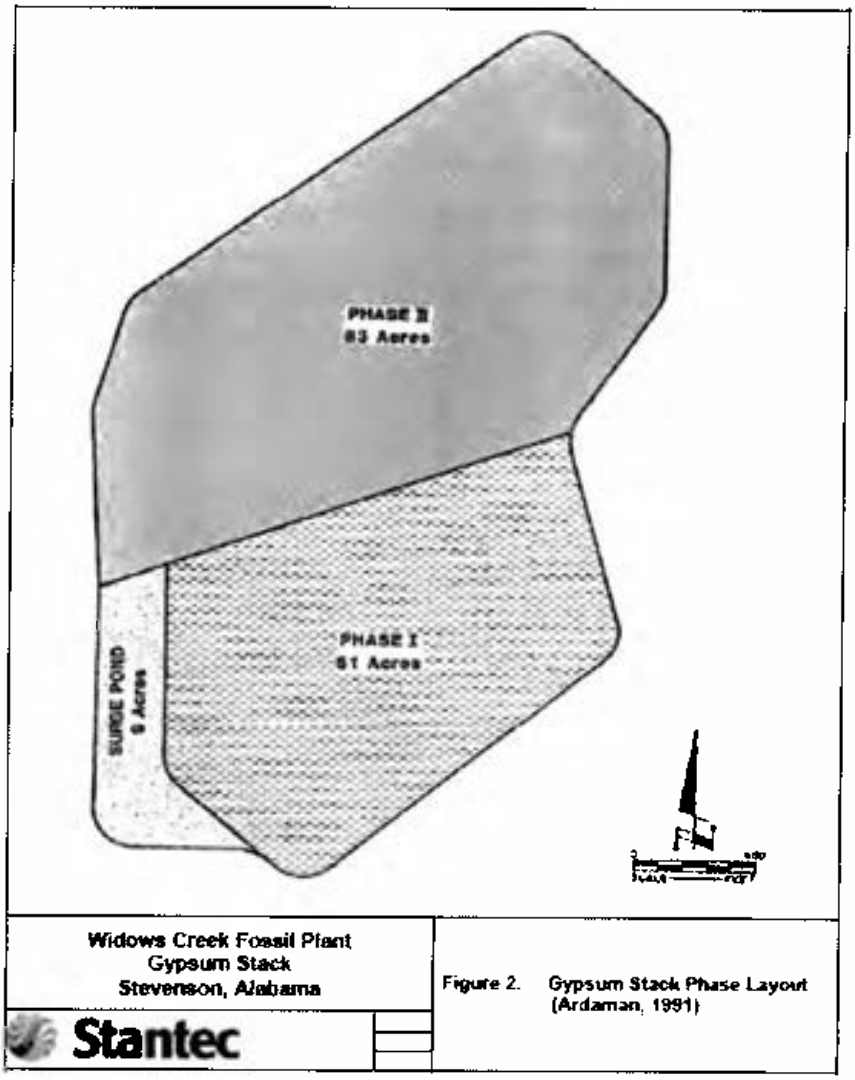


Figure 2 – Gypsum Stack Phase Layout

According to the current Long Term Disposal Plan (see **Section 1402**) the volume of both ash and gypsum sluiced to the Gypsum Stack is 309,600 cubic yards.

Sluiced ash and gypsum travels 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 generally 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 Gypsum Stack Stilling Pond.

1.2.4 REFERENCES

1. Stantec Consulting Services, Inc., Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Stantec Report 003 175569036, February 4, 2010
2. Stantec Consulting Services, Inc., Report of Phase 2 Geotechnical Exploration, Gypsum Stack Widows Creek Fossil Plant, Stevenson, AL, Stantec Report 003 175569039, February 5, 2010

1.3 ROLES AND RESPONSIBILITIES

Roles and responsibilities, **Section 300**, contains a narrative of the responsibilities for the daily operations for routine handling of CCP material. Two responsibility matrices are provided to better define all responsibilities for the routine handling group and TVA.

1.4 CONTACTS

Contacts, **Section 400**, defines contact information for RHO&M group personnel, the facility engineer, client project contacts, emergency service contacts and local hospital information.

The latest RHO&M contact list, facility engineer and client project contacts should be included in this document. This contact list should be updated as personnel change or positions are added. The contacts list should include any third party engineers.

The emergency contact list is provided as a general overview of emergency services. This emergency contact list is not intended to replace any site specific emergency action plans or emergency procedures such as the IPP Plan, Emergency Action Plan or Seepage Action Plan; it only should serve as supplemental information. Contact the site specific RHO&M or CCP personnel for additional emergency plans and procedures. In addition, phone numbers, a map and directions to the local hospital are included.

1.5 HISTORICAL INFORMATION

The historical information, **Section 500**, contains the information pertaining to the history of the facility to assist the operators of the facility during inspections and construction. The following subsections are included: historical timeline, pipe inventory, instrumentation plan, seepage log and FEMA Flood Insurance Rate Maps.

1.5.1 HISTORICAL TIMELINE

The historical timeline included in **Section 501** provides an overview of the history of the CCP facilities at WCF. The timeline provides historical aerial photographs and a description of the major construction events. It also notes any major failures that may have occurred.

1.5.2 PIPE INVENTORY

The pipe inventory included in **Section 502** was prepared by Stantec Consulting Services, Inc (Stantec) in 2010 and updated in June, 2011. The inventory contains a

plan view showing the locations of all major spillways and pipes at the facility. Photos of each spillway (when available) are also included. The inventory should be updated as new pipes/spillways are installed and old pipes are removed/abandoned. Contact CCP Engineering for the latest inventory.

1.5.3 INSTRUMENTATION PLAN

The instrumentation plan included in **Section 503** is a plan view depicting the locations of piezometers and slope inclinometers installed at the active facility during the geotechnical investigations performed by Stantec since 2009. The instrumentation plan should be updated as new instruments are installed and old instruments are removed. Contact CCP Engineering for the latest instrumentation plan.

1.5.4 SEEPAGE LOG

The seepage log included in **Section 504** contains an executive summary of the Seepage Action Plan, followed by the latest seepage log for WCF to be used as an example. The seepage log, in the WCF seepage action plan, is kept electronically at:

\\chapgfs2\ccp\Seepage_Action_Plans\WCF.

The seepage log should be updated as new seeps are identified and/or repaired. Contact CCP Engineering for the latest seepage log.

1.5.5 FEMA FLOOD MAPS

Included in **Section 505** are FEMA Flood Insurance Rate Maps (FIRM) produced under the National Flood Insurance Program. The maps show areas subject to inundation by the 1% annual chance flood for the WCF plant area.

1.6 CONTRACT

The current scope of work for the handling of coal combustion products at WCF is included in **Section 600**. The scope involves the handling of 450,000 cubic yards of fly ash and bottom ash, consisting of 360,000 yd^3 fly ash, 90,000 yd^3 bottom ash, annually. The scope also includes the handling of 700,000 dry tons of sluiced production gypsum annually.

The current scope of work should be included in the document. The routine handling contracts are typically five years in length. The contract section should be updated with the issuance of a new contract.

1.7 CONSTRUCTION PLANS

The construction plans are presented in **Section 700**. The construction plans section is intended to keep 11"x17" copies of the current drawings RHO&M utilizes for active placement of CCP materials and any 'issued for construction' status work plan drawings. In cases where construction drawings have not been developed, the permit drawings should be included, as this is the only information the contractor has for the handling and placement of CCP materials. Only the current drawings for the placement of CCP materials are included in **Section 700**. Remedial measures, unapproved drawings, permit or construction and 'issued for review' status work plan drawings are

not included in this section. Refer to **Section 800** for a list of other drawings for the active facilities.

1.8 DRAWING INVENTORY LIST

The drawing inventory list is presented in **Section 800**. The purpose of the drawing inventory list is to maintain a record of pertinent drawings for the active disposal facility at the plant and any 'completed' status work plans drawings. These drawings should include the original design drawings for the initial construction of the disposal facility as well as any additional designs for changes to the disposal area, remedial measures, or additional work plans that do not directly pertain to the placement of CCP materials.

The list of design drawings is included in this section. As new drawings are developed, revised, or construction is completed the list should be updated. The list includes the TVA series number, number of sheets, revision number, date issued, facility, and a description of the drawing set.

1.9 PERMITS

The Operations Requirement Matrix (see **Section 901**) included in **Section 900** lists the various requirements that pertain to routine handling of CCP materials for the active sites. The matrix is divided based on the source of the requirement. The requirements include permit requirements, NEPA commitments, and recommended practices.

The permit section also contains a list of the active permits at the facilities. A current list of facility permits can be found at:

http://insidenet.tva.gov/org/et/environet/toolbox/tva-facility_permit_list.xls.

Additionally contact the responsible Environmental Media Specialist to obtain a copy of the current active permits. The permit list may or may not be up-to-date as new permits, modifications or renewals may occur prior to incorporating revisions to the list. This section should be updated as updates to it occur.

1.10 SURVEY DATA

The survey data is presented in **Section 1000**. TVA routinely develops topographic survey information for WCF to calculate volume capacity and hydrographic survey information. RHO&M maintains a list of when the most current survey was conducted and when the next planned survey is scheduled for each plant. This section should be updated as updates to the survey schedule occur.

1.11 PROCEDURES

The current procedures for the handling of production ash and gypsum are included in **Section 1100**. As operations change, the procedures document should be updated.

The procedures section includes subsections for operations manuals, specifications, QA/QC plans, standard repair guidelines, daily field report, weekly material summary, weekly facility observation form, monthly/quarterly/special facility observation form, project start up checklist, additional work/change order form, TVA internal procedures for NEPA evaluations, RHO&M work control and clay dike restrictions. The subsections are described in the procedures document.

US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 14

Calculation Package and Basis of Design Report, Slope Stability Improvements, Ash Pond Complex, Jackson County, AL, March 2012, URS Corp.

US EPA ARCHIVE DOCUMENT

Calculation Package and Basis of Design Report

Slope Stability Improvements
IRRP-URSAC002 & IRRP-URSAC003
TVA Project 204372

Widows Creek Fossil Plant
Jackson County, Alabama

MARCH 2012
Rev 0

1375 Euclid Ave., Suite 600
Cleveland, OH 44115

1.0 Objectives & Methodology 1

2.0 Material Properties and Input Parameters 3

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3.2 Seepage Analysis 11

3.3 Global Stability Analysis 11

4.0 Pipe Crushing Calculations 13

4.1 30-in Gypsum Stack Decant Pipe 13

4.2 Sluice Pipe Bridge Crossing 13

US EPA ARCHIVE DOCUMENT

WCF	Widows Creek Facility
IRRP	Interim Risk Reduction Project
AC	Ash Complex
BOD	Basis of Design
FS	Factor of Safety
WP	Work Plan
IFC	Issue for Construction
USACE	US Army Corps of Engineers

US EPA ARCHIVE DOCUMENT

1.0 OBJECTIVES & METHODOLOGY

The overall goal of the Ash Pond Complex Interim Risk Reduction projects (AC-IRRPs) 2 and 3 is identical to that of AC-IRRP 1[1]; to mitigate immediate perimeter dike stability concerns and maintain or improve quality of discharged water while progressing towards the long-term closure of the AC impoundments. The potential for dike failure presents the possibility of an uncontrolled release of coal ash into the environment and inundation of the nearby Widows Creek and/or Tennessee River, possibly leading to severe environmental impacts. AC-IRRPs 2 and 3 consist of rock toe buttresses along the northwest and northeast sides of the Dredge Cell (DC) at the Widows Creek Facility (WCF), see **Figure 1**. This document describes the basis of design (BOD) for the proposed rock buttresses to support interim risk reduction and eventual closure of the Main Ash Pond and includes all supporting calculations. The final issue for construction (IFC) drawings (Work Plan WCF-10W519-WP-29) can be found in **Appendix A**.

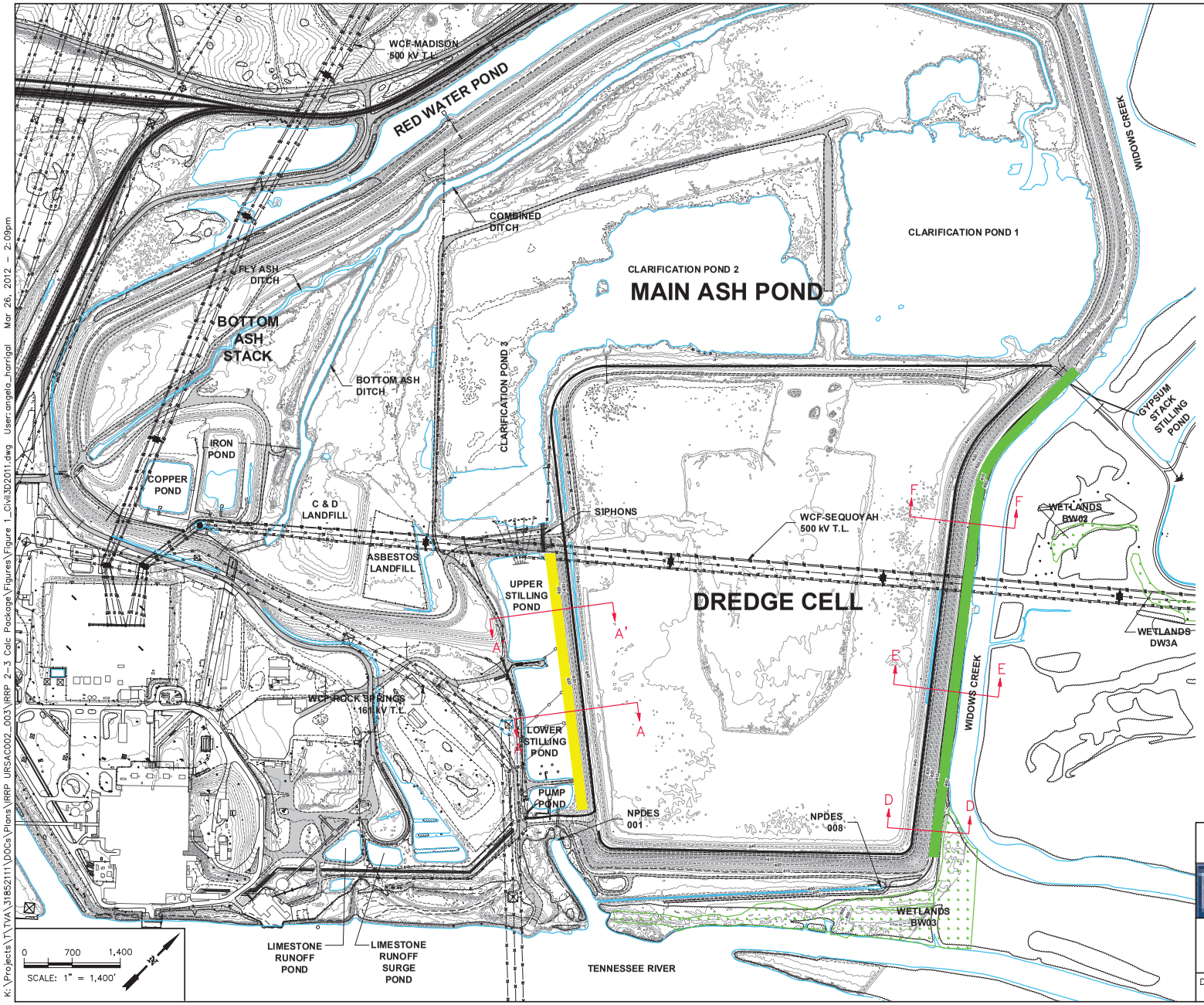
The objective of this project is to improve the slope stability factor of safety (FS) for the perimeter clay dike to 1.5 or better, as required by the Master Programmatic Document, Version 1. Several alternatives were considered to accomplish this objective, some of which included stone columns, deep soil mixing, French drains, and a deep groundwater collection trench system. Stability buttresses were recommended as the best alternative based on cost, long term maintenance, and constructability.

Stability analysis of AC-IRRPs 2 and 3 was completed using GeoStudio 2007 SLOPE/W and SEEP/W Version 7.17, Build 4921. The models were initially inherited from Stantec Inc., developed as part of the Report of Phase 2 Geotechnical Exploration[3], and subsequently enhanced by URS with updated geometry, material properties, and boundary conditions to complete the buttress design.

[1] Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372, Work Plan WCF-100521-WP-24
[2] Slope Improvements IRRP-URSAC002 & IRRP-URSAC003, TVA Project No. 204372, Work Plan WCF-100519-WP-29
[3] Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc.

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K:\Projects\TVA\31852111\DOCs\Plans\IRRP URSAC002_003\IRRP 2-3 Calc Package\Figures\Figure 1_Civil3D2011.dwg User:angela_harrigal Mar 26, 2012 - 2:09pm

URS

WIDOWS CREEK FOSSIL PLANT
STEVENSON, ALABAMA

OVERALL SITE PLAN
IRRP-URSAC002 & IRRP-URSAC003

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	FIGURE No:
AGH	JJH	31852111	3/15/12	1

2.0 MATERIAL PROPERTIES AND INPUT PARAMETERS

Material properties obtained from laboratory data from Stantec's Report of Phase 2 Geotechnical Exploration[3] were reviewed, and the values for angles of internal friction (Phi, '), cohesion (c), and unit weights (Gamma,) were updated by URS.

The material properties for Fill Clay with Gravel, Residual Fat Clay, and Silt (listed in **Table 2**) were updated from those presented in Stantec's Phase II report[3], as well as new data obtained from additional testing completed in the summer of 2010. New effective and total stress envelopes were developed and new Phi, and c values calculated. A $^2/_3$ failure envelope of the plotted values was used to determine Phi and c, see **Figures 2, 3, and 4** and the associated calculations. The remaining material properties, for Sand with Gravel and Silt with Sand were carried over from Stantec's Analysis[3]. The material properties for the stability buttress were developed using engineering judgment and typical values for similar material found in Alabama. The values used for slope stability analyses are summarized below in **Table 1**.

Table 1 Material Properties for Slope Stability Analyses

Material	Angle of Internal Friction, Drained, ' (deg.)	Cohesion, c (psf)	Unit Weight, (pcf)
Sand with Gravel (Bottom Ash)	33.0	0	119
Silt with Sand (Bottom Ash)	33.0	0	112
Fill Clay with Gravel (Embankment Fill)	29.0	150	125
Residual Fat Clay	33.0	50	125
Silt (Fly Ash)	27.5	0	112
Stability Buttress	40.0	0	115

Hydraulic Conductivity parameters were also initially obtained from Stantec's Phase II Report[3]. The models were calibrated with parameters to match existing, measured piezometric surfaces, and laboratory testing data from samples taken in the field[3]. The pore pressures from SEEP/W were "parented" into SLOPE/W, limit-equilibrium slope stability software, to calculate slope stability factors of safety. These values were subsequently modified based on engineering judgment and a parametric study conducted by URS. **Table 2** shows the final material properties used for the SEEP/W analysis. The Seepage Analysis Reports for each critical cross section are in **Appendix B**. The phreatic surface was obtained based on the SEEP/W finite-element seepage modeling software.

[1] Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372, Work Plan WCF-100521-WP-24
[2] Slope Improvements IRRP-URSAC002 & IRRP-URSAC003, TVA Project No. 204372, Work Plan WCF-100519-WP-29
[3] Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc.

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Table 2 Material Properties for Seepage Analysis

	Soil-Water Retention Curve (SWRC)			Unsaturated Hydraulic Conductivity Function				
Material	SWRC Sample Function	Saturated VWC (%)	Residual VWC (%)	K_v-sat (cm/sec)*	K-Function Estimation Method	K_h-sat (cm/sec)	K_h-sat (ft/sec)	Cond. Ratio K_h/K_v
Sand with Gravel (Bottom Ash)	Silty Sand	15.0%	1.0%	1.00E-03	Fredlund & Xing	1.00E-02	3.28E-04	10
Silty with Sand (Bottom Ash)	Silty Sand	50.0%	1.0%	1.00E-03	Fredlund & Xing	1.00E-02	3.28E-04	10
Fill Clay with Gravel (Embankment Fill)	Clay	25.0%	3.0%	1.00E-07	Fredlund & Xing	4.00E-07	1.31E-08	4
Residual Fat Clay	Clay	25.0%	3.0%	3.60E-07	Fredlund & Xing	3.60E-06	1.18E-07	10
Silt (Fly Ash)	Silt	40.0%	2.0%	4.00E-06	Fredlund & Xing	4.00E-05	1.31E-06	10
Stability Buttress	Gravel	39.0%	1.0%	1.01E+00	Fredlund & Xing	1.01E+00	3.31E-02	1

*Ref: Based on Stantec's calibrated seepage models and subsequently modified based on a parametric study by URS.

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[1] Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372, Work Plan WCF-100521-WP-24
[2] Slope Improvements IRRP-URSAC002 & IRRP-URSAC003, TVA Project No. 204372, Work Plan WCF-100519-WP-29
[3] Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc.

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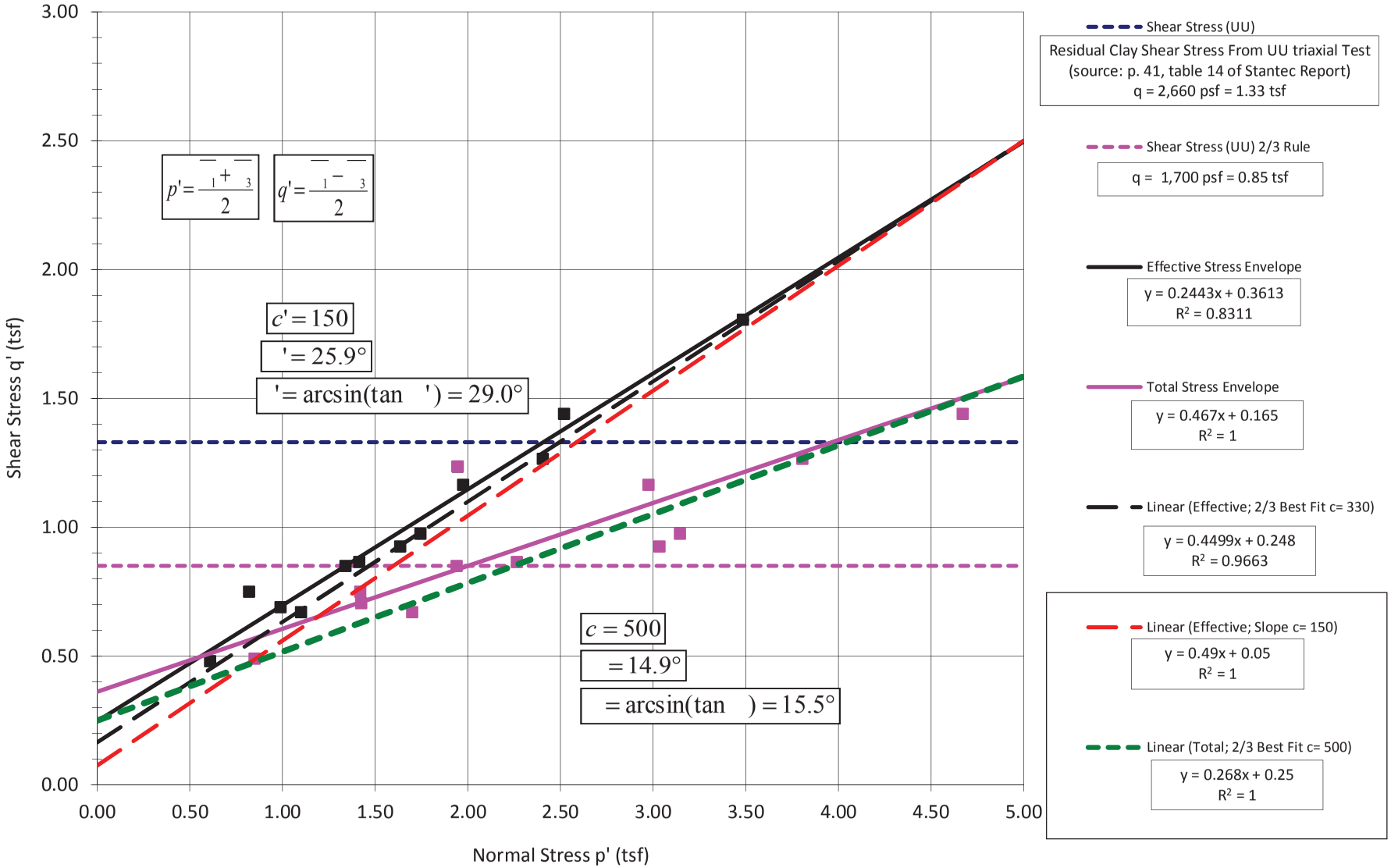
WCF Material Properties Analysis
Calculated: AGH
Checked: JJH

3/16/2012

Figure 2

Clay Fill (linear trend with cohesion)

Effective/Total Stress from CU Triaxial Test[1]

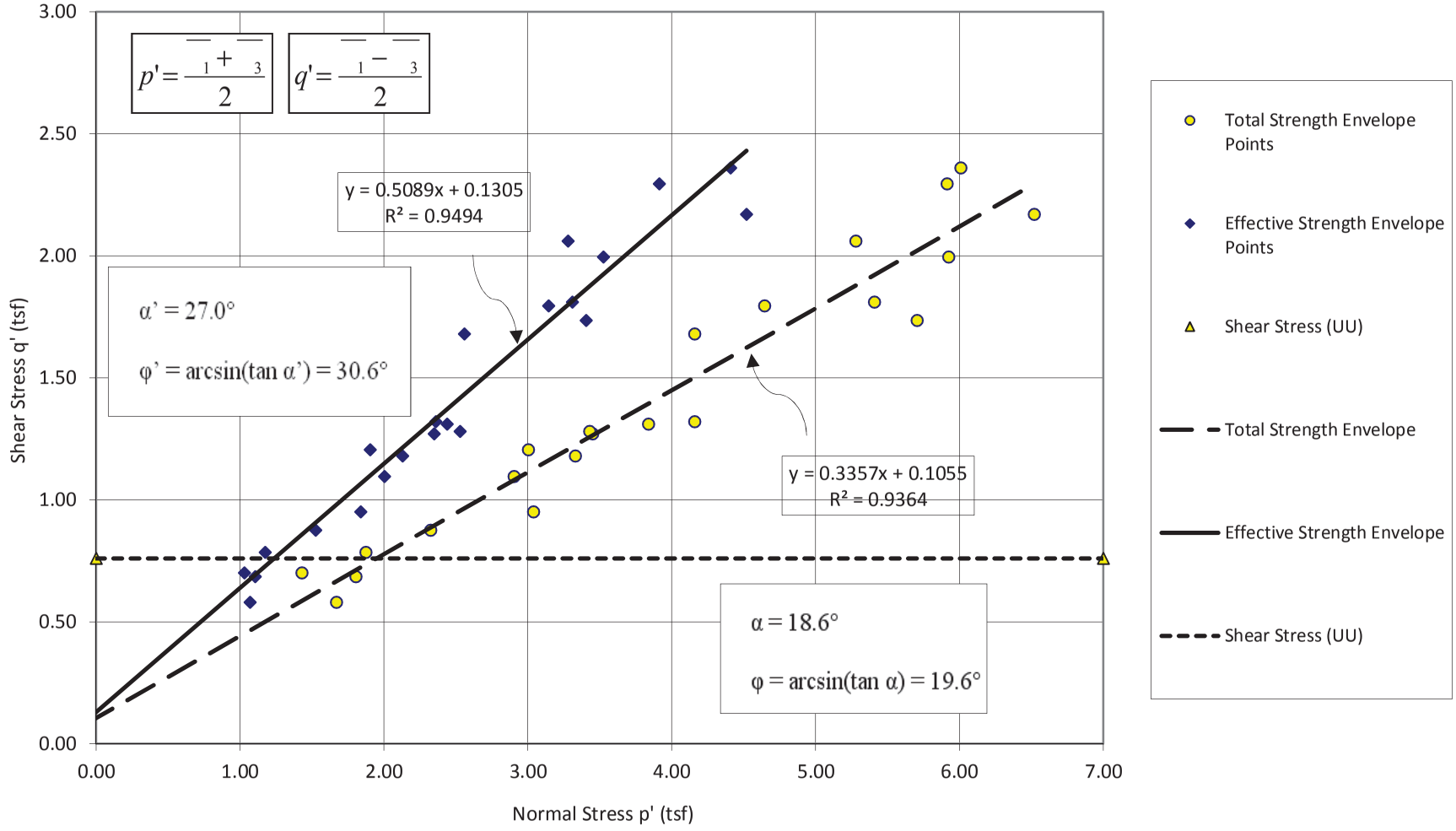


Note: Raw data taken from Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc., and additional labratory testing completed Aug. 2010.

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Figure 3

Residual Clay (linear trend with cohesion)
Effective/Total Stress from CU Triaxial Test[1]



Note: Raw data taken from Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc., and additional labratory testing completed Aug. 2010.

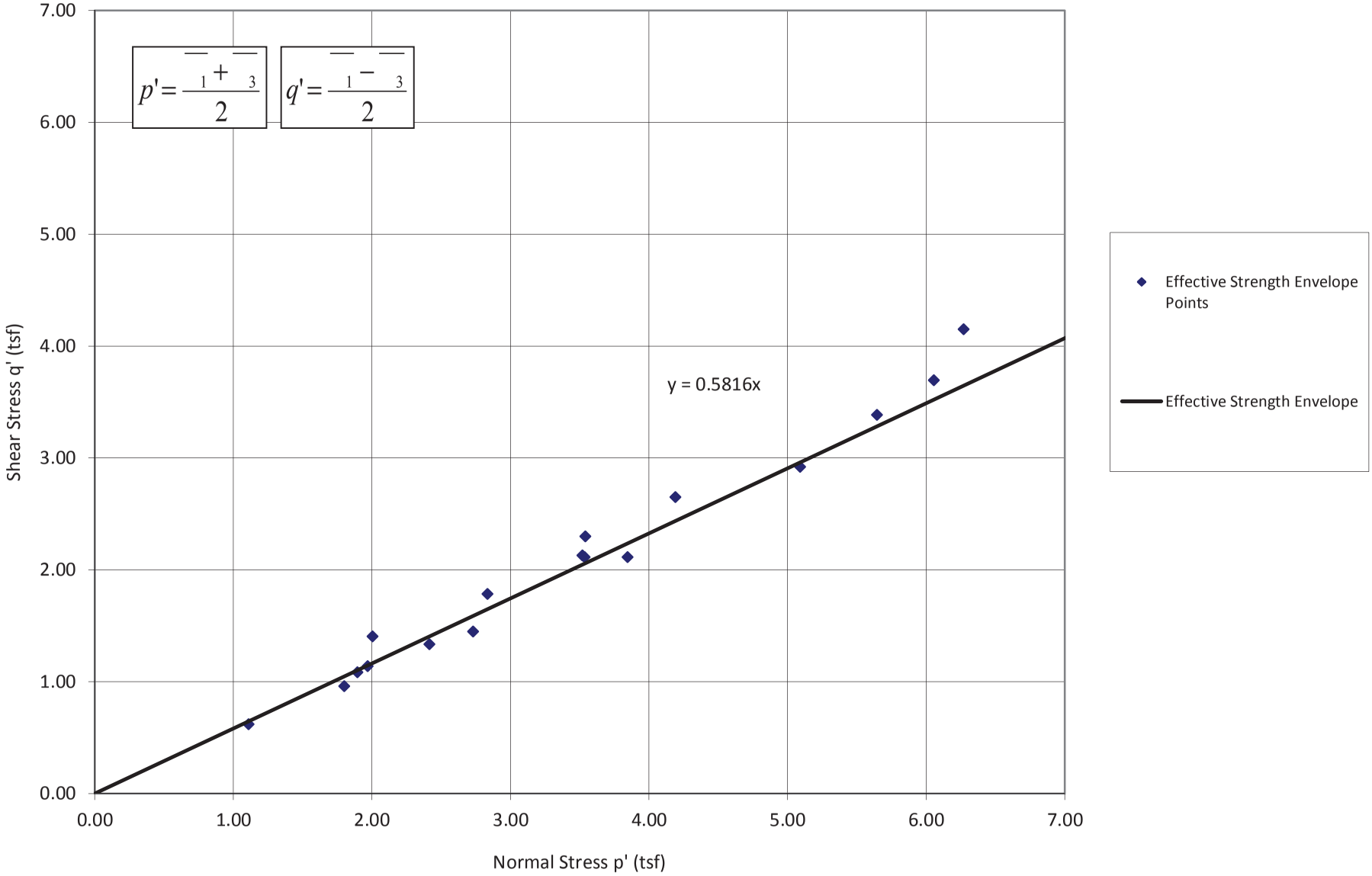
US EPA ARCHIVE DOCUMENT

WCF Material Properties Analysis
Calculated: ANR
Checked: JJH

3/16/2012

Figure 4

Fly Ash (linear trend with zero cohesion)
Effective Stress from CU Triaxial (R) Test[1]



Note: Raw data taken from Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc., and additional labratory testing completed Aug. 2010.

Phi and Cohesion Calculations
Clay Fill

Effective Stress Calculations:

$$\tan(\ ') = \frac{y}{x} \qquad '= \arcsin(\tan\ ')$$

Effective from chart c= 496		
y = 0.4499x + 0.248		
for x=	0	5
y=	0.248	2.4975
α' =	24.2	
Φ' =	26.7	

Effective; Equation set c=0		
y=0.5718x		
for x=	2	5
y=	1.1436	2.859
α' =	29.8	
Φ' =	34.9	

Effective; Slope c= 150		
y = 0.485x + 0.075		
for x=	0	5
y=	0.075	2.5
α' =	25.9	
Φ' =	29.0	

Effective; 2/3 Best Fit c= 330		
y = 0.467x + 0.165		
for x=	0	5
y=	0.165	2.5
α' =	25.0	
Φ' =	27.8	

*Units for cohesion are in psf unless otherwise noted.

Total Stress Calculations:

$$\tan(\ \ ) = \frac{y}{x} \qquad = \arcsin(\tan\ \ )$$

Total from chart c= 723		
y = 0.2443x + 0.3613		
for x=	2	5
y=	0.8499	1.5828
α =	13.7	
Φ =	14.1	

Total; Equation set c=0		
y=0.3542x		
for x=	2	5
y=	0.7084	1.771
α =	19.5	
Φ =	20.7	

Total; 2/3 Best Fit c= 500		
y = 0.267x + 0.25		
for x=	0	5
y=	0.25	1.585
α' =	14.9	
Φ' =	15.5	

US EPA ARCHIVE DOCUMENT

WCF Material Properties Analysis
Calculated: ANR
Checked: JJH

Residual Clay
Phi and Cohesion Calculations

3/16/2012

Effective Stress Calculations: $\tan(\ ') = \frac{y}{x}$ $'= \arcsin(\tan\ ')$

Effective from chart c= 261		
y=0.5089x+0.1305		
for x=	0	5
y=	0.1305	2.675
α' =	27.0	
Φ' =	30.6	

Effective; Equation set c=0		
y=0.5533x		
for x=	2	5
y=	1.1066	2.7665
α' =	29.0	
Φ' =	33.6	

Total Stress Calculations: $\tan(\ ) = \frac{y}{x}$ $= \arcsin(\tan\ )$

Total from chart c= 211		
y=0.3357x + 0.1055		
for x=	2	5
y=	0.7769	1.784
α =	18.6	
Φ =	19.6	

Total; Equation set c=0		
y=0.359x		
for x=	2	5
y=	0.718	1.795
α =	19.7	
Φ =	21.0	

*Units for cohesion are in psf unless otherwise noted.

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Effective Stress Calculations:

$$\tan(\ ') = \frac{y}{x} \qquad '= \arcsin(\tan\ ')$$

Effective; Equation set c=0		
y=0.5816x		
for x=	2	5
y=	1.1632	2.908
α' =	30.2	
Φ' =	35.6	

[1]Fly ash is assumed to be cohesionless.

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3.0 STABILITY ANALYSIS

3.1 General

Five critical cross-sections were examined as a part of this project; two for AC-IRRP 2, Section A and A-prime, and three for AC-IRRP 3, Sections D, E, and F. All sections were analyzed using SEEP/W and SLOPE/W (See **Figure 1** for their locations). The electronic files for the sections were initially provided by Stantec Inc., as part of the Phase II Geotechnical Report[2] and then modified by URS updating cross-section geometry, material properties, boundary conditions, as well as adding the final buttress design.

3.2 Seepage Analysis

Based on design guidelines from USACE and the Master Programmatic Document, Version 1, seepage analyses were conducted and factors of safety for piping computed. Seepage analyses indicated that seepage pressures and gradients within the clay dikes are insufficient to cause significant migration of fine cohesive soil particles within the dike. Therefore, piping is not anticipated and a reverse graded filter is not necessary for either IRRP 2 or 3. The SEEP/W output report files and factor of safety calculations are located in **Appendix B**.

The predicted phreatic surface of the Dredge Cell (upstream boundary condition) was set at elevation 636.0, and the buttress designed accordingly. This was based on the assumption that future projects would be designed and constructed with long-term dewatering solutions, such that a maximum phreatic surface elevation of 636.0 would be maintained for the Dredge Cell.

3.3 Global Stability Analysis

The sections were analyzed for global stability, defined as a slip surface with a minimum depth of 10-ft, that would breach the top of the dike at Elev. 636.0±. Existing conditions as well as post IRRP conditions were modeled. The configuration of the stability buttress is depicted on the stability analysis figures in **Appendix C**, as well as Sheets 7, 8 and 9 of the design drawings[2]. The minimum bench elevation for the toe buttress varied for each IRRP. AC-IRRP 2 was designed with a pipe cradle to accommodate possible future extension of the siphon pipes, therefore the bench width is a 12-ft minimum section, followed by a 2H:1V slope to an upper bench with a 12-ft minimum width. For AC-IRRP 3 a single 15-ft minimum bench was required to meet FS requirements. **Table 3** lists the FS as a result of the stability analysis for existing conditions and post IRRP. The stability analysis output reports for each critical cross section are

[1] Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372, Work Plan WCF-100521-WP-24
[2] Slope Improvements IRRP-URSAC002 & IRRP-URSAC003, TVA Project No. 204372, Work Plan WCF-100519-WP-29
[3] Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc.

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also in **Appendix C**. Total Stress Analyses were also completed to verify that the undrained strength of the clay would support the stability buttress and meet a minimum FS of 1.3 for construction loading conditions.

Table 3 Stability Analysis Results

Analysis Section / Description	Dredge Cell Phreatic Surface Elev.	Factor of Safety (Effective Stress)	Factor of Safety (Total Stress)
Section A' (Existing Condition)	641.0	1.10*	n/a
Section A' (After IRRP Constructed)	636.0	1.55	1.42
Section A (Existing Condition)	641.0	1.20	n/a
Section A (After IRRP Constructed)	636.0	1.58	1.50
Section D (Existing Condition)	638.0	1.29	n/a
Section D (After IRRP Constructed)	636.0	1.53	1.30
Section E (Existing Condition)	636.0	1.45	n/a
Section E (After IRRP Constructed)	636.0	1.54	1.30
Section F (Existing Condition)	636.0	1.41	n/a
Section F (After IRRP Constructed)	636.0	1.50	1.30

Note: Predicted Factors of Safety resulting from this analysis assume a phreatic surface in the Dredge Cell at Elev. 636.0. Installation of dewatering methods, and improved storm water management would further increase these values.
*Section A' represents a combination of worst case scenarios including maximum phreatic surface, perimeter drainage conditions, and slope geometry.

[1] Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372, Work Plan WCF-100521-WP-24
[2] Slope Improvements IRRP-URSAC002 & IRRP-URSAC003, TVA Project No. 204372, Work Plan WCF-100519-WP-29
[3] Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc.

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4.0 PIPE CRUSHING CALCULATIONS

4.1 30-in Gypsum Stack Decant Pipe

A 30-in diameter concrete and steel pipe conveys decant water from the Gypsum Stack Stilling Pond, across the Million Dollar Bridge, continuing along the entire length of the east side (at the toe of slope) of the Dredge Cell, until discharging to an outlet at NPDES monitoring point DSN008. This pipe is located beneath the proposed location of the AC-IRRP 3 buttress.

Pipe crushing calculations were completed to verify if the 30-in existing Gypsum Stack Decant Pipe will withstand the addition of the buttress loading as well as the temporary construction live loads. Based on the American Concrete Pipe Association Design Method, the required three-edge bearing strength was calculated based on a FS of 1.5 and then compared to typical pipe strengths for non-reinforced Class III pipe (assumed to be the type of pipe installed as indicated by the original construction drawings 10H232-1 through 10H232-5). The pipe was found to be adequate to withstand the additional loading. A minimum of 3-ft of cover (2-ft of natural soil plus 1-ft of aggregate road base improvement) will provide sufficient protection against pipe collapse. The pipe calculations are provided in **Appendix D**.

4.2 Sluice Pipe Bridge Crossing

Multiple existing sluice pipes run along the south side of the Pump Pond and then continue north along the west side of the Dredge Cell at the crest of the 636 existing dike. In order to facilitate construction access, these pipes will need to be crossed.

Pipe crushing calculations were completed to analyze the capacity of the solid wall HDPE sluice pipes at the southern end of the Pump Pond for construction live loading. Proper bedding and backfill compaction would be difficult to achieve due to the location of these pipes and their close proximity to each other, resulting in excessive fill heights necessary to attenuate the live load stresses. In addition, the pipe wall thickness could not be verified; pipe scour resulting from sluiced ash conveyance was of significant concern. The combination of these factors led to the development of a temporary pipe bridge crossing as the best alternative. The bridge crossing section and details can be found on sheet 10W519-15 of the design drawings[2]. The associated calculations can also be found in **Appendix E**.

[1] Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372, Work Plan WCF-100521-WP-24
[2] Slope Improvements IRRP-URSAC002 & IRRP-URSAC003, TVA Project No. 204372, Work Plan WCF-100519-WP-29
[3] Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc.

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Appendix A Design Drawings

Appendix B SEEP/W Output Reports and Underseepage Factor of Safety Calculations

Appendix C SLOPE/W Output Reports

Appendix D Pipe Crushing Calculations

Appendix E Sluice Pipe Bridge Crossing

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[1] Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372, Work Plan WCF-100521-WP-24
[2] Slope Improvements IRRP-URSAC002 & IRRP-URSAC003, TVA Project No. 204372, Work Plan WCF-100519-WP-29
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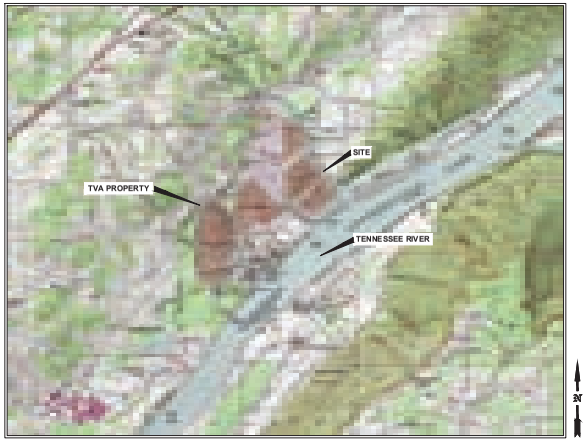
Design Drawings

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TENNESSEE VALLEY AUTHORITY
WIDOWS CREEK FOSSIL PLANT
WORK PLAN FOR ASH COMPLEX SLOPE STABILITY IMPROVEMENTS IRRP-URSAC002 & IRRP-URSAC003
TVA PROJECT NO. 204372
WORK PLAN WCF-10W519-WP-29

STEVENSON, JACKSON COUNTY, ALABAMA
NOVEMBER 21, 2011



SITE VICINITY MAP

0 2000 4000

SCALE IN FEET

DWG. NO.	DWG. DESCRIPTION
10W519-01	COVER SHEET
10W519-02	GENERAL NOTES
10W519-03	LEGEND SHEET
10W519-04	EXISTING CONDITIONS PLAN
10W519-05	PROPOSED CONDITIONS & ALIGNMENT - IRRP 2
10W519-06	PROPOSED CONDITIONS & ALIGNMENT - IRRP 3
10W519-07	IRRP 2 TOE BUTTRESS PLAN & TYPICAL SECTIONS
10W519-08	IRRP 3 TOE BUTTRESS PLAN & TYPICAL SECTIONS I
10W519-09	IRRP 3 TOE BUTTRESS PLAN & TYPICAL SECTIONS II
10W519-10	IRRP 2 CROSS SECTIONS I
10W519-11	IRRP 2 CROSS SECTIONS II
10W519-12	IRRP 3 CROSS SECTIONS I
10W519-13	IRRP 3 CROSS SECTIONS II
10W519-14	IRRP 3 CROSS SECTIONS III
10W519-15	MISCELLANEOUS DETAILS
10W519-16	CONSTRUCTION BEST MANAGEMENT PRACTICES PLAN DETAILS

2 WORKING DAYS
BEFORE YOU DIG
205-353-2636
ALABAMA UTILITIES PROTECTION SERVICE

PREPARED FOR:
TENNESSEE VALLEY AUTHORITY
1101 MARKET STREET
CHATTANOOGA, TN 37402-2801

PREPARED BY:
URS CORPORATION
1375 EUCLID AVENUE
SUITE 600
CLEVELAND, OH 44115-1808
PHONE: (216) 622-2400

ISSUED FOR CONSTRUCTION

FOR SUPPORTING DESIGN CALCULATIONS SEE: CDX000000020110004

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT #	AS CONST
R 0	11/21/11	AGH	AN/KV	JJH	JDM	JM	MST	JCK	204372	0

SCALE: NONE EXCEPT AS NOTED

COVER SHEET

WORK PLAN WCF-10W519-WP-29
ASH COMPLEX IRRP-URSAC002 & IRRP-URSAC003
SLOPE STABILITY IMPROVEMENTS

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
AGH	AN/KV	JJH	JDM	JM	MST	JCK

URS

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD 2010 | DATE 11/21/11 | 34 | C | 10W519-01 | R 0

URS CORPORATION | 0
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:XX W_TVA

C.A.D. DRAWING DO NOT ALTER MANUALLY

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10W519-01 | C | 34

GENERAL NOTES:

1. SUMMARY OF WORK/GENERAL PROVISIONS

1.1. PURPOSE: THE PURPOSE OF THIS WORK IS TO CONSTRUCT A TOE BUTTRESS ON SELECTED AREAS OF THE DOWNSTREAM FACE OF THE PERIMETER CLAY DIKE OF THE DREDGE CELL. THE TOE BUTTRESS WILL CONSIST OF RIP-RAP MEETING THE REQUIREMENTS GIVEN HEREIN.

1.2. DEFINITIONS: THE OWNER IS TENNESSEE VALLEY AUTHORITY (TVA). THE ENGINEER, URS CORPORATION (URS), HAS PREPARED THE PLANS AND NOTES HEREIN. THE CONTRACTOR IS TVA OR THE ENTITY WITH WHOM TVA HAS CONTRACTED AN AGREEMENT WITH TO PERFORM THE CONSTRUCTION OF THE PROJECT.

1.3. PERMITTING: PERMITTING REQUIREMENTS ASSOCIATED WITH THIS WORK SHALL BE COMPILED BY TVA AND SHALL BE MAINTAINED/VERIFIED BY THE ENVIRONMENTAL GROUP OF TVA.

1.4. MAPPING: THE EXISTING CONDITIONS, TOPOGRAPHIC AND SURVEY INFORMATION SHOWN ON THESE DRAWINGS WAS PROVIDED BY TVA AND IS BASED UPON AN AERIAL SURVEY CONDUCTED ON FEBRUARY 2, 2010. THIS INFORMATION WAS SUPPLEMENTED WITH SOUNDING DATA, AND SURVEY DATA COLLECTED AT VARIOUS INTERVALS TO DATE, FURNISHED BY TVA AND/OR THEIR OTHER CONSULTANTS AND CONTRACTORS, OR OBTAINED FROM DATA UTILIZED ON PREVIOUS TVA PROJECTS. THE EXISTING CONDITIONS SHOWN ON THESE DRAWINGS ARE BELIEVED TO REPRESENT THE BEST KNOWN DATA AVAILABLE. ALTHOUGH THIS INFORMATION HAS BEEN INCLUDED IN THE CONSTRUCTION PLANS BY URS CORP. (URS), THEY ARE NOT NECESSARILY THE WORK PRODUCT OF URS AND HAVE BEEN INCORPORATED IN RESPONSE TO CERTAIN ASSIGNED CONDITIONS. URS HAS NOT SURVEYED OR FIELD VERIFIED ANY OF THE EXISTING CONDITION INFORMATION AND CANNOT BE HELD RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF THIS DATA.

TOPOGRAPHIC MAPPING IS BASED UPON THE FOLLOWING DATUM AND PROJECTION:

ALABAMA MERCATOR EAST STATE PLANE
HORIZONTAL DATUM: TVA LOCAL
VERTICAL DATUM: NGVD 29

BENCHMARK LOCATIONS TO BE PROVIDED BY TVA SURVEYING & PROJECT SERVICES GROUP. IF BENCHMARKS ARE DISTURBED, CONTRACTOR SHALL COORDINATE WITH SURVEYING & PROJECT SERVICES TO RE-ESTABLISH BENCHMARKS.

1.5. EXISTING INSTRUMENTATION & UTILITIES: ALL EXISTING PIEZOMETERS SHALL BE PROTECTED BY CONTRACTOR DURING CONSTRUCTION ACTIVITIES. IF NECESSARY CONTRACTOR TO EXTEND PIPE STICK UP AND RE-FINISH WELL IF BUTTRESS CONSTRUCTION COVERS OR INHIBITS THE ABILITY TO OBTAIN WELL MONITORING. IF DAMAGE TO AN EXISTING WELL OCCURS CONTRACTOR IS RESPONSIBLE FOR RELOCATION AND/OR REPLACEMENT OF THE WELL. FOR DETAILS SEE MISCELLANEOUS DETAILS SHEET.

EXISTING UNDERGROUND UTILITY LOCATIONS AS SHOWN ON THESE PLANS ARE APPROXIMATE AND ARE TO BE FIELD VERIFIED. CONTRACTOR SHALL CONTACT TVA-WCF PLANT ENGINEER (ROBERT KNOX – 256-437-4250) AND UNDERGROUND LOCATORS OF NASHVILLE (JAMES R. PEMERTON – 615-943-7292) FOR LOCATION VERIFICATION BEFORE DIGGING.

THE SCOPE DESCRIBED HEREIN INVOLVES WORK IN THE VICINITY OF EXISTING OVERHEAD HIGH-VOLTAGE LINES. PRIOR TO COMMENCING CONSTRUCTION ACTIVITIES, THE CONTRACTOR SHALL COORDINATE WITH TVA REGARDING THE CONSTRAINTS, CLEARANCE REQUIREMENTS, AND RESTRICTIONS ASSOCIATED WITH WORK IN THE VICINITY OF THESE FEATURES. SEE CROSS SECTION SHEETS FOR APPROXIMATE ELEVATIONS AND DETAILED NOTES.

1.6. ACCESS: VEHICULAR ACCESS TO THE SITE SHALL BE DETERMINED BY TVA. NO CONSTRUCTION ACTIVITIES SHALL BE PERFORMED THAT CAUSE AN INTERFERENCE WITH THROUGH TRAFFIC OR TVA OPERATIONS. CONTRACTOR SHALL PROVIDE APPROVED SIGNAGE, BARRIERS, AND TRAFFIC SAFETY PERSONNEL RESPONSIBLE FOR DIRECTING CONSTRUCTION AND PLANT TRAFFIC AT ALL POINTS OF INGRESS AND EGRESS.

1.7. EXCAVATION: NO EXCAVATION IS ANTICIPATED FOR THIS PROJECT.

2. CONTRACTOR ACTIVITIES

2.1. SAFETY: CONTRACTOR SHALL BE RESPONSIBLE FOR THE HEALTH AND SAFETY OF ITS PERSONNEL AND SHALL MEET ALL APPLICABLE LOCAL, STATE, AND FEDERAL SAFETY REGULATIONS, INCLUDING BUT NOT LIMITED TO OSHA STANDARDS AND TVA SAFETY REQUIREMENTS.

2.2. QUANTITIES: CONTRACTOR SHALL EMPLOY A COMPETENT SURVEYOR LICENSED IN THE STATE OF ALABAMA TO ESTABLISH OR CONFIRM ALL LINES, ELEVATIONS, REFERENCE MARKS, ETC. NEEDED BY THE CONTRACTOR AND ENGINEER DURING CONSTRUCTION. THE QUANTITIES OF MATERIALS PRESENTED HEREIN SHALL BE CONSIDERED APPROXIMATE AND ARE SUBJECT TO CHANGE BASED ON FIELD CONDITIONS.

2.3. MEASUREMENTS: CONTRACTOR SHALL BASE ALL MEASUREMENTS, BOTH VERTICAL AND HORIZONTAL, ON ESTABLISHED BENCHMARKS AND MONUMENTS AS PROVIDED BY TVA. CONTRACTOR IS RESPONSIBLE FOR VERIFICATION OF THE TVA MONUMENTATION AND ALL CONSTRUCTION LAYOUT. EXISTING GRADES AS WELL AS DIMENSIONS AND CONDITIONS AT THE SITE AFTER CONSTRUCTION SHALL BE VERIFIED BY PERFORMING AN AS-BUILT SURVEY.

2.4. AS-BUILT DRAWINGS: NO DEVIATIONS FROM THE PLANS SHALL BE MADE WITHOUT PRIOR APPROVAL FROM TVA AND THE ENGINEER. CONTRACTOR SHALL MAINTAIN AT THE SITE A COMPLETE SET OF PROJECT PLANS AS ISSUED WITH THE CONSTRUCTION DOCUMENTS. PROJECT PLANS SHALL BE UPDATED BY THE CONTRACTOR TO SHOW ANY AND ALL DEVIATIONS MADE DURING CONSTRUCTION. THESE PLANS SHALL BE LABELED AS-BUILT AND SUBMITTED TO THE ENGINEER AT THE COMPLETION OF THE PROJECT OR WITHIN SEVEN (7) DAYS OF THE PROJECT COMPLETION. ALL SUCH REVISIONS SHALL BE MARKED UPON EXECUTION AND AT A MINIMUM EVERY WEEK TO KEEP THE DRAWING SET CURRENT DURING THE CONSTRUCTION PROCESS AND PRIOR TO ANY ITEM BECOMING INACCESSIBLE FOR AN AS-BUILT DRAWING TO BE PERFORMED.

2.5. CONSTRUCTION LIMITS: CONTRACTOR SHALL CONFINE HIS EQUIPMENT, STORAGE, AND OPERATIONS TO ONLY THE LIMITS OF WORK AND AREAS DESIGNATED FOR THOSE USES BY TVA. WHERE TEMPORARY ENTRY IS REQUIRED, THE CONTRACTOR SHALL ENTER THOSE AREAS ONLY FOR THE DURATION OF TIME REQUIRED TO COMPLETE THE WORK. THE OWNER MAY HAVE OTHER CONTRACTORS PERFORMING WORK WITHIN THESE DEFINED CONSTRUCTION LIMITS AND AT NEARBY AREAS DURING THE TIME OF THIS WORK. THE CONTRACTOR SHALL COORDINATE WITH OTHER CONTRACTORS IN THE EXECUTION OF THE WORK AND SHALL ACCOMMODATE ACCESS ROUTE REVISIONS, MINOR DELAYS, AND SHARE WORK AREAS AND OTHER COORDINATION EFFORTS WITHOUT ADDITIONAL COST TO THE OWNER. IN ADDITION, THE CONTRACTOR SHALL MAINTAIN CARE WHEN WORKING ADJACENT TO WETLAND, OR OTHER SIMILAR AREAS, TO AVOID AND/OR MINIMIZE DISTURBANCE TO SUCH AREAS.

2.6. BLASTING: BLASTING IS PROHIBITED FOR THIS PROJECT.

2.7. EROSION AND SEDIMENT CONTROL: SILT FENCES AND OTHER EROSION AND SEDIMENT CONTROL MEASURES SHALL BE INSTALLED AT LOCATIONS AS SHOWN IN THESE DRAWINGS AND AS REQUIRED BY THE CONSTRUCTION BEST MANAGEMENT PRACTICES PLAN (CBMPP). INSTALLATION, INSPECTION, AND MAINTENANCE OF SILT FENCES AND OTHER EROSION AND SEDIMENT CONTROL MEASURES SHALL FOLLOW REQUIREMENTS OF ALABAMA DEPARTMENT OF ENVIRONMENTAL MANAGEMENT (ADEM) AS DETAILED IN THE ALABAMA HANDBOOK FOR EROSION CONTROL, SEDIMENT CONTROL AND STORMWATER MANAGEMENT ON CONSTRUCTION SITES AND URBAN AREAS. SEE THE CBMPP PLAN FOR ADDITIONAL DETAILS.

2.8. FUGITIVE DUST CONTROL: CONTRACTOR IS RESPONSIBLE FOR FUGITIVE DUST CONTROL TO PREVENT THE GENERATION OF AIRBORNE DUST DURING SITE OPERATIONS.

2.9. MATERIALS STORAGE: DELIVERED MATERIALS FOR INCORPORATION INTO THE WORK MAY BE TEMPORARILY STORED IN AREAS DESIGNATED FOR SUCH USE BY THE OWNER. ALL MATERIAL STOCKPILES SHALL CONFORM WITH THE REQUIREMENTS OF THE CBMPP.

2.10. DEBRIS DISPOSAL: ALL EXCESS CONSTRUCTION SPOILS AND DEBRIS SHALL BE DEWATERED AND DISPOSED OFFSITE IN AREAS AS APPROVED BY THE OWNER.

2.11. SITE RECLAMATION: UPON COMPLETION OF THE PROJECT, THE CONTRACTOR SHALL REMOVE ALL PERSONNEL AND EQUIPMENT FROM THE SITE, INCLUDING ANY TEMPORARY ITEMS INSTALLED BY THE CONTRACTOR DURING THE COURSE OF THE WORK. THE SITE SHALL BE LEFT IN A SIGHTLY CONDITION WITH ALL TRASH AND RUBBISH REMOVED. ANY PARKING AND STAGING AREAS PREPARED BY THE CONTRACTOR SHALL BE REMOVED AND RESTORED TO THEIR ORIGINAL CONDITION, AT MINIMUM.

2.12. FIELD ADJUSTMENTS: ADJUSTMENTS WILL BE NECESSARY AS THE FIELD WORK PROGRESSES AND SHOULD BE AGREED UPON BY THE OWNER, ENGINEER AND CONTRACTOR PRIOR TO IMPLEMENTATION. IT IS RECOMMENDED THAT CONSTRUCTION MEASURES BE MONITORED BY URS PERSONNEL.

2.13. CONSTRUCTION QUALITY ASSURANCE (CQA) PLAN: A CQA PLAN HAS BEEN PREPARED FOR THIS PROJECT, AND SHALL BE ADHERED TO BY THE CONTRACTOR.

3. MATERIALS

3.1. ROCK TOE BUTTRESS: MATERIALS TO BE USED FOR CONSTRUCTION OF THE TOE BUTTRESS CONSIST OF CRUSHED LIMESTONE AGGREGATES. THESE MATERIALS SHALL CONFORM TO THE REQUIREMENTS OF THE ALABAMA DEPARTMENT OF TRANSPORTATION (ALDOT) STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION, 2008 EDITION, SECTION 814 RIPRAP MATERIALS FOR CLASS 1 RIPRAP (OR THE TENNESSEE DEPARTMENT OF TRANSPORTATION (TDOT) STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION, 2006 EDITION, MAY BE SUPPLEMENTED, AS APPROVED BY THE ENGINEER, FOR SECTION 709 RIPRAP AND SLOPE PAVEMENT FOR CLASS A-1 RIP RAP.)

3.1.1. RIPRAP SHALL CONSIST OF FURNISHING AND SETTING OR PLACING STONES, CONSTRUCTED WITHIN REASONABLY CLOSE CONFORMITY TO THE LINES, GRADES, AND CROSS SECTIONS, AND AT THE LOCATIONS INDICATED ON THE PLANS OR AS DIRECTED BY THE ENGINEER, AND IN CONFORMITY WITH THE REQUIREMENTS AND PROVISIONS OF THESE SPECIFICATIONS:

3.1.1.1. ALDOT CLASS 1 RIPRAP: STONE FOR THIS CLASS RIPRAP SHALL CONSIST OF GRADED STONES RANGING FROM 10 TO 100 POUNDS (5 TO 50 kg) WITH NOT MORE THAN 10% HAVING A WEIGHT (MASS) OVER 100 POUNDS (50 kg) AND AT LEAST 50% HAVING A WEIGHT (MASS) OVER 50 POUNDS (25 kg) AND NOT OVER 10% HAVING A WEIGHT (MASS) UNDER 10 POUNDS (5 kg).

3.1.1.2. TDOT CLASS A-1 RIPRAP: (MACHINED RIPRAP) SHALL VARY IN SIZE FROM 2 IN. TO 1.25 FEET (50 TO 375 MM) WITH NO MORE THAN 20% BY WEIGHT BEING LESS THAN 4 IN. (100 MM). THE MINIMUM THICKNESS OF THE STONE LAYER SHALL BE 1.5 FT. (450 MM) WITH A TOLERANCE OF 3 IN. (75 MM).

3.2. ACCESS ROUTES: MATERIALS TO BE USED FOR THE CONSTRUCTION OF NEW ACCESS RAMPS OR IMPROVEMENT TO EXISTING ACCESS ROADS CONSIST OF THE FOLLOWING

3.2.1. CHOKER STONE: CRUSHED AGGREGATES SHALL MEET THE REQUIREMENTS OF ALDOT SECTION 825 AND GRADATION TYPE B.

3.2.2. COARSE AGGREGATES: COARSE AGGREGATES SHALL MEET THE REQUIREMENTS OF ALDOT SECTION 801.11 AND GRADATION SIZE 1 OR 2.

3.3. CLAY BUTTRESS: MATERIALS TO BE USED FOR THE CONSTRUCTION OF THE CLAY BUTTRESS FOR IRRP 2 ON THE INTERIOR SLOPE OF THE PUMP POND SHALL CONSIST OF THE FOLLOWING:

3.3.2. ENGINEERED FILL: SUITABLE MATERIALS SHALL CONSIST OF CLAY SOILS THAT EXHIBIT A CLASSIFICATION OF CL, CL-ML, CL-CH, OR CH IN ACCORDANCE WITH ASTM D-2487. THE MATERIAL SHALL BE FREE OF ORGANIC MATTER, TOPSOIL, RUBBISH, DEBRIS, WASTE MATERIALS, AND ROCK PIECES GREATER THAN 3 INCHES (MAXIMUM DIMENSION) IN SIZE, AND SHALL BE AT A MOISTURE CONTENT THAT IS SUITABLE FOR ACCEPTABLE COMPACTION, AS REQUIRED HEREIN.

3.3.3. ENGINEERED FILL SHALL BE PLACED AND SPREAD IN HORIZONTAL LOOSE LIFTS THAT DO NOT EXCEED 8 INCHES IN THICKNESS. EACH LIFT SHALL BE THOROUGHLY COMPACTED TO ACHIEVE A MINIMUM DENSITY OF 95% OF ITS MAXIMUM DRY DENSITY AS DETERMINED BY ASTM D-698 (CONTRACTOR TO PERFORM PROCTOR TESTING AND SUBMIT RESULTS TO ENGINEER). THE MOISTURE CONTENT DURING COMPACTION SHALL BE WITHIN A RANGE OF +2 TO -2 PERCENT OF THE OPTIMUM MOISTURE CONTENT AS DETERMINED BY ASTM D-698. FIELD COMPACTION TESTS, AT A FREQUENCY OF NOT LESS THAN 3 TESTS PER LIFT, SHALL BE PERFORMED ON EACH LIFT TO VERIFY THAT SPECIFIED LEVELS OF COMPACTION ARE ACHIEVED.

3.3.4. WHEN HORIZONTAL LIFTS ARE PLACED ADJACENT TO THE SIDE SLOPES OF THE EXISTING DIKE, LIFTS SHALL BE KEYED INTO THE EXISTING CLAY DIKE A MINIMUM OF 2-FT PER LIFT.

3.4. GEOGRID: TENSAR BX1300, OR APPROVED EQUAL, WILL BE USED IF NECESSARY FOR BASE STABILIZATION AS DIRECTED BY THE ENGINEER.

3.5. QUANTITIES: ESTIMATED MATERIAL QUANTITIES ARE SHOWN IN THE TABLE ON THIS SHEET. NOTE: CONTRACTORS SHALL PERFORM THEIR OWN ESTIMATES OF MATERIAL QUANTITIES FOR BIDDING AND CONSTRUCTION PURPOSES.

4. CONSTRUCTION ACTIVITIES

4.1. LAYOUT: CONTRACTOR SHALL FIELD-ESTABLISH THE BASELINE SHOWN ON DRAWINGS AND SHALL ESTABLISH OFFSETS FOR THE TOP OF THE TOE BUTTRESS AS SHOWN ON DRAWINGS. CONTRACTOR SHALL LOCATE/LAYOUT PROJECT BASELINE TO THE EXTENT NECESSARY TO DETERMINE BEGINNING AND END OF BUTTRESS SECTIONS, AND AS IS SUFFICIENT FOR FIELD LOCATION ALONG ITS ALIGNMENT.

4.2. MOWING: PRIOR TO PLACEMENT OF BUTTRESS MATERIALS, THE SLOPE AND TOE AREA SHALL BE PREPARED BY MOWING ANY GRASS TO THE LOWEST PRACTICAL LIMIT (MAX 3" HIGH). TYPICAL MOWING EQUIPMENT (E.G., "BUSH-HOG", ETC.). CUT GRASS SHALL BE REMOVED OR RAKED FROM THE LIMITS OF THE BUTTRESS, AS DIRECTED BY THE OWNER.

4.3. CLEARING AND GRUBBING: IN GENERAL, THE AREAS DESIGNATED FOR TOE BUTTRESS INSTALLATION SHALL ONLY REQUIRE MOWING PRIOR TO COMMENCEMENT OF ACTIVITIES. HOWEVER, IN SPECIFIC AREAS, TREES OR OTHER SIGNIFICANT VEGETATION MAY NEED TO BE CLEARED AND GRUBBED IN ORDER TO PROVIDE CONSTRUCTION ACCESS AND/OR BUTTRESS INSTALLATION. THESE AREAS SHALL BE GRUBBED TO A DEPTH OF 6 INCHES TO REMOVE GRASS, ROOTS, AND OTHER ORGANIC MATERIAL. IN ADDITION, VEGETATION AND TOPSOIL SHALL BE STRIPPED FROM AREAS TO RECEIVE EMBANKMENTS/ACCESS RAMPS. THE CONTRACTOR SHALL BE RESPONSIBLE FOR OBTAINING THE APPROVAL FOR THE REUSE, OR DISPOSAL, OF ANY CLEARED AND GRUBBED MATERIALS.

4.4. EROSION CONTROL: MEASURES WILL BE IMPLEMENTED TO CONTROL SEDIMENT PRESENT DURING CONSTRUCTION ACTIVITIES (SEE CBMPP). THE CONTRACTOR WILL BE RESPONSIBLE FOR MAINTAINING APPROPRIATE SEDIMENT CONTROL MEASURES DURING THESE ACTIVITIES.

4.5. RIPRAP: RIPRAP SHALL BE PLACED IN ACCORDANCE WITH THE REQUIREMENTS OF THE ALDOT (OR TDOT) STANDARD SPECIFICATIONS AS DESCRIBED IN THE MATERIALS SECTION ABOVE. THE METHODS AND EQUIPMENT FOR PLACEMENT OF RIPRAP SHALL BE APPROVED BY THE ENGINEER PRIOR TO CONSTRUCTION. THE MATERIAL SHALL BE LAID TO THE GRADES SHOWN, AND SHALL BE PLACED IN SUCH A MANNER AS TO PROVIDE A DENSE MATRIX.

4.6. TOLERANCES: ACCEPTABLE CONSTRUCTION TOLERANCES FROM PLAN DIMENSIONS, ELEVATIONS, AND GRADES SHALL AS SPECIFIED IN THE CQA PLAN. MINIMUM DIMENSIONS RELATED TO BUTTRESS THICKNESS SHALL BE MAINTAINED AS SHOWN ON PLANS REGARDLESS OF TOLERANCES.

4.7. SEEPS: CONTRACTOR SHALL NOTIFY TVA AND THE ENGINEER IF WATER SEEPS ARE OBSERVED DURING THE WORK. THIS CONDITION MAY REQUIRE ADDITIONAL MEASURES AS DIRECTED BY THE ENGINEER.

4.8. STORMWATER MANAGEMENT: THE CONTRACTOR SHALL MAINTAIN THE SITE IN ACCORDANCE WITH THE CBMPP.

4.9. EXISTING TURBIDITY CURTAIN: MAINTAIN EXISTING TURBIDITY CURTAN LOCATIONS AND ORIENTATIONS. LOCATIONS OF ANCHOR POINTS AND/OR TETHERS SHALL BE ADJUSTED TO GRADE AS WORK PROCEEDS. NOTIFY TVA IF TURBIDITY CURTAIN LOCATIONS AND ORIENTATIONS REQUIRE SIGNIFICANT ADJUSTMENT.

5. ANTICIPATED CONSTRUCTION SEQUENCE

5.1. FIELD-ESTABLISH CONSTRUCTION BASELINE AND OTHER LAYOUT CONTROLS. THE CONTRACTOR IS RESPONSIBLE FOR VERIFICATION OF THE TVA MONUMENTATION.

5.2. INSTALLATION OF EROSION AND SEDIMENTATION CONTROL BEST MANAGEMENT PRACTICES (IN ACCORDANCE WITH CBMPP) MUST BE COMPLETED PRIOR TO COMMENCEMENT OF ANY FILL ACTIVITIES OR SOIL DISTURBANCE.

5.3. CONTRACTOR SHALL MINIMIZE USE OF HEAVY EQUIPMENT OR LOADED TRUCKS ON UNIMPROVED PORTIONS OF THE SLOPES. CONTRACTOR SHALL UTILIZE EXISTING ACCESS RAMPS AND HAUL ROADS TO FACILITATE ACCESS AS FOLLOWS:

5.3.1. STAGING AREA LOCATIONS AND TRAFFIC PATTERNS INDICATING NEW ACCESS RAMPS AND/OR HAUL ROAD IMPROVEMENTS SHALL BE SUBMITTED TO THE ENGINEER FOR REVIEW.

5.3.2. IF NEW ACCESS RAMPS AND/OR HAUL ROAD IMPROVEMENTS ARE REQUIRED, VEGETATION SHALL BE STRIPPED, TOPSOIL SHALL BE REMOVED AND EXISTING SOILS SHALL BE SCARIFIED IF NEEDED.

5.3.3. THE FINAL DRIVABLE SURFACE OUTSIDE OF BUTTRESS AREAS SHALL BE CONSTRUCTED OF A MINIMUM 9-INCH LAYER OF COARSE AGGREGATE AND A MINIMUM 3-INCH CHOKING LAYER OF CRUSHED AGGREGATE.

5.4. IRRP 2 ROCK TOE BUTTRESS CONSTRUCTION SHALL GENERALLY PROCEED FROM EAST TO WEST, PLACED TO THE LINES AND GRADES INDICATED ON THE DRAWINGS, IN ACCORDANCE WITH THE FOLLOWING:

5.4.1. PLACE AND COMPACT CLAY FILL ALONG THE PUMP POND INTERIOR SLOPE.
5.4.2. PLACE RIPRAP IN THE PUMP POND.
5.4.3. PLACE RIPRAP IN THE STILLING POND, FORMING AN ACCESS ROAD JUST ABOVE THE WATER LEVEL.
5.4.4. PLACE RIPRAP IN THE STILLING POND, USING THE ACCESS ROAD FOR ONE-WAY TRAFFIC.

5.5. IRRP 3 ROCK TOE BUTTRESS CONSTRUCTION SHALL GENERALLY PROCEED FROM WEST TO EAST, PLACED TO THE LINES AND GRADES INDICATED ON THE DRAWINGS, IN ACCORDANCE WITH THE FOLLOWING:
5.5.1. IMPROVE PERIMETER ACCESS RAMPS AND HAUL ROADS IN ACCORDANCE WITH 5.3 TO FACILITATE ONE-WAY TRAFFIC.
5.5.2. PLACE RIPRAP ALONG THE EXTERIOR SLOPE OF THE DREDGE CELL.

5.6. DELINEATED WETLANDS ARE SHOWN ON DRAWINGS. CONTRACTOR SHALL NOT ENCROACH UPON THESE AREAS WITH ANY EQUIPMENT OR MATERIALS PLACEMENT WITHOUT APPROVAL OF TVA. ORANGE CONSTRUCTION FENCING OR OTHER DELINEATION AS APPROVED BY TVA ENVIRONMENTAL GROUP SHALL BY USED TO IDENTIFY THE LIMITS OF DELINEATED WETLANDS ADJACENT TO THE WORK AREA TO AVOID UNCONTROLLED ENCROACHMENT. THE USE OF "CRANE MATS" OR TEMPORARY FILL TO BRIDGE THESE AREAS MAY BE ACCEPTABLE. IF CONSTRUCTION EQUIPMENT AND MATERIALS STAGING IS DETERMINED TO BE NECESSARY IT WILL BE UNDER DIRECTION OF TVA.

5.7. ALL DISTURBED AREAS SHALL BE PERMANENTLY STABILIZED UPON THE PROJECT COMPLETION, BEFORE EROSION CONTROLS ARE REMOVED. THIS STABILIZATION INCLUDES THE PLACEMENT OF CLASS A-1 RIP-RAP (BUTTRESS AREAS), SEEDING AND MULCHING, OR OTHER APPROVED STABILIZATION TECHNIQUES.

IRRP-URSAC002 ESTIMATED QUANTITIES

MATERIAL	QUANTITY	UNIT
RIPRAP (TOE BUTTRESS)	19,400	CY
RIPRAP (ACCESS ROADS)	2,700	CY
COARSE AGGREGATE	700	CY
CHOKER STONE	250	CY
ENGINEERED FILL	300	CY
GEOGRID (CONTINGENCY)	1,470	SY
TURBIDITY CURTAIN	1,500	LF
MOWING	0.75	AC
3/4" STEEL PLATE	1	SF
CONCRETE	25	CY
WBX28	16	EA

IRRP-URSAC003 ESTIMATED QUANTITIES

MATERIAL	QUANTITY	UNIT
RIP-RAP (TOE BUTTRESS)	14,200	CY
RIP-RAP (ACCESS ROADS)	4,100	CY
COARSE AGGREGATE	1,760	CY
CHOKER STONE	600	CY
GEOGRID (CONTINGENCY)	3,770	SY
TURBIDITY CURTAIN	2,400	LF
SILT FENCE	3,400	LF
MOWING	3.5	AC
MANHOLE RECONSTRUCTION	1	EA

FOR SUPPORTING DESIGN CALCULATIONS SEE: CDX0000000201110004

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV NO.
R 0	11/21/11	AGH	AN/KV	JJH	JDM	JM	MST	JCK	204372		0

DISCIPLINE KEY/PACE

SCALE: NONE EXCEPT AS NOTED

GENERAL NOTES

WORK PLAN WCF-10W519-WP-29

ASH COMPLEX IRRP-URSAC002 & IRRP-URSAC003

SLOPE STABILITY IMPROVEMENTS

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
AGH	AN/KV	JJH	JDM	JM	MST	JCK

URS

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD 2010 | DATE 11/21/11 | 34 | C | 10W519-02 | R 0

URS Corporation | 0
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:XX W_TVA

C.A.D. DRAWING DO NOT ALTER MANUALLY

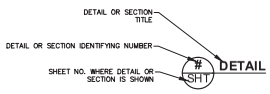
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US EPA ARCHIVE DOCUMENT

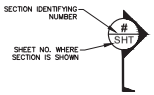
LEGEND

PROPOSED	EXISTING	DESCRIPTION
630	630	MAJOR CONTOUR INTERVAL
629	629	MINOR CONTOUR INTERVAL
		SITE VEGETATION BOUNDARY
N/A		EXISTING FENCE
N/A	OE	EXISTING OVERHEAD ELECTRIC LINE
N/A		EXISTING LIGHT POLE
N/A		EXISTING UTILITY POLE
		PIPES
		CULVERTS
	N/A	LIMITS OF BUTTRESS
	N/A	CENTER LINE OF ALIGNMENT
N/A		EDGE OF WATER
N/A		100 YEAR FLOOD EL. 608.5
		TURBIDITY CURTAIN
SF	N/A	SILT FENCE
630	N/A	BUTTRESS GRADING
		RIP-RAP
N/A		WETLANDS (DELINEATED BY TVA)
		ROADWAY EXTENTS
		TREE
N/A	STN-160	INSTRUMENTATION LOCATION
		DRAINAGE MANHOLE STRUCTURE
		DRAINAGE BASIN/INLET STRUCTURE
N/A		EXISTING TRANSMISSION TOWER (STRAIGHT LINE)
N/A		EXISTING TRANSMISSION TOWER (ANGLE)

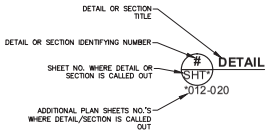
DETAILING & SECTION IDENTIFICATION NOMENCLATURE



DETAIL, SECTION, AND ELEVATION REFERENCE
ON SHEET WHERE DETAIL OR SECTION IS CALLED OUT

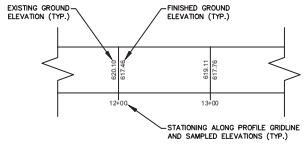


SECTION REFERENCE
ON SHEET WHERE SECTION IS CALLED OUT



DETAIL, SECTION, AND ELEVATION NUMBER & TITLE
ON SHEET WHERE DETAIL OR SECTION IS SHOWN

PROFILE DATA BAND ON PLAN & PROFILE SHEETS



ABBREVIATIONS

CURTAILMENT/ACRONYM	DESCRIPTION
ALDOT	ALABAMA DEPARTMENT OF TRANSPORTATION
ALDOT SPECS.	ALDOT STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION, 2008 EDITION.
ASTM	AMERICAN SOCIETY FOR TESTING AND MATERIALS
@	AT
BVCS	BEGINNING OF VERTICAL CURVE STATION
BVCE	BEGINNING OF VERTICAL CURVE ELEVATION
C/L	CENTER LINE
CBMPP	CONSTRUCTION BEST MANAGEMENT PRACTICES PLAN
CL.	CLEARANCE
DIA	DIAMETER
D.I.	DROP IN
DD#	DOWN DRAIN
EL., ELEV.	ELEVATION
EVCS	END OF VERTICAL CURVE STATION
EVCE	END OF VERTICAL CURVE ELEVATION
E.W.	EACH WAY
EXP	EXPANSION
HORZ.	HORIZONTAL
I.D.	INNER DIAMETER
STN#	INSTRUMENTATION (INCLINOMETERS/PIEZOMETERS)
IRRP	INTERIM RISK REDUCTION PROJECT
INV.	INVERT
LF	LINEAL FEET
LVC	LENGTH OF VERTICAL CURVE
MAX.	MAXIMUM
MIN.	MINIMUM
N/A	NOT APPLICABLE
N.I.C	NOT IN THIS CONTRACT
N.T.S.	NOT TO SCALE
NO.	NUMBER
O.C.	ON CENTER
O.D.	OUTER DIAMETER
PVI	POINT OF VERTICAL INTERSECTION
RGF	REVERSE GRADED FILTER – SEEPAGE/PIPING ARMORING
SHEET#	WP 29 SHEET 10W519-# (#=SHEET NUMBER SO INDICATED)
SP#	SEEPAGE PIPE
STA	STATION OR STATIONING
TBD	TO BE DETERMINED
TBR&R	TO BE REMOVED AND REPLACED
TDOT	TENNESSEE DEPARTMENT OF TRANSPORTATION
TDOT SPECS.	TDOT STANDARD SPECIFICATIONS FOR ROAD & BRIDGE CONSTRUCTION, 2006 EDITION.
TYP.	TYPICAL
TVA	TENNESSEE VALLEY AUTHORITY
VERT.	VERTICAL
WCF	WIDOWS CREEK FOSSIL PLANT
WWF	WELDED WIRE FABRIC
WP	WORK PLAN

FOR SUPPORTING DESIGN CALCULATIONS SEE: CDX0000002011O004

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST
R 0	11/21/11	AGH	AN/KV	JJH	JDM	JM	MST	JOK	204372	D

SCALE: NONE EXCEPT AS NOTED

LEGEND SHEET

WORK PLAN WCF-10W519-WP-29

ASH COMPLEX IRRP-URSAC002 & IRRP-URSAC003

SLOPE STABILITY IMPROVEMENTS

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
AGH	KV	JJH	JDM	JM	MST	JOK

URS

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

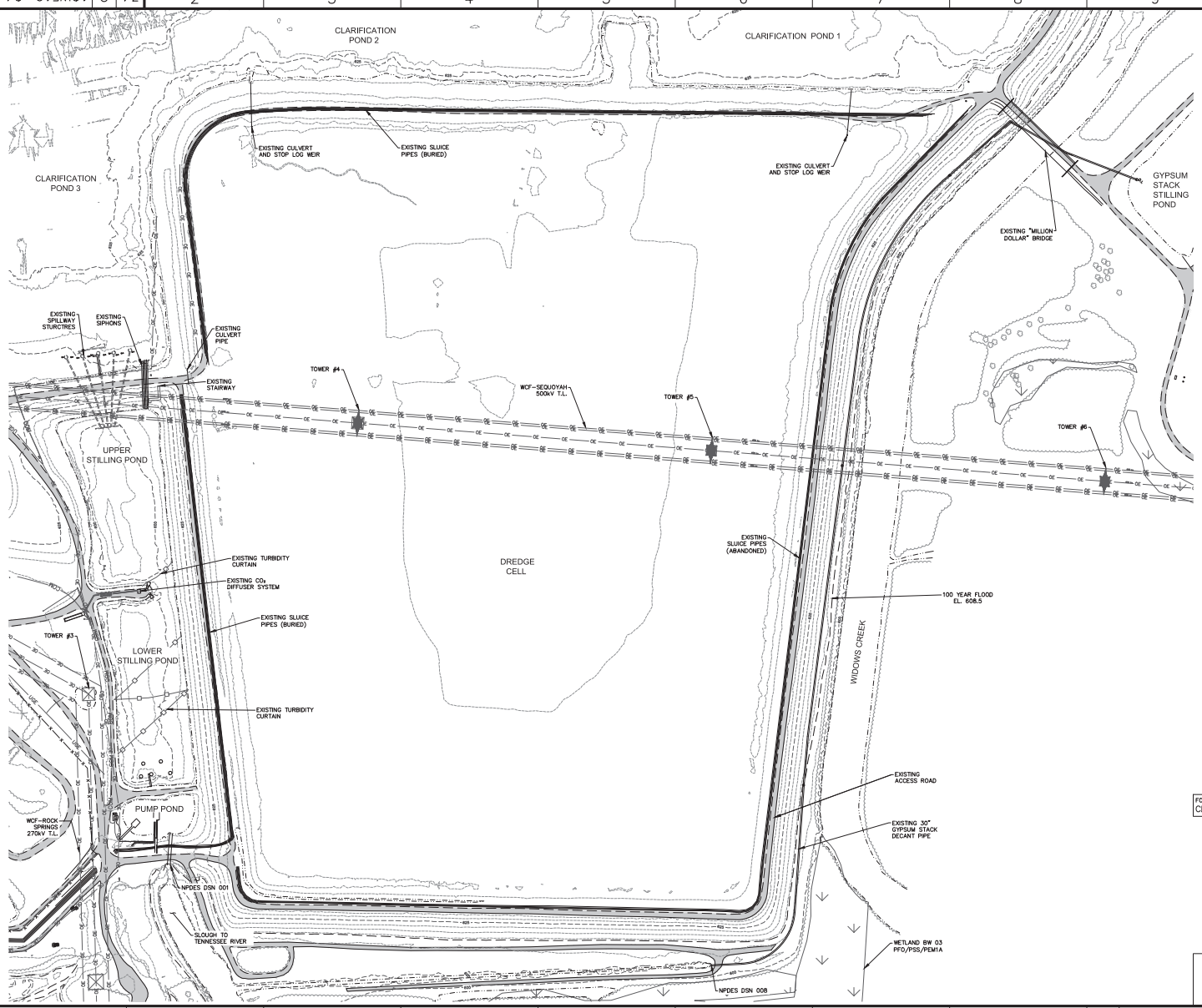
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FOR SUPPORTING DESIGN CALCULATIONS SEE: CDX000000020110004

SCALE: AS SHOWN EXCEPT AS NOTED

EXISTING CONDITIONS PLAN

WORK PLAN WCF-10W519-WP-29

ASH COMPLEX IRRP-URSAC002 & IRRP-URSAC003

SLOPE STABILITY IMPROVEMENTS

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD 2010 | 11/21/11 | 34 | C | 10W519-04 | R 0

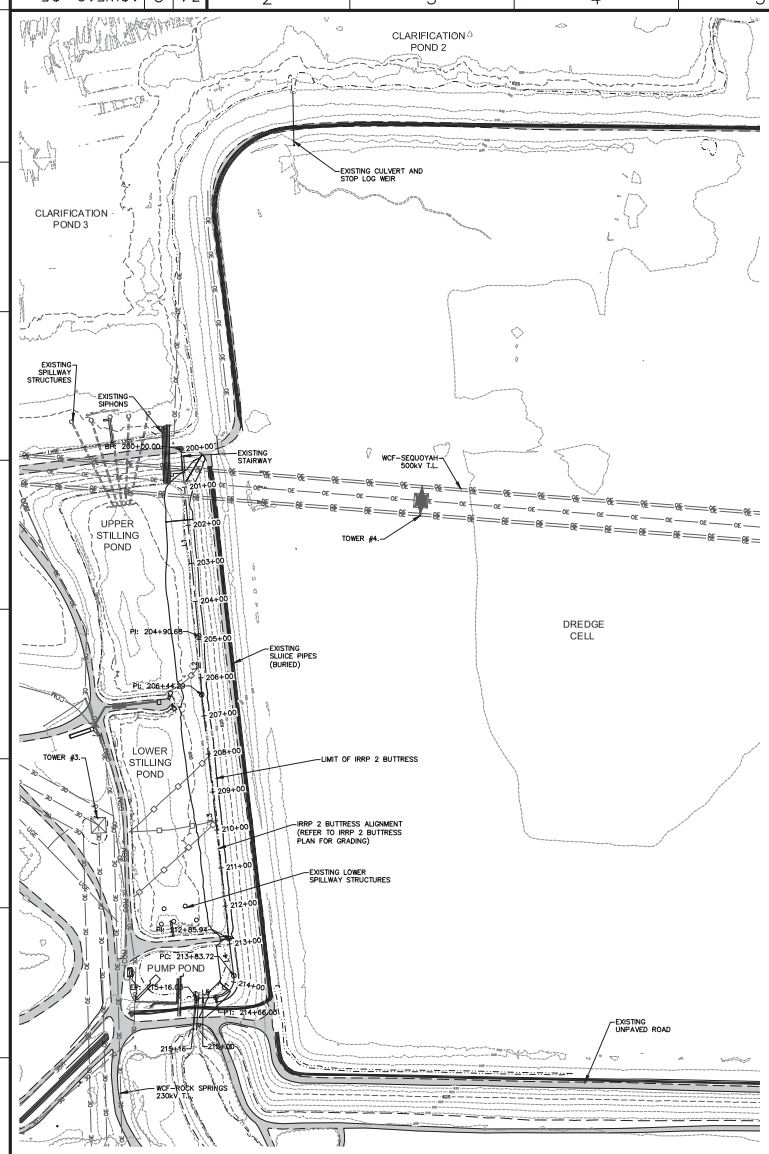
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PLOT FACTOR:XX W_TVA

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US EPA ARCHIVE DOCUMENT

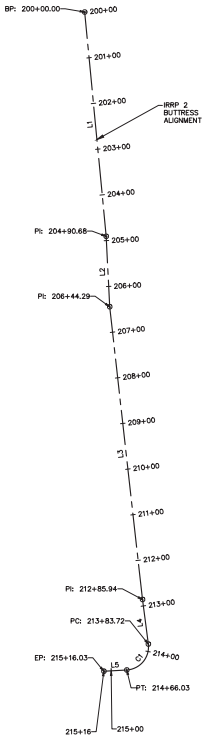


AC-IRRP 2 PLAN

SCALE IN FEET 120 0 120 240

IRRP 2 BUTTRESS ALIGNMENT GEOMETRY

NUMBER	STARTING STATION	STARTING NORTHING	STARTING EASTING	ENDING STATION	ENDING NORTHING	ENDING EASTING	LENGTH	LINE/CHORD DIRECTION	DELTA	RADIUS	TANGENT
L1	200+00.00	1598152.04	524974.8384	204+90.68	1597839.74	525353.31	490.68	S50° 28' 20.76"E			
L2	204+90.68	1597839.74	525353.3111	206+44.29	1597736.67	525467.20	153.61	S47° 51' 10.67"E			
L3	206+44.29	1597736.67	525467.2002	212+85.94	1597336.30	525968.62	641.65	S51° 23' 36.95"E			
L4	212+85.94	1597336.30	525968.6169	213+83.72	1597276.66	526046.11	97.78	S52° 25' 06.64"E			
C1	213+83.72	1597276.66	526046.11	214+66.03	1597203.65	526052.83	82.31	S5° 15' 40.63"E	94.3175	50.000	53.917
L5	214+66.03	1597203.65	526052.8304	215+16.03	1597166.43	526019.44	50.00	S41° 53' 50.79"W			



AC-IRRP 2 ALIGNMENT GEOMETRY

SCALE IN FEET 100 0 100 200

NOTES:

1. BASELINE AND TIE-IN LOCATIONS WERE DEVELOPED BASED ON 2010 TOPOGRAPHY. ACTUAL TIE-IN LOCATIONS MAY VARY FROM THOSE DEPICTED ON THE DRAWINGS.

FOR SUPPORTING DESIGN CALCULATIONS SEE: CDX00000020110004

R #											
R 0	11/21/11	AGH	AN/KV	JJH	JDM	JM	MST	JCK	204372		0

SCALE: AS SHOWN EXCEPT AS NOTED

PROPOSED CONDITIONS & ALIGNMENT – IRRP 2

WORK PLAN WCF-10W519-WP-29

ASH COMPLEX IRRP-URSAC002 & IRRP-URSAC003

SLOPE STABILITY IMPROVEMENTS

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
AGH	AN/KV	JJH	JDM	JM	MST	JCK

URS

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD 2010	DATE 11/21/11	34	C	10W519-05	R 0

URS Corporation 0
TASK COMPLETED BY: REV NO.

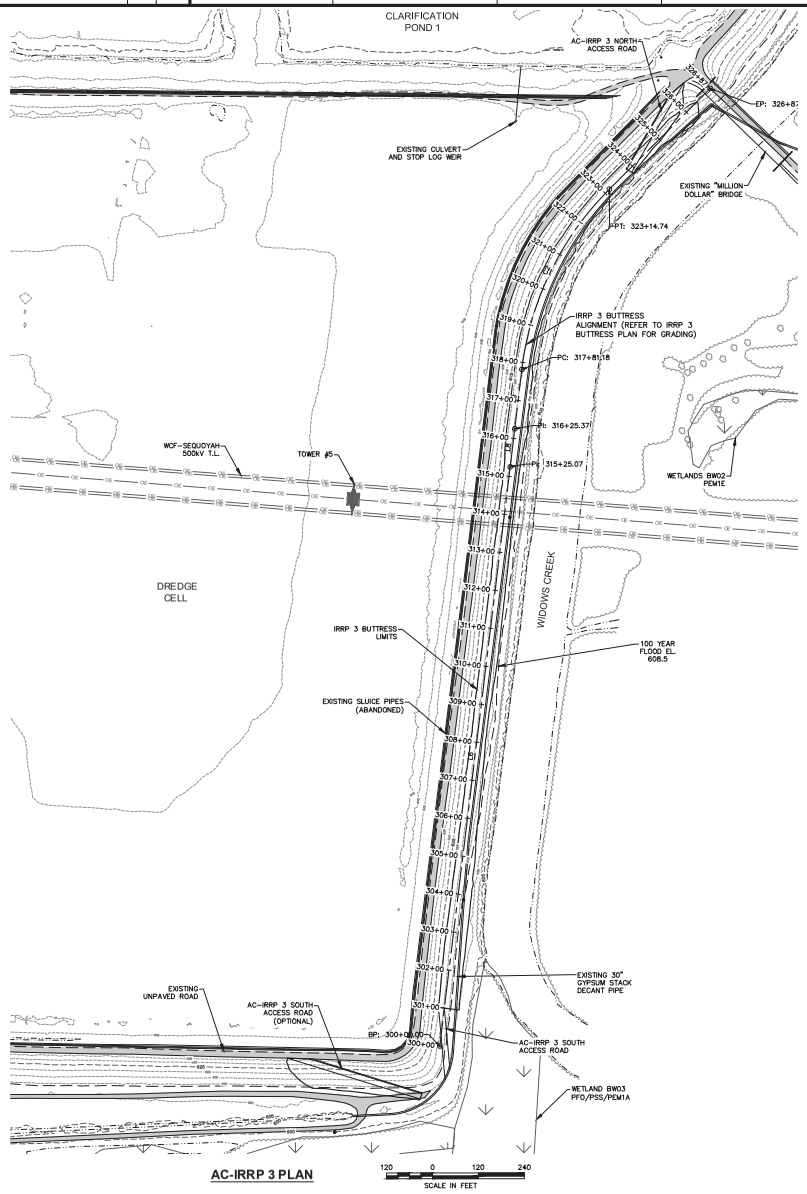
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C.A.D. DRAWING DO NOT ALTER MANUALLY

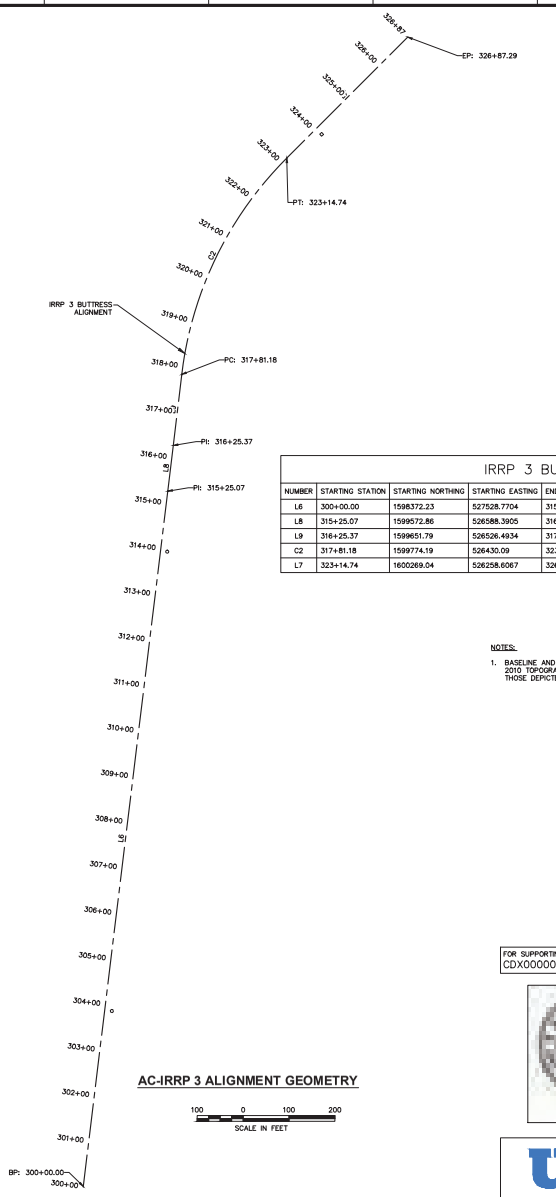
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US EPA ARCHIVE DOCUMENT



AC-IRRP 3 PLAN



AC-IRRP 3 ALIGNMENT GEOMETRY

IRRP 3 BUTTRESS ALIGNMENT GEOMETRY

NUMBER	STARTING STATION	STARTING NORTHING	STARTING EASTING	ENDING STATION	ENDING NORTHING	ENDING EASTING	LENGTH	LINE/CHORD DIRECTION	DELTA	RADIUS	TANGENT
L6	300+00.00	1598372.23	527528.7704	315+25.07	1599572.86	526588.39	1525.07	N38° 04' 09.49"W			
L8	315+25.07	1599572.86	526588.3905	316+25.37	1599651.79	526526.49	100.30	N38° 06' 14.37"W			
L9	316+25.37	1599651.79	526526.4934	317+81.18	1599774.19	526430.09	155.81	N38° 13' 31.75"W			
C2	317+81.18	1599774.19	526430.09	323+14.74	1600269.04	526258.61	533.55	N19° 06' 45.88"W	38.2255	799.740	277.134
L7	323+14.74	1600269.04	526258.6067	326+87.29	1600641.59	526258.61	372.55	N0° 00' 00.00"E			

NOTES:

1. BASELINE AND TIE-IN LOCATIONS WERE DEVELOPED BASED ON 2010 TOPOGRAPHY. ACTUAL TIE-IN LOCATIONS MAY VARY FROM THOSE DEPICTED ON THE DRAWINGS.

FOR SUPPORTING DESIGN CALCULATIONS SEE: CDX000000020110004

R #											
R 0	11/21/11	AGH	AN/KV	JJH	JDM	JM	MST	JCK	204372		0

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV NO.

SCALE: AS SHOWN EXCEPT AS NOTED

PROPOSED CONDITIONS & ALIGNMENT – IRRP 3

WORK PLAN WCF-10W519-WP-29

ASH COMPLEX IRRP-URSAC002 & IRRP-URSAC003

SLOPE STABILITY IMPROVEMENTS

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
AGH/AN	AN/KV	JJH	JDM	JM	MST	JCK

URS

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD	2010	DATE	11/21/11	34	C	10W519-06	R 0

URS Corporation	0
TASK COMPLETED BY:	REV NO.

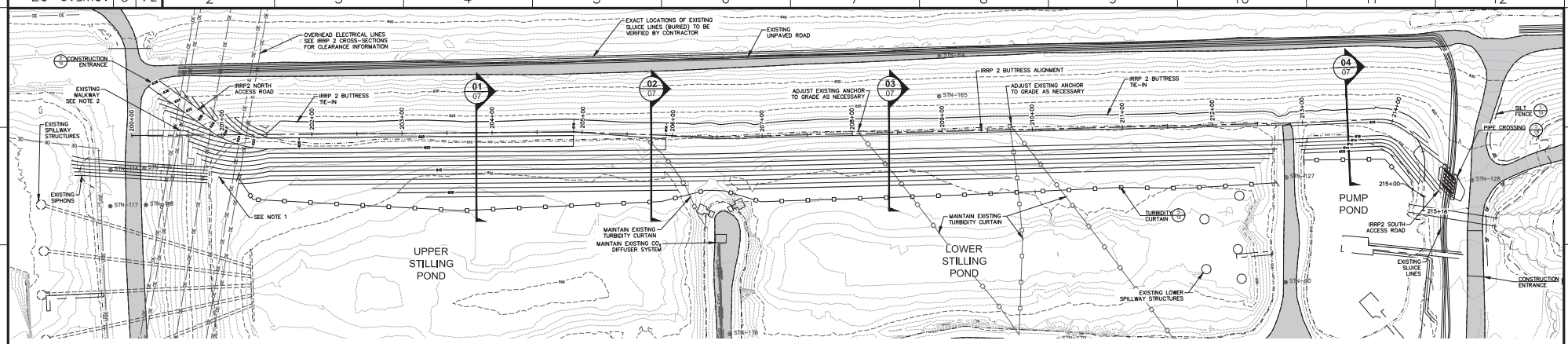
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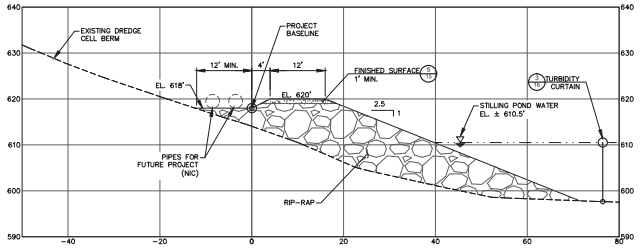
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US EPA ARCHIVE DOCUMENT

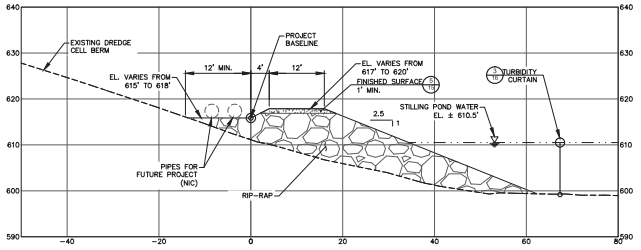


IRRP 2 TOE BUTTRESS PLAN

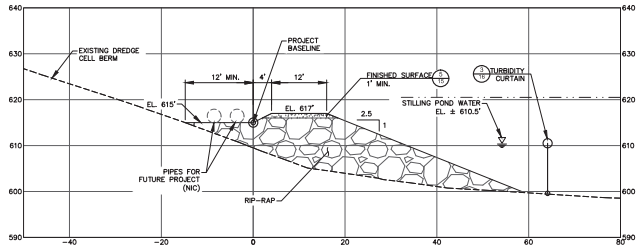
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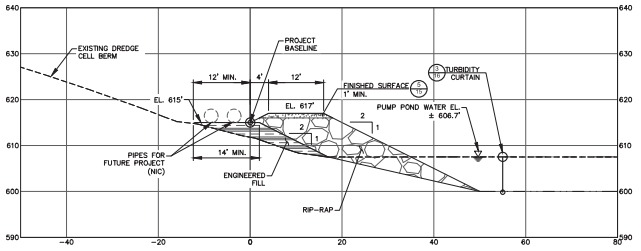
01/07 IRRP 2 TYPICAL SECTION STA. 201+85 TO STA. 204+90



02/07 IRRP 2 TYPICAL SECTION STA. 204+90 TO STA. 206+44



03/07 IRRP 2 TYPICAL SECTION STA. 206+44 TO STA. 212+85



04/07 IRRP 2 TYPICAL SECTION STA. 212+85 TO STA. 214+66

HORIZONTAL SCALE: 10 0 10 20

VERTICAL SCALE: 10 0 10 20

NOTES:

1. CONTRACTOR SHALL TIE IN BUTTRESS TO EXISTING SIPHON RIPRAP. ELEVATIONS OF TIE IN POINTS MAY VARY SLIGHTLY FROM THE LINES AND GRADES SHOWN. BUTTRESS SHALL BE INSTALLED SO AS NOT TO IMPACT SIPHON OPERATION. CONTRACTOR TO NOTIFY TVA IMMEDIATELY IF CONSTRUCTION OPERATIONS CAUSE ANY DISTURBANCE TO THE OPERATING SIPHONS.
2. CONTRACTOR SHALL GRADE BUTTRESS AROUND WALKWAY SUPPORT POST SO AS NOT TO DAMAGE THEM. IF ANY DAMAGE TO THE EXISTING WALKWAY OCCURS DURING CONSTRUCTION, CONTRACTOR IS RESPONSIBLE FOR REPAIRS.

FOR SUPPORTING DESIGN CALCULATIONS SEE: CDX000000020110004

REV	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST
R 0	11/21/11	AGH	AN/KV	JJH	JDM	JM	MST	JCK	204372	0

SCALE: NONE EXCEPT AS NOTED

IRRP 2 TOE BUTTRESS PLAN & TYPICAL SECTIONS

WORK PLAN WCF-10W519-WP-29

ASH COMPLEX IRRP-URSAC002 & IRRP-URSAC003

SLOPE STABILITY IMPROVEMENTS

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
AGH	AN/KV	JJH	JDM	JM	MST	JCK

URS

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

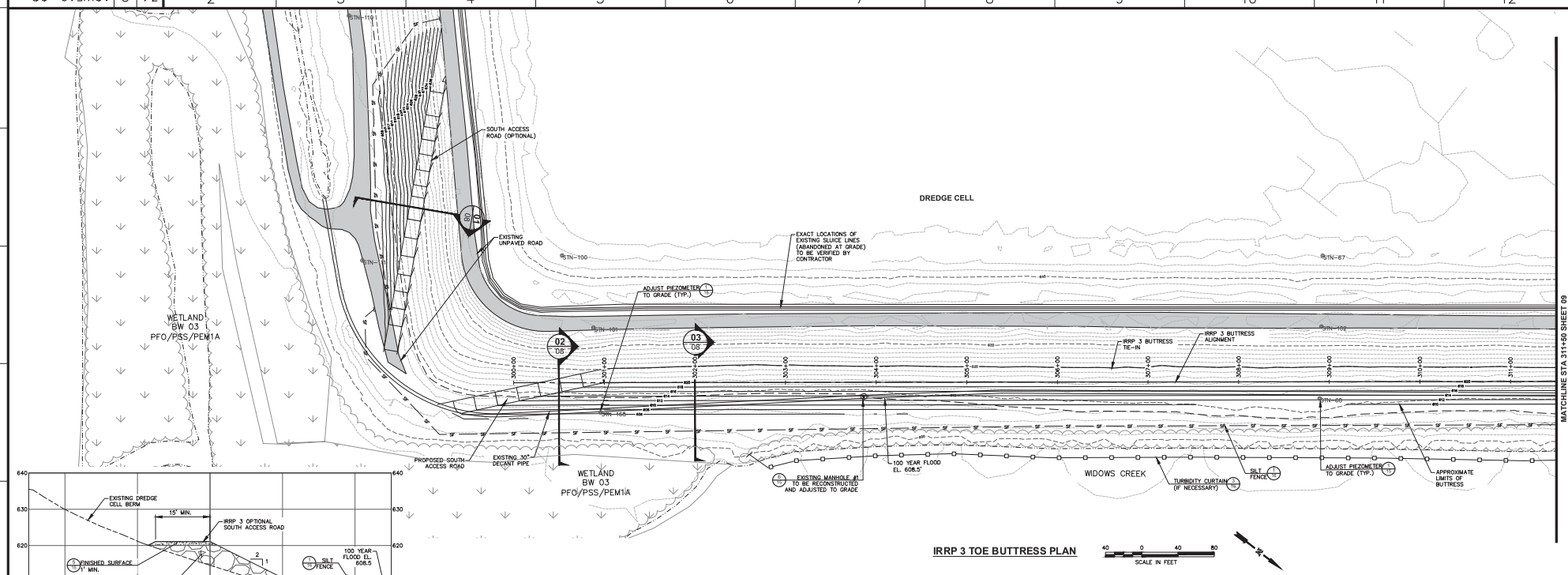
AUTOCAD 2010 | DATE 11/21/11 | 34 | C | 10W519-07 | R 0

URS Corporation | 0
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:XX W-TVA

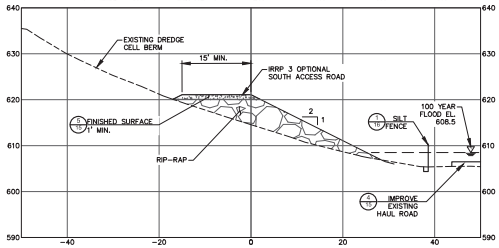
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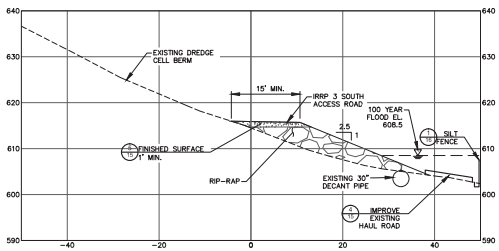


IRRP 3 TOE BUTTRESS PLAN

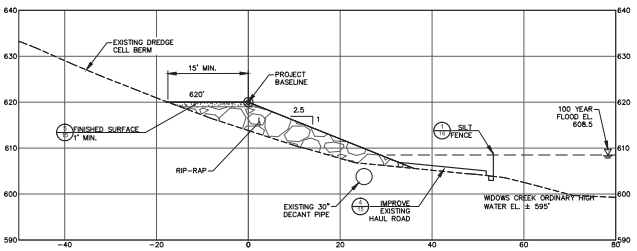
SCALE IN FEET



01/08 IRRP 3 OPTIONAL SOUTH ACCESS ROAD TYPICAL SECTION



02/08 IRRP 3 SOUTH ACCESS ROAD TYPICAL SECTION



03/08 IRRP 3 TYPICAL SECTION STA. 301+00 TO STA. 315+25

HORIZONTAL SCALE

VERTICAL SCALE

FOR SUPPORTING DESIGN CALCULATIONS SEE: CDX000000020110004

SCALE: NONE EXCEPT AS NOTED

IRRP 3 TOE BUTTRESS PLAN & TYPICAL SECTIONS I

WORK PLAN WCF-10W519-WP-29

ASH COMPLEX IRRP-URSAC002 & IRRP-URSAC003

SLOPE STABILITY IMPROVEMENTS

URS

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD 2010 | 11/21/11 | 34 | C | 10W519-08 | R 0

URS Corporation | 0
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:XX W-TVA

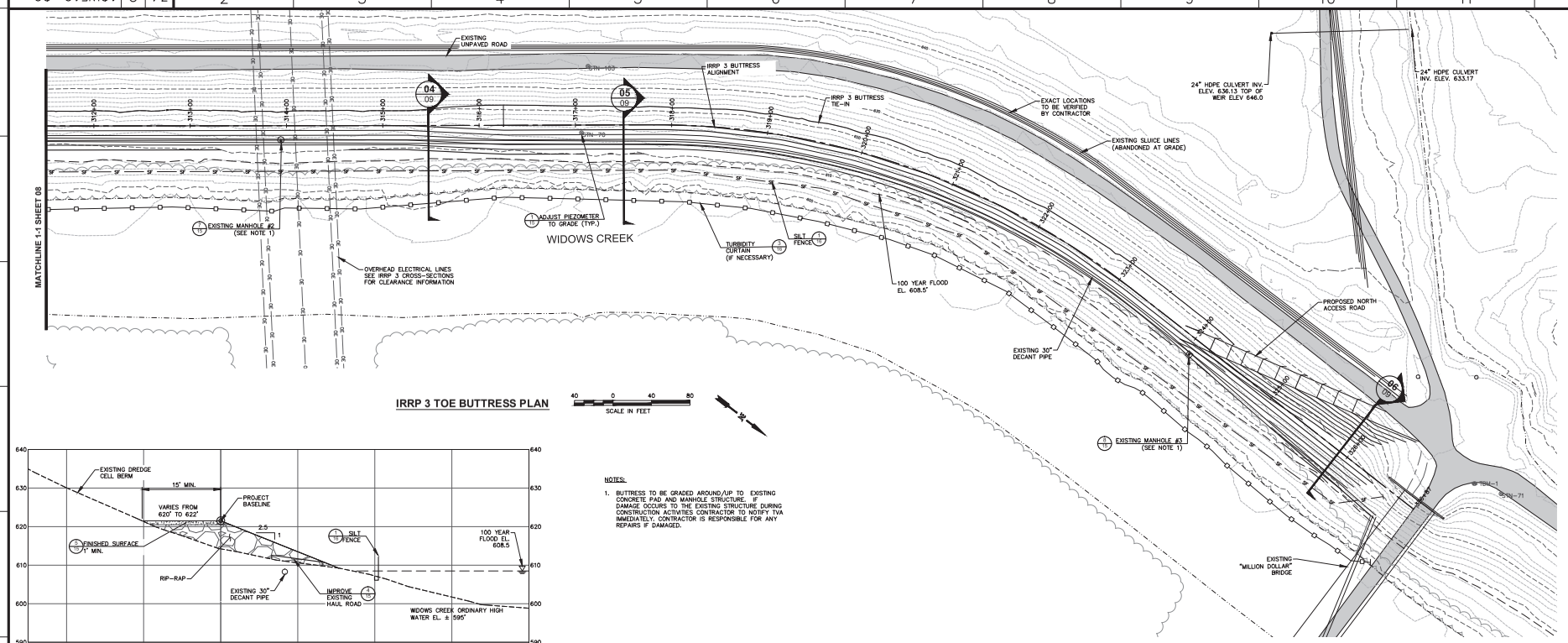
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34 C 10W519-08

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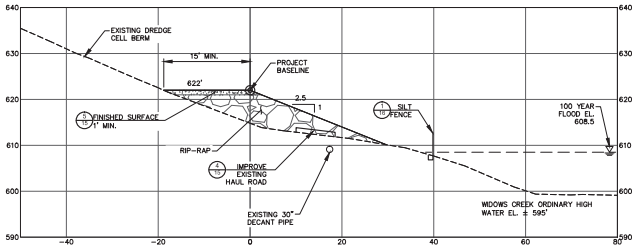
US EPA ARCHIVE DOCUMENT



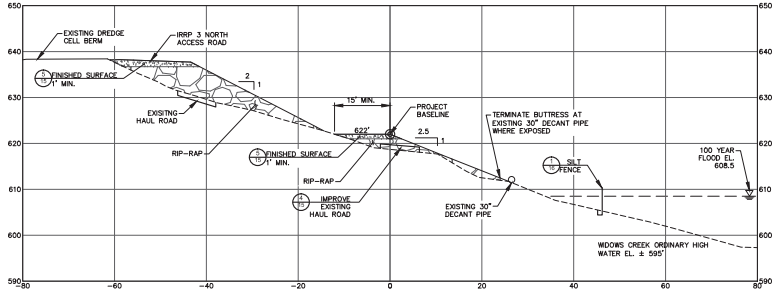
NOTES:

1. BUTTRESS TO BE GRADED AROUND/UP TO EXISTING CONCRETE PAD AND MANHOLE STRUCTURE. IF DAMAGE OCCURS TO THE EXISTING STRUCTURE DURING CONSTRUCTION ACTIVITIES CONTRACTOR TO NOTIFY TVA IMMEDIATELY. CONTRACTOR IS RESPONSIBLE FOR ANY REPAIRS IF DAMAGED.

04/09 IRRP 3 TYPICAL SECTION STA. 315+25 TO STA. 316+25



05/09 IRRP 3 TYPICAL SECTION STA. 316+25 TO STA. 325+72



06/09 IRRP 3 SOUTH ACCESS ROAD TYPICAL SECTION

HORIZONTAL SCALE

VERTICAL SCALE

FOR SUPPORTING DESIGN CALCULATIONS SEE: CDX0000002011O004

REV	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST
R 0	11/21/11	AGH	AN/KV	JJH	JDM	JM	MST	JCK	204372	0

SCALE: NONE EXCEPT AS NOTED

IRRP 3 TOE BUTTRESS PLAN & TYPICAL SECTIONS II

WORK PLAN WCF-10W519-WP-29

ASH COMPLEX IRRP-URSAC002 & IRRP-URSAC003

SLOPE STABILITY IMPROVEMENTS

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
AGH	AN/KV	JJH	JDM	JM	MST	JCK

URS

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD 2010 | DATE 11/21/11 | 34 | C | 10W519-09 | R 0

PLOT FACTOR:XX W-TVA

C.A.D. DRAWING DO NOT ALTER MANUALLY

URS Corporation | 0
TASK COMPLETED BY: | REV NO.

34 C 10W519-09

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US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 15

Basis of Design Report, IRRP #1, Clay Dike buttress, Widows Creek, Jackson County, AL, December, 2010, URS Corp.

US EPA ARCHIVE DOCUMENT

WCF IRRP-URSAC001

BASIS OF DESIGN REPORT

IRRP #1 CLAY DIKE BUTTRESS

Widows Creek Fossil Plant
Jackson County, Alabama

December 2010
Rev 0

1375 Euclid Ave., Suite 600
Cleveland, OH 44115

US EPA ARCHIVE DOCUMENT

OBJECTIVES & METHODOLOGY

IRRP-URSAC001 consists of a rock toe buttress along the northwest and northern portions of the perimeter clay dike adjacent to the Main Ash Pond at WCF, see Figure 1. This document describes the basis of design (BOD) for the proposed toe buttress to support interim risk reduction and eventual closure of the Main Ash Pond.

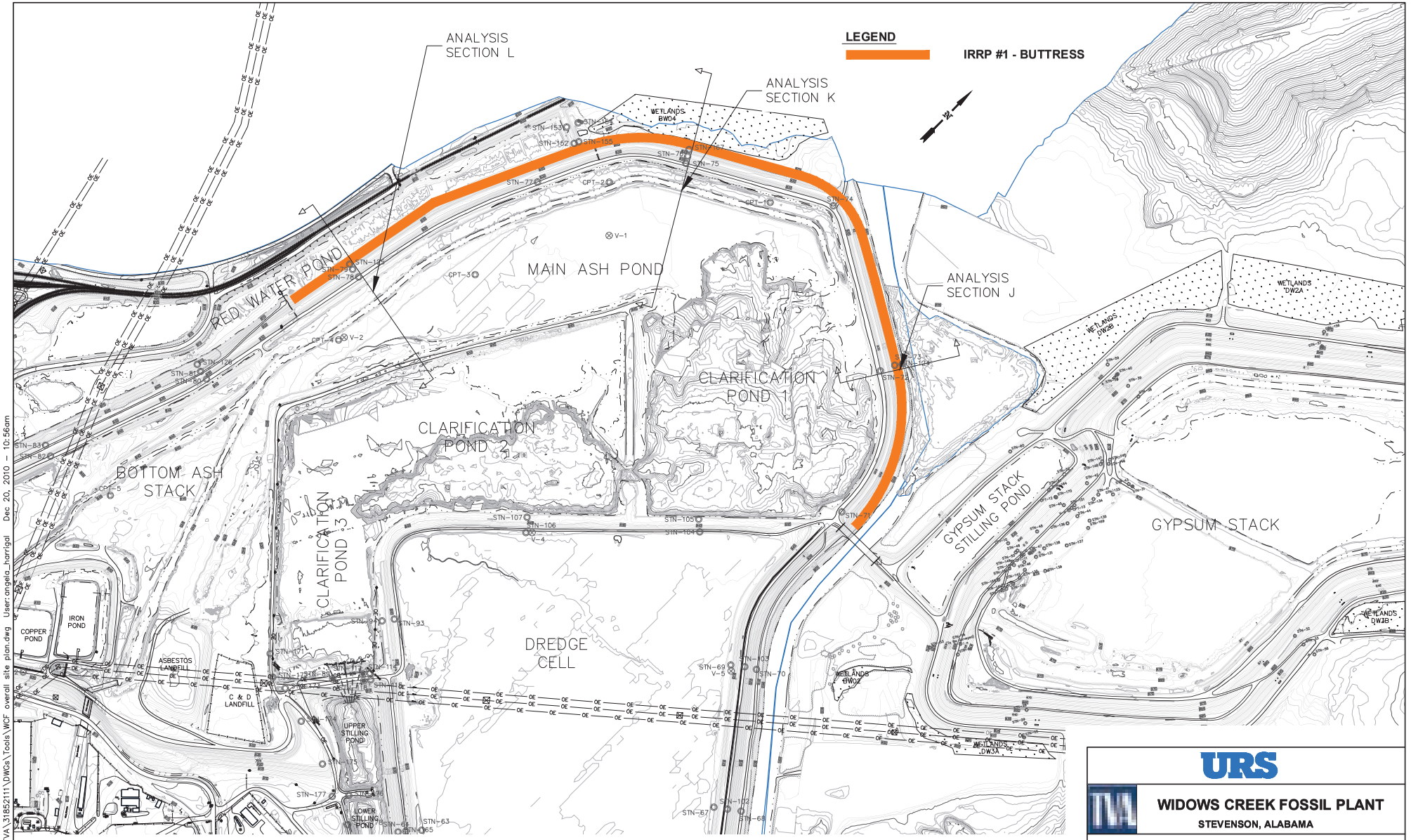
The main goal of this project is to improve the slope stability factor of safety (FS) for the perimeter clay dike at the Widows Creek Fossil Plant (WCF) along the northwest and northern portions of the Main Ash Pond. Among several options considered (e.g. stone columns, deep soil mixing, French drain), a stability buttress was recommended based on cost and constructability (see Attachment A; "Alternatives Widows Creek Fossil Plant - Main Ash Pond IRRP-URSAC001 Exterior Perimeter Clay Dike Remediation Alternatives," dated 9/24/10). This project (Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372 Work Plan WCF-100521-WP-24) is part of a series of projects within the Ash Pond Complex called Interim Risk Reduction Projects (IRRPs). IRRP-URSAC001 consists of a rock toe buttress, which includes a reverse-graded filter, extending along the length of the dike, extending from the crest of the clay dike to just past the toe. The design of the buttress is intended to increase the slope stability FS to 1.50 or greater, as required by the Master Programmatic Document, Version 1.

The development of IRRP URSAC001 was optimized through an iterative process of reducing the normal operating pool elevation and sizing the buttress to a minimal cross-section required to achieve target slope stability FS greater than or equal to 1.50, while also maintaining the required free water volume for the facility at an acceptable cost and risk level. This process is documented in the Decision Matrix Summary Memorandum (see Attachment B), dated April 12, 2010 and revised and agreed to on May 3, 2010. The final geometry of IRRP-URSAC001 meets the target FS in combination with a reduced normal operating pool elevation of 627.0.

Stability analysis of IRRP-URSAC001 was completed using GeoStudio SLOPE/W and SEEP/W Version 7.16, Build 4840. The models developed were initially inherited from Stantec Inc., developed as part of the Report of Phase 2 Geotechnical Exploration[2], and then enhanced by URS with updated geometry, material properties, and boundary conditions to complete the buttress design.

[1] Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372, Work Plan WCF-100521-WP-24
[2] Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc.

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NOTES

1. BORINGS SHOWN WERE COMPLETED BY STANTEC, INC.

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URS

TVA

WIDOWS CREEK FOSSIL PLANT
STEVENSON, ALABAMA

FIGURE 1
IRRP-URSAC001
LOCATION PLAN

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	FIGURE No:
AGH	JKH	31852111	12/2010	1

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SEEPAGE INPUT PARAMETERS

The phreatic surface was obtained based on the SEEP/W finite-element seepage modeling software. The models were calibrated with parameters to match existing, measured piezometric surfaces, and laboratory testing data from samples taken in the field[2]. The pore pressures from SEEP/W were "parented" into SLOPE/W, limit-equilibrium slope stability software, to calculate slope stability factors of safety. Table 1 shows the material properties used for the SEEP/W analysis. The Seepage Analysis Reports for each critical cross section are in Attachment D.

The toe buttress has also been deigned to provide protection against seepage/piping at the toe of the perimeter dike in the form of a reverse graded filter. The core of the toe buttress includes a minimum of 12 inches of Alabama Department of Transportation (ALDOT) concrete sand (No. 100) overlain by a minimum of 12 inches of ALDOT No. 7 stone. These materials were checked for filter compatibility according to USACE standards (see Attachment C). The balance of the buttress will be comprised of ALDOT Class 1 riprap.

Table 1 Material Properties for Seepage Analysis

	Soil-Water Retention Curve (SWRC)			Unsaturated Hydraulic Conductivity Function				
Material	SWRC Sample Function	Saturated VWC (%)	Residual VWC (%)	K_v-sat (cm/sec)*	K-Function Estimation Method	K_h-sat (cm/sec)	K_h-sat (ft/sec)	Cond. Ratio K_h/K_v
Sand with Gravel (Bottom Ash)	Silty Sand	15.0%	1.0%	1.00E-03	Fredlund & Xing	1.00E-02	3.28E-04	10
Silty with Sand (Bottom Ash)	Silty Sand	50.0%	1.0%	1.00E-03	Fredlund & Xing	1.00E-02	3.28E-04	10
Fill Clay with Gravel (Embankment Fill)	Clay	25.0%	3.0%	1.00E-07	Fredlund & Xing	4.00E-07	1.31E-08	4
Residual Fat Clay	Clay	25.0%	3.0%	3.60E-07	Fredlund & Xing	3.60E-06	1.18E-07	10
Silt (Fly Ash)	Silt	40.0%	2.0%	4.00E-06	Fredlund & Xing	4.00E-05	1.31E-06	10
Stability Buttress	Gravel	39.0%	1.0%	1.01E+00	Fredlund & Xing	1.01E+00	3.31E-02	1

*Ref: Based on STANTEC's calibrated seepage models and subsequently modified by URS parametric study.

STABILITY INPUT PARAMETERS

The laboratory data from Stantec's Report of Phase 2 Geotechnical Exploration was reviewed, and the values for angles of internal friction (Phi,), cohesion (c), and unit weights (Gamma,) were updated. The values used for slope stability analysis and are summarized below in Table 2.

[1] Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372, Work Plan WCF-100521-WP-24
[2] Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc.

Table 2 Material properties for slope stability analyses

Material	Angle of Internal Friction, Drained, ' (deg.)	Cohesion, C (psf)	Unit Weight, (pcf)
Sand with Gravel (Bottom Ash)	33.0	0	119
Silt with Sand (Bottom Ash)	33.0	0	112
Fill Clay with Gravel (Embankment Fill)	29.0	150	125
Residual Fat Clay	33.0	50	125
Silt (Fly Ash)	27.5	0	112
Stability Buttress	40.0	0	115

The material properties for the Fill Clay with Gravel, Residual Fat Clay, and Silt listed in Table 3 were updated from those presented in STANTECS Phase II report[2], as well as new data obtained from additional testing completed in the summer of 2010. New effective and total stress envelopes were developed and new Phi, and c values calculated. A $^2/_3$ failure envelope of the plotted values was used to determine Phi and c, see Figures 2, 3, and 4 and the associated calculations.

The remaining material properties, for Sand with Gravel and Silt with Sand were carried over from Stantec's Analysis[2]. The material properties for the Stability Buttress were developed using engineering judgment and typical values for similar material found in Alabama.

[1] Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372, Work Plan WCF-100521-WP-24
[2] Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc.

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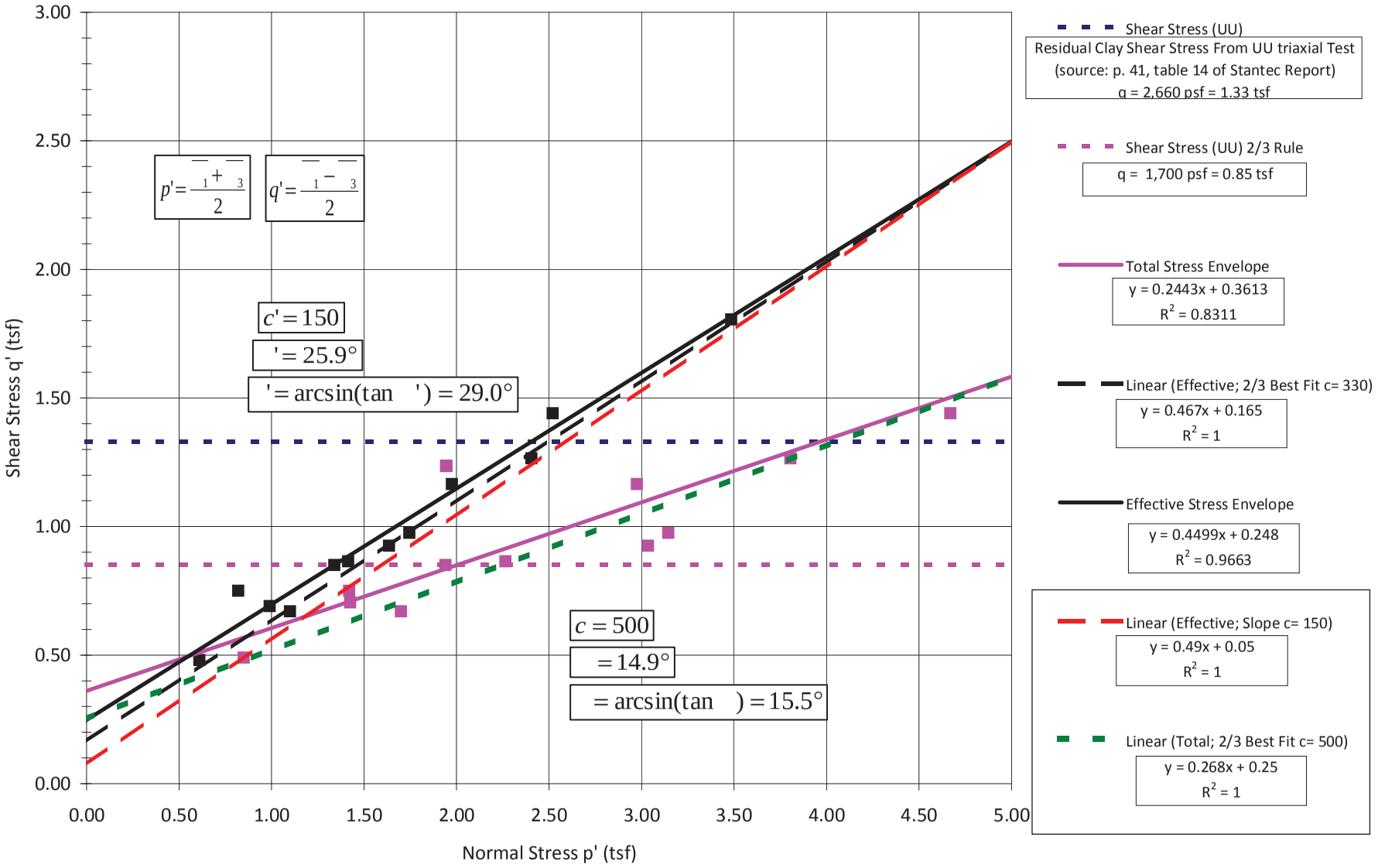
WCF Material Properties Analysis
Calculated: AGH
Checked: JJH

12/27/2010

Figure 2

Clay Fill (linear trend with cohesion)

Effective/Total Stress from CU Triaxial Test[1]



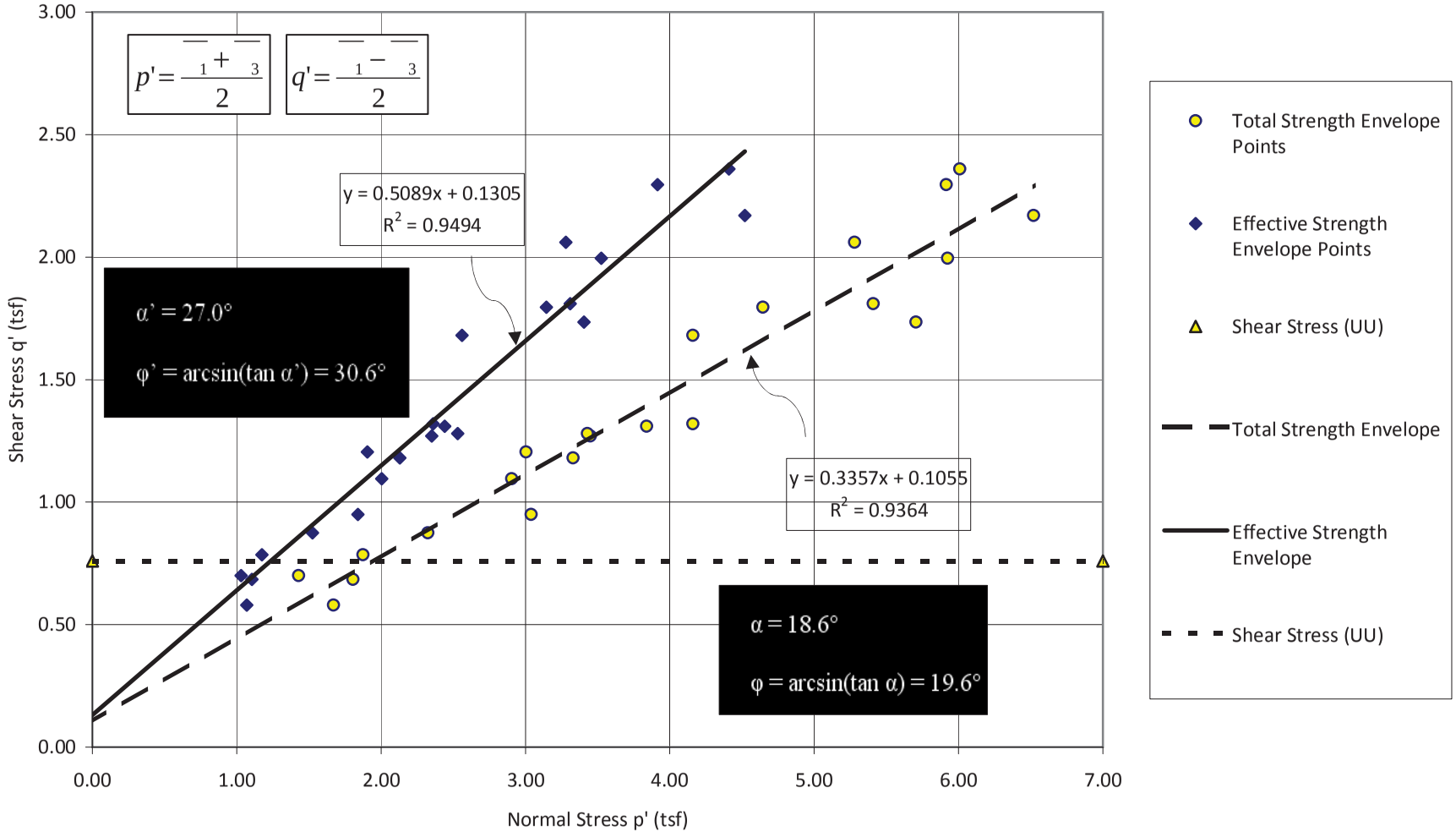
[1]Raw data taken from Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc., and additional labratory testing completed Aug. 2010.

WCF Material Properties Analysis
Calculated: ANR
Checked: JJH

12/27/2010

Figure 3

Residual Clay (linear trend with cohesion)
Effective/Total Stress from CU Triaxial Test[1]

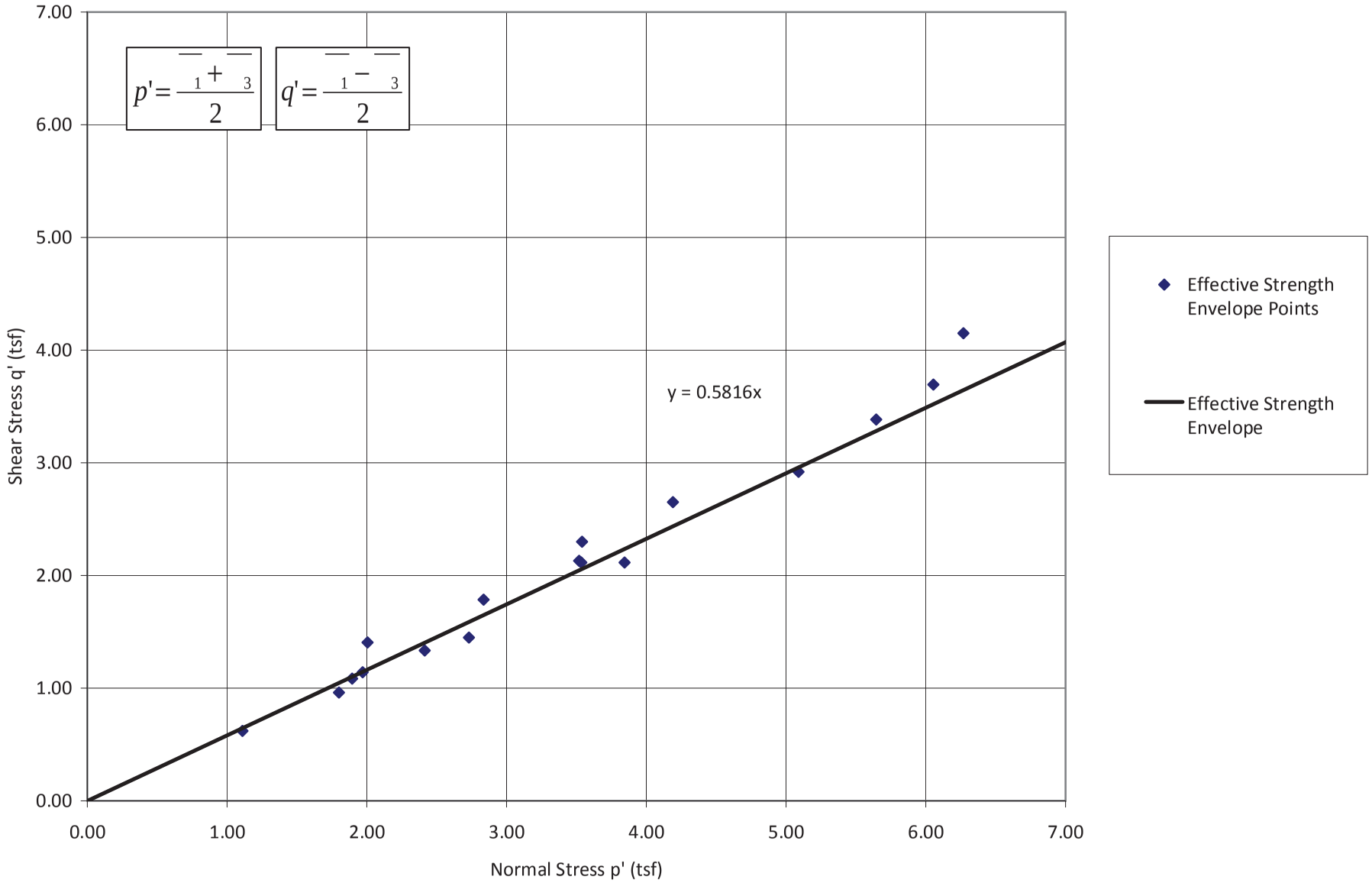


[1]Raw data taken from Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc., and additional labratory testing completed Aug. 2010.

WCF Material Properties Analysis
Calculated: ANR
Checked: JJH

12/27/2010

Figure 4
Fly Ash (linear trend with zero cohesion)
Effective Stress from CU Triaxial (R) Test[1]



[1]Raw data taken from Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc., and additional labratory testing completed Aug. 2010.

Effective Stress Calculations:

$$\tan(\ ') = \frac{y}{x} \qquad '= \arcsin(\tan\ ')$$

Effective from chart c= 496		
y = 0.4499x + 0.248		
for x=	0	5
y=	0.248	2.4975
α' =	24.2	
Φ' =	26.7	

Effective; Equation set c=0		
y=0.5718x		
for x=	2	5
y=	1.1436	2.859
α' =	29.8	
Φ' =	34.9	

Effective; Slope c= 150		
y = 0.485x + 0.075		
for x=	0	5
y=	0.075	2.5
α' =	25.9	
Φ' =	29.0	

Effective; 2/3 Best Fit c= 330		
y = 0.467x + 0.165		
for x=	0	5
y=	0.165	2.5
α' =	25.0	
Φ' =	27.8	

*Units for cohesion are in psf unless otherwise noted.

Total Stress Calculations:

$$\tan(\ \ ) = \frac{y}{x} \qquad = \arcsin(\tan\ \ )$$

Total from chart c= 723		
y = 0.2443x + 0.3613		
for x=	2	5
y=	0.8499	1.5828
α =	13.7	
Φ =	14.1	

Total; Equation set c=0		
y=0.3542x		
for x=	2	5
y=	0.7084	1.771
α =	19.5	
Φ =	20.7	

Total; 2/3 Best Fit c= 500		
y = 0.267x + 0.25		
for x=	0	5
y=	0.25	1.585
α' =	14.9	
Φ' =	15.5	

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WCF Material Properties Analysis
Calculated: ANR
Checked: JJH

Residual Clay
Phi and Cohesion Calculations

12/27/2010

Effective Stress Calculations: $\tan(\ ') = \frac{y}{x}$ $'= \arcsin(\tan\ ')$

Effective from chart c= 261		
y=0.5089x+0.1305		
for x=	0	5
y=	0.1305	2.675
α' =	27.0	
Φ' =	30.6	

Effective; Equation set c=0		
y=0.5533x		
for x=	2	5
y=	1.1066	2.7665
α' =	29.0	
Φ' =	33.6	

Total Stress Calculations: $\tan(\ ) = \frac{y}{x}$ $= \arcsin(\tan\ )$

Total from chart c= 211		
y=0.3357x + 0.1055		
for x=	2	5
y=	0.7769	1.784
α =	18.6	
Φ =	19.6	

Total; Equation set c=0		
y=0.359x		
for x=	2	5
y=	0.718	1.795
α =	19.7	
Φ =	21.0	

*Units for cohesion are in psf unless otherwise noted.

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Effective Stress Calculations:

$$\tan(\ ') = \frac{y}{x}$$

$$'= \arcsin(\tan\ ')$$

Effective; Equation set c=0		
y=0.5816x		
for x=	2	5
y=	1.1632	2.908
α' =	30.2	
Φ' =	35.6	

[1]Fly ash is assumed to be cohesionless.

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STABILITY ANALYSIS

Three critical cross-sections; J, K, and L were analyzed using SEEP/W and SLOPE/W (See Figure 1 for their locations). The sections were analyzed using SLOPE/W 2007 electronic files provided by Stantec Inc., and then modified by URS updating cross-section geometry, material properties, and boundary conditions, as well as adding the final buttress design.

The configuration of the stability buttress is depicted on the stability analysis Figures in Attachment D, as well as Sheets 3, 4 and 5 of the design drawings[1]. The minimum bench elevation for the toe buttress was held constant at elevation 608.0, with a minimum bench width of 11-ft and a minimum thickness of 3.7-ft (modeled with a 3.5-ft minimum thickness). Outside slopes are graded at 3H:1V. Application of the standard IRRP geometry to the critical cross sections yielded the results shown in Table 4 for effective stress analysis. The Stability Analysis Reports for each critical cross section are also in Attachment D. Total Stress Analysis was also completed to verify that the undrained strength of the clay can support the stability buttress.

Table 4 Stability Analysis Results

Analysis Section / Description	Ash Pond Pool Elev.	Factor of Safety (Effective Stress)	Factor of Safety (Total Stress)
Section J (Existing Condition)	632.0	1.22	
Section J (After Dredging/Drawdown)	627.0	1.41	
Section J (After IRRP Constructed)	627.0	1.69	1.42
Section J (After IRRP Constructed)	629.0	1.62	
Section K (Existing Condition)	632.0	1.36	
Section K (After Dredging/Drawdown)	627.0	1.50	
Section K (After IRRP Constructed)	627.0	1.68	1.50
Section K (After IRRP Constructed)	629.0	1.62	
Section K (After ROAD is Constructed)**	627.0	1.31	
Section L (Existing Condition)	632.0	1.22	
Section L (After Dredging/Drawdown)	627.0	1.37	
Section L (After RWP Rapid Drawdown)*	627.0	1.33	
Section L (After IRRP Constructed)	627.0	1.69	1.45
Section L (After IRRP Constructed)	629.0	1.61	

*Note: For this analysis two piezometric surfaces were manually drawn into the stability model for analysis.

**Note: 450.0 pcf surcharge loading used for temporary truck loading.

[1] Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372, Work Plan WCF-100521-WP-24
[2] Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc.

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SEQUENCE OF CONSTRUCTION

As depicted in the design drawings[1], the toe buttress will be installed in three construction phases. Phase 1 and 3 are along the northwest side of the Main Ash Pond, adjacent to the Red Water Pond. Phase 2 runs along the northern perimeter of the Main Ash Pond, parallel to Widows Creek. Details of the construction phasing can be found in the notes on Sheet 10 of the design drawings[1].

Access improvements are also proposed to facilitate construction of the toe buttress. Construction of a temporary isolation berm is proposed to allow more direct access from County Road 96 and isolate portions of the Red Water Pond for dewatering. Construction of an access road near the existing bridge (crossing to the Gypsum Stack) is also proposed to allow access to the toe of slope from the existing perimeter access road around the Main Ash Complex. Temporary access, utilizing crane mats or other approved materials, is anticipated to be incorporated into the project as necessary in the field.

The aforementioned toe buttress phasing is based on the proposed construction access and environmental permitting requirements to facilitate initiation of work in areas that are unaffected by USACE 401/404 water quality and wetland permits. A “one-time” discharge of water impounded in the Red Water Pond, via outfall DNS 007, will be necessary to construct the temporary isolation berm and Phase 1. The entirety of IRRP URSAC001 will also be constructed within and below the 100-Yr. floodplain. Permitting and any conditional requirements are being coordinated by TVA. Construction of the proposed to buttress requires approximately 20,000 cubic yards of Class 1 riprap, 12,500 cubic yards of No. 7 stone, and 12,000 cubic yards of concrete sand. The Contractor should refer to typical sections, details and notes on Sheet 6 of the design drawings for detailed construction requirements and suggested sequence.

[1] Slope Improvements/Clay Dike Buttress IRRP-URSAC001, TVA Project No. 204372, Work Plan WCF-100521-WP-24
[2] Report of Phase 2 Geotechnical Exploration, Ash Pond Complex Widows Creek Fossil Plant, Stevenson, AL, Feb. 4, 2010, Stantec Consulting Services Inc.

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Attachment A

Ash Pond Complex Alternatives IRRPs Memorandum

1375 Euclid Avenue
Cleveland, OH 44115-1808
Tel: 216.622.2400
Fax: 216.622.2428
Architectural and Engineering Services

MEMORANDUM

TO:	Jim Blackburn, TVA Shawn Keef, TVA	DATE:	September 24, 2010
BY:	Jay Mokotoff, PE Jeremy Heyerly, PE	PROJECT:	TVA WCF Main Ash Complex
RE:	Widows Creek Fossil Plant - Main Ash Pond IRRP-URSAC001 Exterior Perimeter Clay Dike Remediation Alternatives		

URS Corporation (URS) has recommended an interim risk reduction project (IRRP-URSAC001) to support remediation of the existing exterior perimeter clay dike and eventual closure of the Main Ash Pond Complex at the Widows Creek Fossil Plant. The preferred IRRP-URSAC001 (IRRP #1) design consists of a rock toe buttress and reverse graded filter (Rock Toe Buttress) designed to manage seepage and increase the global slope stability safety factors to 1.5 or higher. However, the proposed rock toe buttress encroaches into both existing wetlands and the 100-year flood elevation of the Tennessee River. TVA has determined that permitting of wetlands will be required (permitting to construct in the floodplain will not be required) if the rock toe buttress and reverse graded filter design is constructed. In addition to the Rock Toe Buttress alternative, URS has evaluated six other remediation alternatives to establish if an alternative remediation method, that reduces wetland encroachment, can be effectively utilized at the site. In addition, an overall evaluation of each of the seven alternatives has been conducted with regard to remedial effectiveness, encroachment into wetlands, constructability, construction duration, durability, and cost.

As discussed and agreed with TVA during early project meetings, interim risk reduction project (IRRP) designs will be developed based on the current wet-sluice operating methods, thereby taking into account current potentiometric conditions within the Ash Pond Complex and perimeter clay dike materials. While the future operating methods will be dry stacking, resulting in the eventual lowering of the potentiometric surface within the Ash Pond Complex, these conditions were conservatively not considered in the IRRP #1 design nor in the evaluation of alternatives.

The following alternatives have been evaluated:

1. Lowering the Normal Pool Elevation / Ash dredging
2. Rock Toe Buttress (preferred IRRP-URSAC001 design)
3. Deep-Soil Mixing
4. Vibro-Stone Columns
5. Soil Nailing
6. French Drains
7. Bentonite Slurry Seepage Cutoff Wall

NOTE: The Rock Toe Buttress design (Alternative 2) has been optimized for implementation in conjunction with a reduction in the Main Ash Pond normal pool elevation from its current elevation of 632-ft. to 627-ft., as documented in the Decision Matrix Summary Memorandum, by URS Corporation

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dated May 3, 2010. The cost associated with reduction in normal pool elevation is inherent for every alternative, as lowering the normal pool elevation provides some benefits to each of the alternatives.

URS has also prepared preliminary cost estimates, conceptual drawings of each remediation method, and a matrix summarizing each option, all of which are attached.

A discussion of each alternative for the Main Ash Pond perimeter clay dike follows:

1. Lowering the Normal Pool Elevation / Ash Dredging

Global slope stability and seepage safety factor improvements in the Main Ash Pond are most immediately achieved by reducing the normal operating pool elevation. However, reducing the normal operating pool elevation will adversely affect the free water volume. Hydraulic dredging of fly ash from the Clarification Ponds can be utilized to offset free water volume loss associated with pool reduction.

Reduction of the normal pool elevation (alone, without addition of other remediation methods) by approximately 10-ft, from elev. 632.0- to 622.0-ft, can satisfactorily increase global slope stability safety factors for the Main Ash Pond perimeter dike. Prior to drawdown, hydraulically dredging of approximately 996,300 CY of fly ash would be required to maintain adequate free water volume within the Main Ash Pond. Dredged materials from the Main Ash Pond would be piped, processed and subsequently transported (on-site or off-site) for dry placement.

A large amount of material removal is required to reduce the water level within the clay dike to a point where slope instability is no longer an issue. However, the relatively high cost of dredging ($8.85/CY) and the operational buffer volume of 150,000 CY (in addition to the minimum required free water volume of 535,000 CY) limit the feasibility of significant reductions in the normal pool elevation. The estimated cost of the dredging is $10,650,000 and the estimated construction duration is 7 to 8 months.

The advantages of pool reduction/ash dredging include: (1) no rock toe buttress or wetland disturbance; and (2) reductions in the normal operating pool elevation, immediately improving risk and factor of safety.

In summary, lowering the normal pool elevation with ash dredging, without addition of other remediation methods, from the Main Ash Pond is a viable alternative. However, the high cost of ash dredging alone and resulting operational constrains for on-going fly ash removal operations make ash dredging and dewatering unrealistic as the sole means for stability improvements.

2. Rock Toe Buttress (preferred IRRP-URSAC001 design, referred to as IRRP #1)

The Rock Toe Buttress (IRRP #1) consists of a reverse-graded filter for seepage control overlain with a crushed stone buttress to increase global slope stability. The Rock Toe Buttress has a total linear length of approximately 5,460 feet. Our analyses indicate the rock toe buttress can increase the slope stability factor of safety to an acceptable level of 1.5 or greater. The rock toe buttress has been optimized for implementation in conjunction with a reduction in the Main Ash Pond normal pool elevation from its current elevation of 632-ft. to 627-ft., as documented in the Decision Matrix Summary Memorandum, by URS Corporation dated May 3, 2010. The total estimated cost for rock toe buttress construction is

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$1,950,000 and the estimated duration is 2 to 3 months. The estimated construction costs and durations for areas encroaching on wetlands on the north side of the Ash Pond Complex are $2890,000 and 1 month, respectively.

The rock toe buttress is advantageous for several reasons: (1) it can be constructed easily with minimal site preparation and without the use of specialty contractors; (2) it is the least expensive remedial option; (3) it has the shortest construction duration; and (4) it provides a low failure risk level both during construction and after completion. However, the rock toe buttress encroaches into existing wetlands, leading to permitting requirements with regulatory agencies, which may substantially delay construction.

3. Deep Soil Mixing

The deep soil mixing method is a ground improvement technique commonly used to improve the shear strength and bearing capacity of weak soils. Deep soil mixing is performed by mixing Portland cement and soil by advancing a large auger (typically 3 to 5 feet in diameter) into the soil. The auger is retrieved, leaving a column of soil with higher shear strength and reduced compressibility. Deep-mixed columns are typically installed in panels of several columns in width, with typical replacement ratios of 30%, meaning 30% of the soil within the deep-mixed zone has been improved through deep mixing.

Based on evaluations of this technology for similar applications, typical analyses show that deep soil mixing can increase the stability of the clay perimeter dike to an acceptable factor of safety of 1.5 or greater. Detailed design and analysis would be necessary to determine if additional seepage control measures need to be taken. This may be accomplished by placing a small reverse-graded filter on the face of the clay dike; however the filter could potentially still impact wetlands. The total estimated cost of deep soil mixing and a small reverse graded filter is $5,790,000 and the estimated construction duration is 6 to 7 months using 4 rigs. The estimated construction costs and durations for areas encroaching on wetlands are $1,160,000 and 1 to 2 months, respectively.

One advantage of deep soil mixing is that the repair itself does not encroach into wetlands (if additional seepage control is not necessary).

However, deep soil mixing has multiple disadvantages. Specialty contractors are required to perform the deep mixing, rather than a local contractor. Additionally, deep mixing rigs needed for this project typically weigh 50 to over 100 tons. As the perimeter clay dike is marginally stable under current conditions, the placement of the deep mixing rig on the clay dike would require the construction of a working pad and toe stability berm, which would encroach into existing wetlands, leading to the same disadvantages as preferred alternative IRRP #1, at a higher cost.

Additionally, deep soil mixing typically produces large amounts of spoil, which is a potentially flowable material consisting of water, Portland cement and soil. The soil would have to be carefully controlled to prevent contamination of the adjacent wetlands.

Furthermore, test sections are typically needed to validate mix design and column layouts for deep mixing. The added expense and construction duration of a test section have not been included in the estimated cost.

In summary, deep soil mixing is not a viable alternative to the rock toe buttress for this project. Deep soil mixing has all of the disadvantages of the preferred IRRP #1, along with additional disadvantages and a higher cost.

4. Vibro-Stone Columns

Vibro-stone columns are a ground improvement technique where a probe is inserted into the soil and retrieved while crushed stone is added as backfill. The probe vibrates during retrieval, densifying the crushed stone and surrounding soil. Columns are typically installed on a square or triangular grid pattern with a spacing of 5-10 feet and replacement ratios of 25-30%. Vibro-stone columns provide an increase in shear strength and a reduction in compressibility to treated soils.

Based on evaluations of this technology for similar applications, typical analyses show that vibro-stone columns can increase the slope stability factor of safety to an acceptable level of 1.5 or greater when installed within the perimeter clay dike. A small reverse-graded filter on the face of the clay dike would also be required to control seepage. The total estimated cost of the vibro-stone columns and the small reverse graded filter is $8,050,000 and the estimated construction duration is 4 to 5 months using 3 rigs. The estimated construction costs and durations for areas encroaching on wetlands are $1,580,000 and 1 to 2 months, respectively.

The advantages of vibro-stone columns include: (1) the final repair does not encroach into existing wetlands; and (2) the construction duration is relatively short if multiple vibro-stone column rigs are used.

However, vibro-stone columns have multiple disadvantages, many of which are the same as deep-soil mixing. First, a specialty contractor would need to be hired to install the columns rather than a local contractor. Second, a work pad and stability buttress would also be required for the clay dike, leading to the same permitting issues as the preferred alternative IRRP #1. Third, the replacement ratio required to improve dike stability is 40%, which is the upper bound of replacement ratios typically used for vibro-stone columns. The high replacement ratio may result in considerable ground heave during column installation, complicating the construction process. Additionally, the soils at the site are stiff enough that the columns may need to be pre-drilled, adding additional expense.

In summary, vibro-stone columns are not a viable alternative to the rock toe buttress for this project. Vibro-stone columns have all of the disadvantages of the preferred alternative IRRP #1, along with additional disadvantages and a higher overall cost.

5. Soil Nailing

Soil nails are steel bars grouted into angled holes drilled into a slope. The nails are anchored to an exterior shotcrete facing, and stabilize the slope by providing mechanical resistance across potential slip surfaces. Soil nails are commonly used to stabilize excavations, cut slopes, and existing slopes. Geotextile drainage strips are typically installed beneath the shotcrete facing to handle seepage from the slope.

Based on evaluations of this technology for similar applications, typical analyses show that soil nails installed within the clay dike can increase the slope stability factor of safety to an acceptable level of 1.5

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or greater. A final shotcrete facing would also have to be applied to the stabilized slope. The total estimated cost of the soil nails is $11,200,000 and the estimated construction duration is 8 to 9 months using 3 crews. The estimated construction costs and durations for areas encroaching on wetlands are $2,601,000 and 2 to 3 months. The high cost and long construction duration are due to the large number of soil nails required (approximately 7,644) and the placement of two shotcrete facings across the face of the entire clay dike, roughly 30,333 square yards in area.

The advantages of soil nails include: (1) the final repair does not encroach into wetlands and (2) a temporary construction access work pad could be smaller due to the much lighter weight of installation equipment.

However, the cost of soil nails is quite high, over four times the cost of the Rock Toe Buttress. A specialty contractor would also be required to install the soil nails. The elevated groundwater table within the clay dike may lead to installation problems with conventional drilled and grouted nails. This problem could potentially be avoided by using nails pneumatically "shot" into the clay dike, although only a few specialty contractors are available to perform this type of work.

In summary, soil nails are not a viable alternative to the rock toe buttress for this project. While soil nails avoid permitting issues caused by encroachment into wetlands, they cost more than four times as much as the preferred alternative IRRP #1 and have potential installation and durability issues.

6. French Drains

French drains consist of an excavated trench, typically 2-3 feet in width, backfilled with a free-draining material, and a perforated drainage pipe installed near the bottom of the trench. French drains are effective in clayey soils for locally depressing the water table (piezometric surface), due to the large drainage surface offered by the drain. French drains are installed on a slight slope along their profile, typically 0.25-1.0%, and seepage water is directed to a surface discharge point or sump for pumping away from the collection location.

Based on evaluations of this technology for similar applications, typical analyses show that a French drain installed to a minimum depth of 20 feet in the middle of the clay dike is effective for both controlling seepage and increasing slope stability to an acceptable level of 1.5 or greater. The total estimated cost of the French drains is $2,780,000 and the estimated construction duration is 2 to 3 months using one crew. The estimated construction costs and durations for areas encroaching on wetlands are $780,000 and 1 month, respectively.

The advantages of French drains include: (1) a reverse-graded seepage control filter is not required, as the French drains depress the ground water table below the face of the clay dike; (2) the repair would avoid the majority of the wetlands encroachment; (3) it has a relatively low construction cost; and (4) it has a relatively short construction duration.

However, French drains cannot be routinely installed to depths of 20 feet in saturated soils without employing one-pass trenching, a technique used by specialty contractors. During one-pass trenching, the drain is cut and held open at the same time using a specially designed trencher, and a pipe is placed along with backfill material in one operation.

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In addition, due to the low existing factors of safety of the clay dike (1.2 or less), a small rock buttress may still be required at the toe of the dike. The buttress could be located out of the majority of the wetlands, but construction equipment would still likely have to drive through the wetlands, causing a temporary disturbance. Even considering the use of one-pass trenching, the disturbance through installation of the drain within the marginally stable clay dike presents an increased risk.

Further, the depth of the French drain would also require that pumps be installed to remove seepage water, as the drain is too deep to effectively drain by gravity. Pumping would require power and continual maintenance until after the Ash Pond Complex is permanently closed. Additionally, the pumped French drains are not a redundant remediation option, meaning that if the drain was to clog, the seepage water would accumulate in the drain and the slope stability factor of safety could again drop below 1.5.

In summary, French drains are an effective method for increasing slope stability and controlling seepage, without the wetland encroachment issues of the current preferred alternative IRRP #1. While they are less in cost than most of the other alternatives, French drains have several durability issues that would need to be addressed. They require significantly more operation and maintenance than other options, and carry significantly more risk than the other remediation options, limiting the viability of this alternative.

7. Bentonite Slurry Seepage Cutoff Wall

This remediation alternative consists of a vertical bentonite-filled trench installed around the perimeter of a seepage source, essentially "cutting off" potential flow from the seepage source. Cutoff walls are often constructed by excavating and simultaneously backfilling a trench with bentonite-water slurry. The bentonite slurry acts to stabilize the walls of the trench and also forms a low-permeability barrier, which will restrict seepage through the wall. For this particular project, the cutoff wall would be installed along the crest of the Main Ash Pond perimeter clay dike, for constructability and effectiveness. The cutoff wall would vertically extend to several feet within the native clay soils below the Ash Pond Complex. The total estimated cost of the Bentonite Slurry Seepage Cutoff Wall is $4,460,000 and the estimated construction duration is 2 to 3 months using one crew. The estimated construction costs and durations for areas encroaching on wetlands are $1,940,000 and 1 month, respectively.

Our analyses show that a bentonite slurry cutoff wall may improve stability factors of safety, to the required minimum of 1.5 or greater (in addition to lowering the Ash Pond normal operating level to an elevation of 627-ft). However, the long-term performance of the bentonite slurry wall is suspect, as the CCP materials may chemically alter the bentonite, making it more permeable. A similar bentonite slurry wall installed at the WCF Bottom Ash Stack is reportedly ineffective in reducing seepage.

To avoid the potential chemical reaction between the CCP materials the bentonite, the slurry cutoff wall would have to be placed such that it would not come into contact with any CCP materials. The geometry of the existing perimeter clay dike does not readily facilitate such construction. As such, utilization of a bentonite slurry seepage cutoff wall is not considered a viable alternative.

Summary and Conclusions

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Remediation Alternatives Summary Matrix

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REMEDIATION ALTERNATIVES SUMMARY MATRIX
WCF Ash Pond Complex, Tennessee Valley Authority

Remediation Option	Description	Estimated Construction Cost	Estimated Construction Duration	Advantages	Disadvantages
1. Lowering the Normal Pool Elevation/Ash Dredging	Removal of approximately 996,300 cy of Ash from the Ash Pond Complex and placement at an off-site location	$10,650,000	7 to 8 Months	1. Does not encroach into wetlands 2. Low failure risk after construction	1. Extremely high cost.
2. Rock Toe Buttress (preferred IRRP-URSAC001 design)	Reverse-graded filter installed at toe of clay dike. Controls seepage and increases slope stability.	$1,950,000 Wetlands only: $290,000	2 to 3 Months (20 12-cy trucks making 5 trips/day) Wetlands only: 2 to 3 months	1. Least expensive repair option 2. Low failure risk during and after construction 3. Relatively short construction duration	1. Permitting issues due to encroachment into wetlands
3. Deep-Soil Mixing	Columns of soil enhanced with Portland cement through mixing installed within clay dike. Improves slope stability. Reverse-graded filter also required to control seepage.	$5,790,000 Wetlands only: $1,160,000	6 to 7 Months (4 rigs) Wetlands only: South – 1 to 2 months North – 1 month	1. Repair does not encroach into wetlands	1. Permitting issues due to encroachment into wetlands 2. Specialty contractor required for construction 3. Does not control seepage, small reverse-graded filter still required 4. Temporary construction berm and work pad required for construction, which will encroach into wetlands 5. Test sections needed to ensure effectiveness of method, which are not included in cost 6. Spoil control required to prevent soil/cement mix from entering wetlands, which is not included in cost. 7. More expensive than other repair options.
4. Vibro-Stone Columns	Columns of vibratory-compacted stone installed within the clay dike. Improves slope stability. Reverse-graded filter also required to control seepage.	$8,050,000 Wetlands only: $1,580,000	4 to 5 Months (3 rigs) Wetlands only: 1 to 2 months	1. Repair does not encroach into wetlands 2. Relatively short construction duration	1. Specialty contractor required for construction 2. Does not control seepage, small reverse-graded filter still required 3. Temporary construction berm and work pad required for construction, which will encroach into wetlands 4. Ground heave will likely occur during construction 5. Columns may act as pathways for excess seepage to exit the clay dike 6. Considerably more expensive than other repair options
5. Soil Nailing	Reinforcing steel “nails” grouted into the slope, bearing on a shotcrete facing. Improves slope stability. Reverse-graded filter also required to control seepage.	$11,200,000 Wetlands only: $2,601,000	8 to 9 Months (3 crews) Wetlands only: 2 to 3 months	1. Repair does not encroach into wetlands 2. Small construction access pad needed	1. Specialty contractor required for construction 2. Does not control seepage, small reverse-graded filter still required 3. Leachate may be corrosive to steel soil nails 4. Problematic installation in saturated soils 5. Most expensive repair option 6. Longest construction duration
6. French Drains	Sand-filled trench with perforated drain pipe installed 20’ deep on the crest of the clay dike. Installation must be completed using “one-pass” trenching method.	$2,780,000 Wetlands only: $780,000	2 to 3 Months (1 rig) Wetlands only: 1 month	1. Repair does not encroach into wetlands 2. Reverse-graded filter not required to control seepage 3. Relatively short construction duration	1. Specialty contractor required for construction 2. Temporary berm required for construction, which extends slightly into existing wetlands 3. Pumps and piping will require continued maintenance 4. Carries increased risk relative to other repair options
7. Bentonite Slurry Seepage Cutoff Wall	Deep trench filled with bentonite-water slurry installed at the top edge of the Ash Pond Complex.	$4,460,000 Wetlands only: $1,940,000	2 to 3 Months Wetlands only: 1 month	1. Repair does not encroach into wetlands 2. Relatively short construction duration 3. Reverse-graded filter not required to control seepage	1. Specialty contractor required for construction 2. Potential chemical reactivity with bentonite

Notes:
1. Construction duration is only for portion of project replacing Rock Toe Buttress (IRRP #1).
2. Estimated costs include the following: (1) materials, (2) labor, (3) site clearing, and (4) 20% contingency.

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Our analyses and experience have shown that potential effective remediation alternatives to the preferred rock toe buttress IRRP #1 design include lowering of the normal pool elevation / ash dredging; deep-soil mixing; vibro-stone columns; soil nailing; French drains; and a Bentonite Slurry Cutoff Wall. Of these remediation alternatives, only the lowering of the normal pool elevation / ash dredging and French drains were considered viable alternatives. However, when compared to the preferred Rock Toe Buttress design (IRRP #1), even these two remaining viable alternatives either carry a significantly higher cost than IRRP #1, encroach significantly into existing wetlands, or both.

URS recommends that the preferred Rock Toe Buttress design (IRRP #1) be constructed as the proposed remediation alternative.

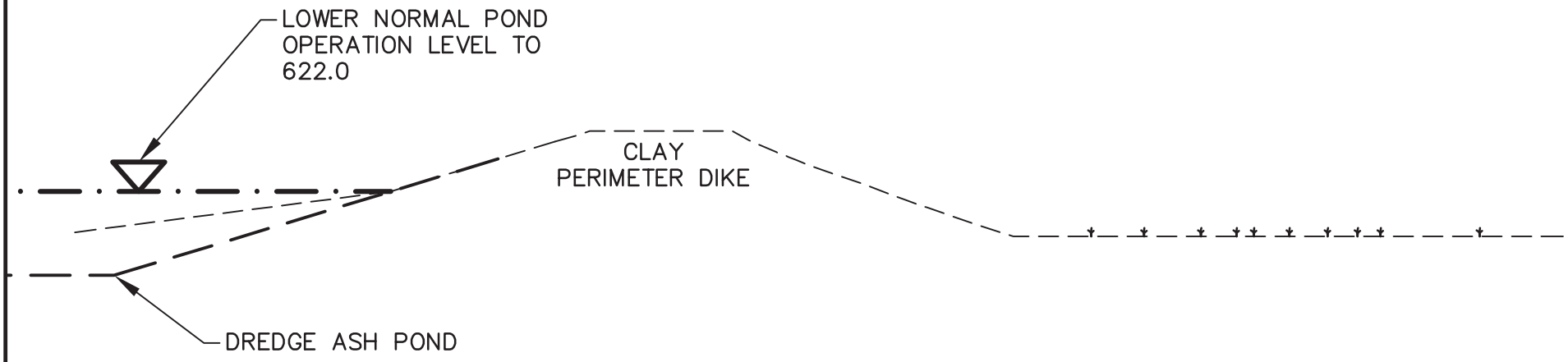
Enclosures: Remediation Alternatives Summary Matrix
Conceptual Drawings of Remediation Alternatives
Preliminary Cost Estimates

Cc: J. Blackburn (TVA)
J. Mokotoff, PE (URS)
A. Harrigal, PE (URS)

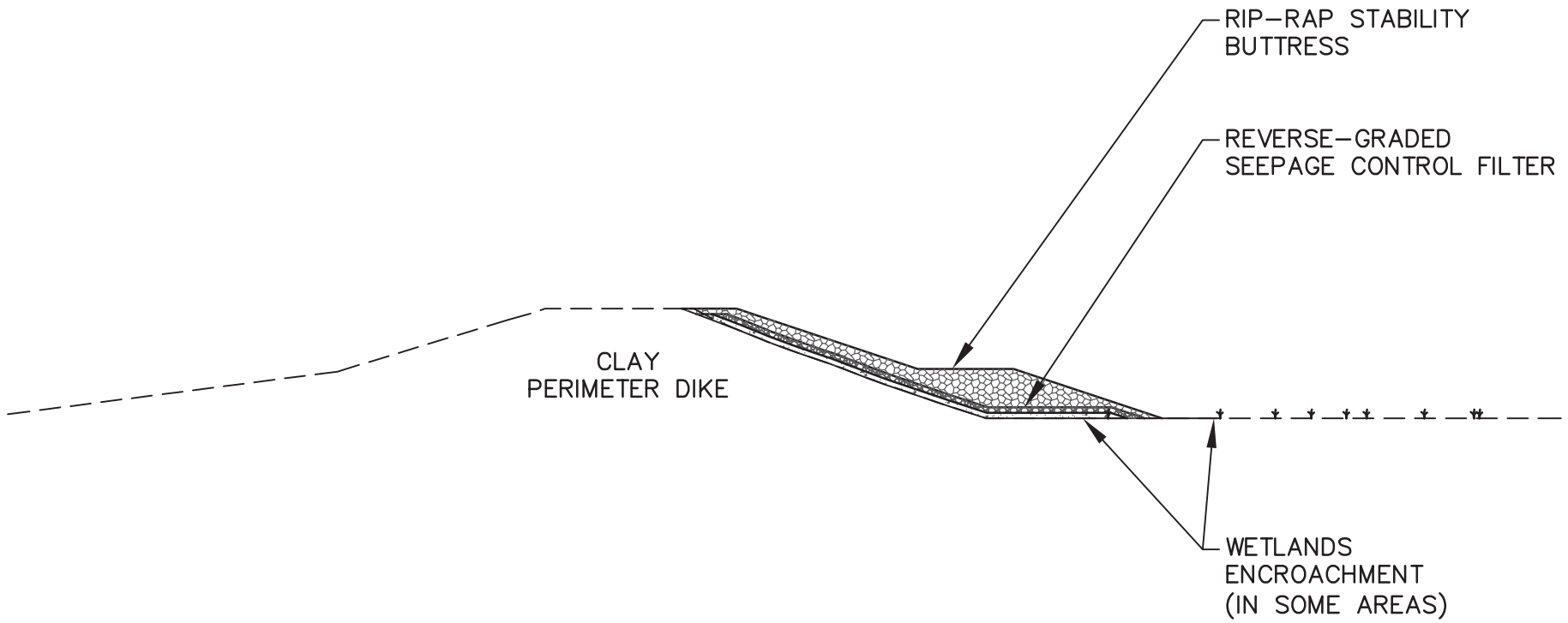
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Conceptual Drawings of Remediation Alternatives

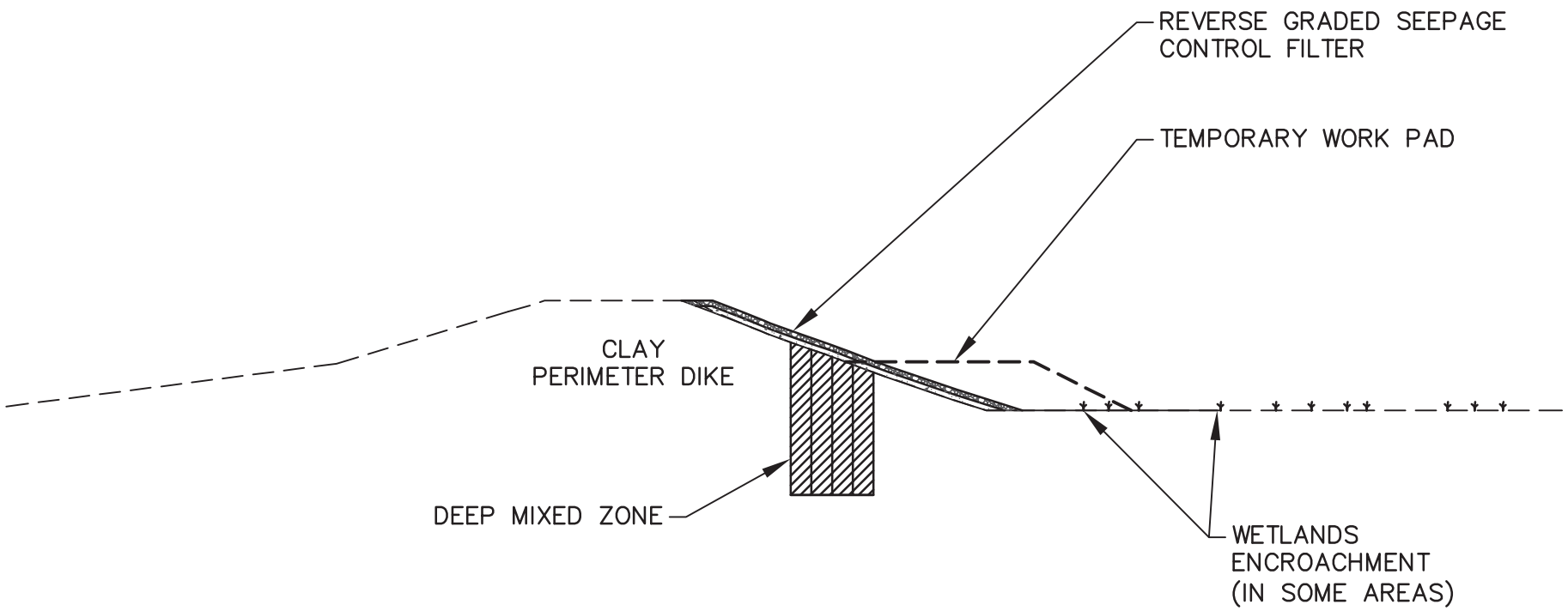
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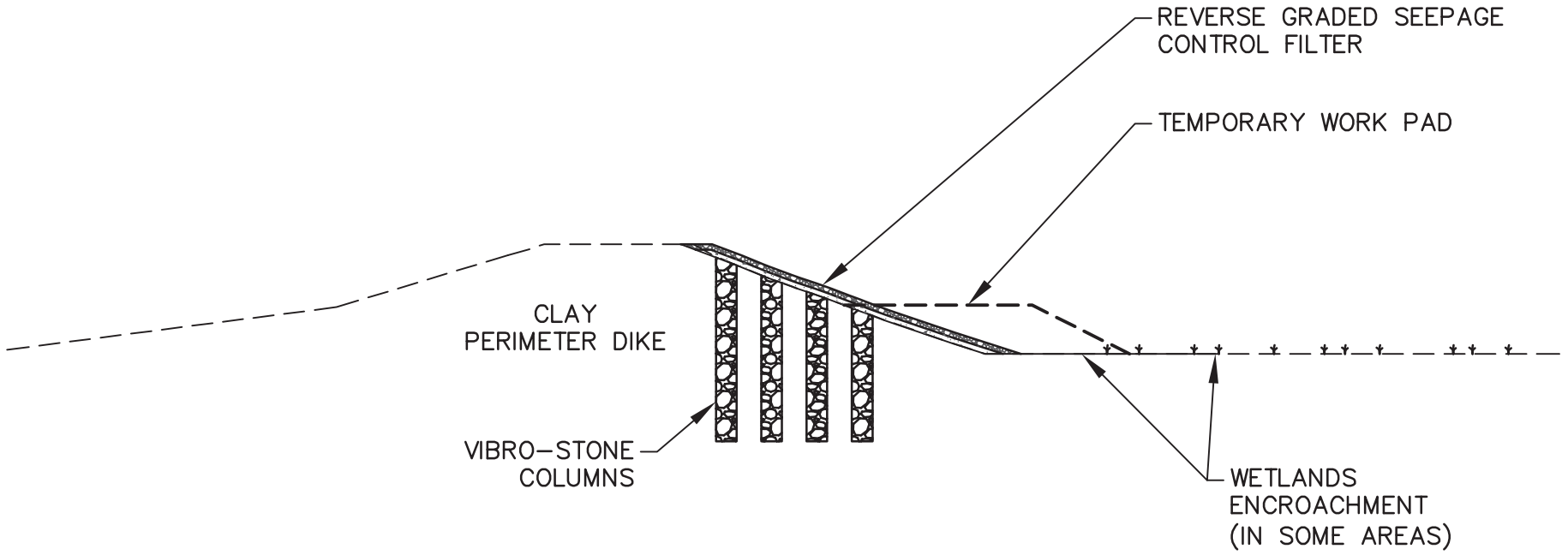
REPAIR ALTERNATIVE 1
LOWERING THE NORMAL POOL ELEVATION/ASH DREDGING



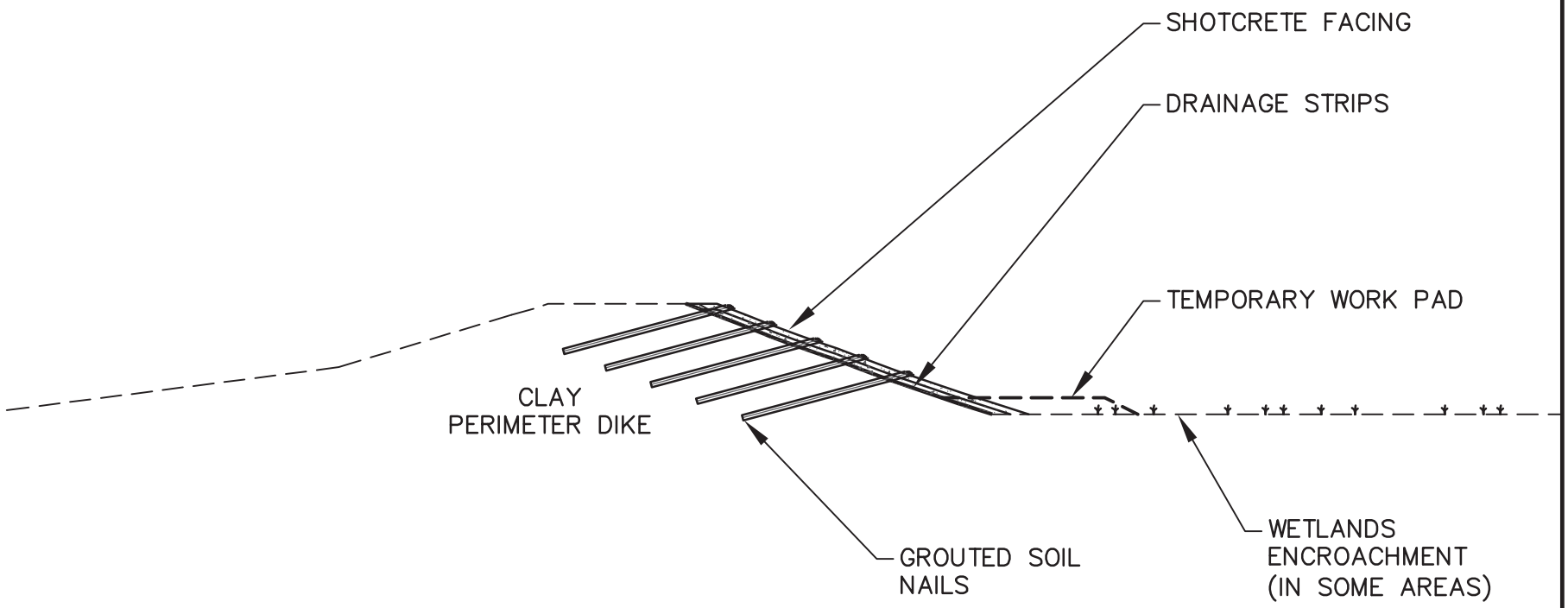
REPAIR ALTERNATIVE 2
ROCK TOE BUTTRESS
(PREFERRED IRRP-URSAC001 DESIGN)



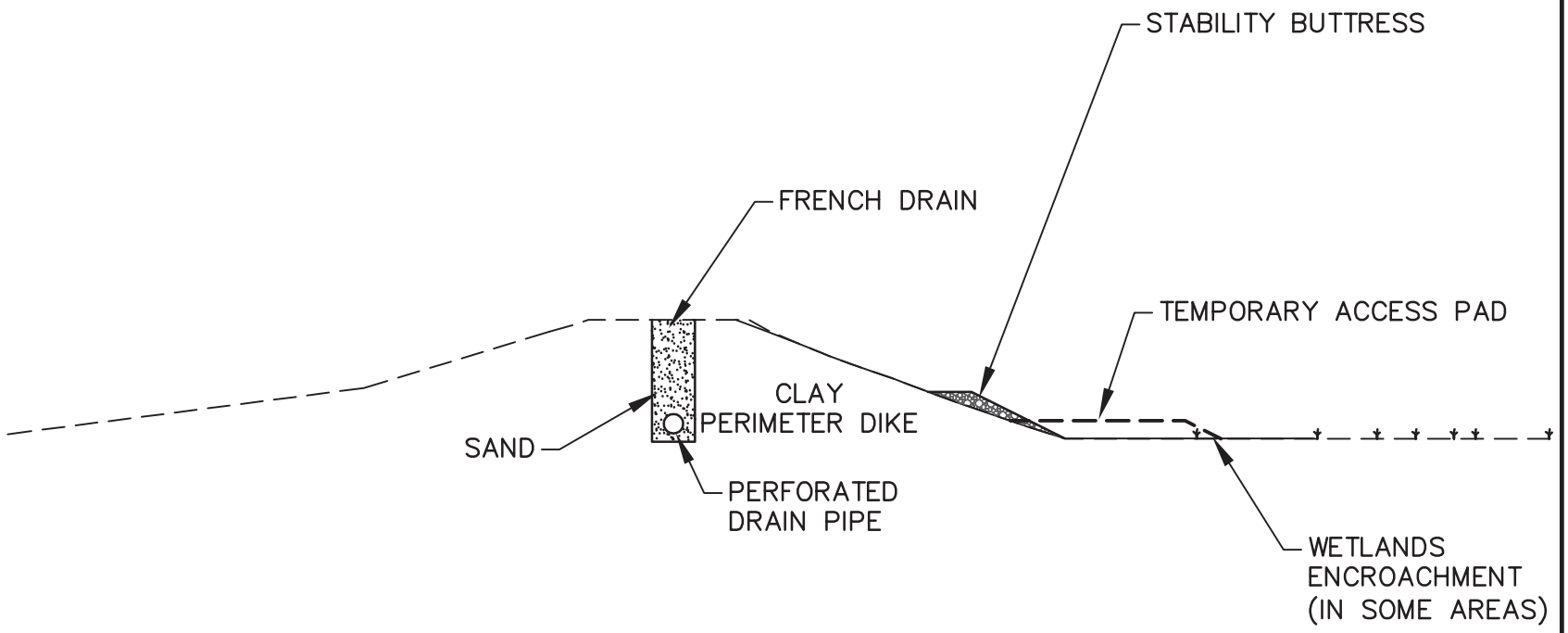
REPAIR ALTERNATIVE 3
DEEP SOIL MIXING



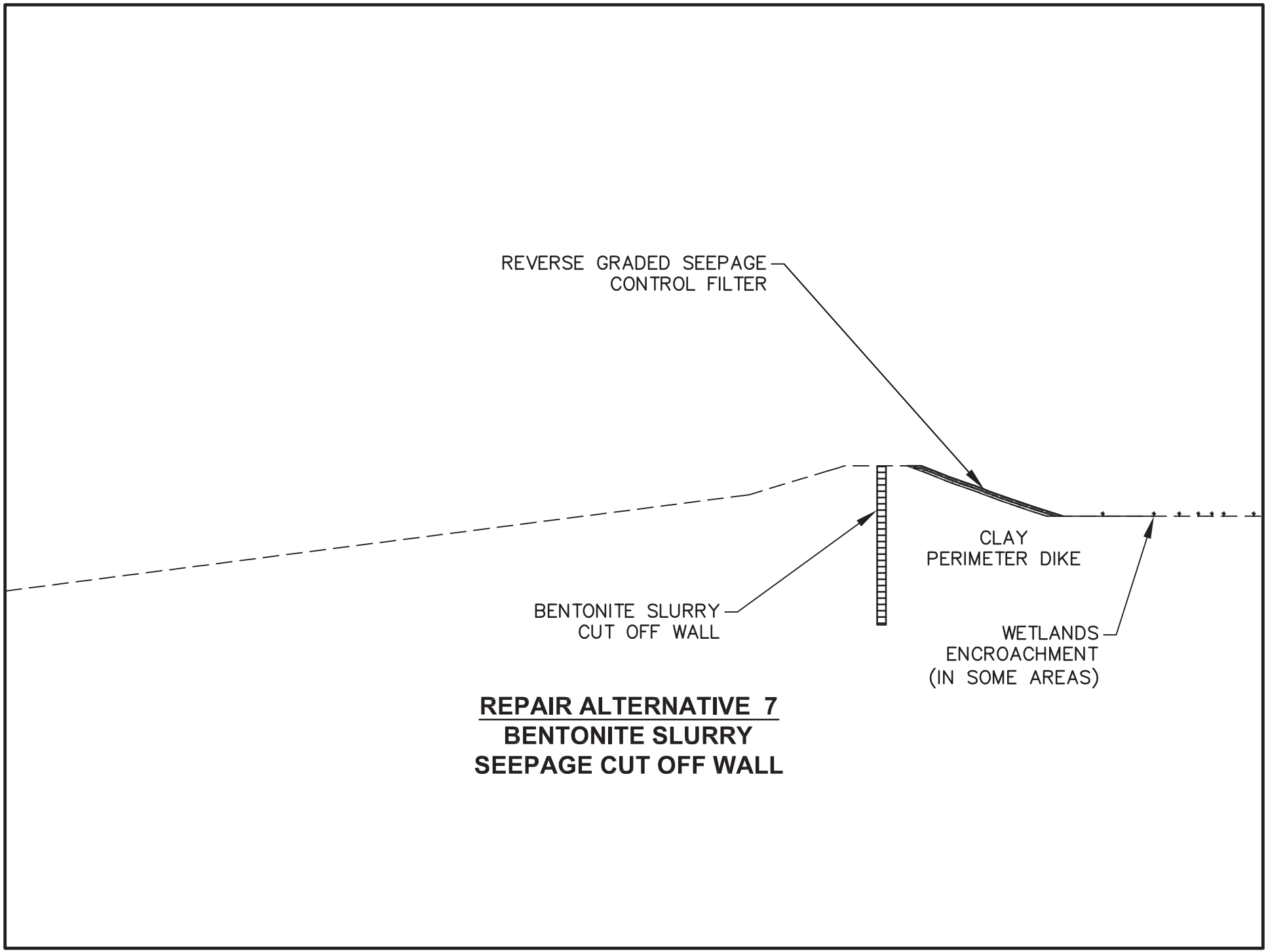
REPAIR ALTERNATIVE 4
VIBRO-STONE COLUMNS



REPAIR ALTERNATIVE 5
SOIL NAILING



REPAIR ALTERNATIVE 6
FRENCH DRAIN



REPAIR ALTERNATIVE 7
BENTONITE SLURRY
SEEPAGE CUT OFF WALL

Preliminary Cost Estimates

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TVA Widows Creek Fossil Plant
Conceptual Cost Estimate
Alternative 1: Dredging

Calculated By: AGH
Checked By: JJH
Date: 9/24/2010

Item	Length ft	Volume cy	Weight ton	Unit Rate	Unit	Cost	Duration
Dredging	-	600,700	-	$8.85	cy	$5,316,195	7 to 8 Months
Mobilization	-	-	-	$50,000.00	LS	$50,000	-
					Total	$5,366,195	
					20% contingency	$1,073,239	
					Total Cost	**$6,439,434**	

Notes:
1. All alternatives presented are in comparison to IRRP-URSGS001 (clay dike buttress).
2. Duration of material removal and seepage control placement can be shortened if number of trucks and total trips per day are increa
3. Material removal unit rate from TVA experience with fly ash removal at the Kingston Fossil Plant.

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TVA Widows Creek Fossil Plant
Conceptual Cost Estimate
Alternative 2: Rock Toe Buttress (IRRP-URSAC001)

Calculated By: AN
Checked By: JJH
Date: 9/24/2010

Item	Length ft	Area ac	Volume cy	Unit Rate	Unit	Cost	Duration days	
Construction Stakeout & Survey	-	-	-	$31,746.33	LS	$31,746		
Buttress Rock (Class 1 Rip Rap)[1,2,4]	-	-	20,000	$27.00	cy	$540,000		20 -12 cy haul trucks making 5 trips/day
Buttress Rock (No. 7 Stone)[1,2,4]	-	-	12,500	$27.00	cy	$337,500		20 -12 cy haul trucks making 5 trips/day
Buttress Conc. Sand[1,2,4]	-	-	12,000	$26.16	cy	$313,920		20 -12 cy haul trucks making 5 trips/day
Buttress Construction[3,5]	-	-	-	$2,123.84	day	$193,269	91	2 - 1.5 cy tracked dozers at 60 lf of buttress/day
Isolation Berm Engineered Fill[4]	-	-	11,481	$10.00	cy	$114,815		
Isolation Berm Coarse Aggregate[1,4]	-	-	172	$27.00	cy	$4,650		
Dewatering System Construction[3]				$2,123.84	day	$21,238	10	2 - 1.5 cy tracked dozers
Mobilization/Demobilization	-	-	-	$5,973.24	LS	$5,973		
Clearing and Grubbing	-	2.4	-	$2,000.00	ac	$4,800		
Storm Water E&S Control Mgmt	5,460	-	-	$5.00	ft	$27,300		
Seeding/Restoration	-	2.4	-	$1,000.00	ac	$2,400		
Working in Saturated Areas	-	-	-	$21,450.78	LS	$21,451		
					Sub Total	$1,619,063		
					20% contingency	$323,813		
					Total Cost	**$1,942,876**		

Wetlands Only (BW04):

Item	Length ft	Area ac	Volume cy	Unit Rate	Unit	Cost	Duration days	
Construction Stakeout & Survey	-	-	-	$4,730.04	LS	$4,730		
Buttress Rock (Class 1 Rip Rap)[1,2,4]	-	-	540	$27.00	cy	$14,580		20 -12 cy haul trucks making 5 trips/day
Buttress Rock (No. 7 Stone)[1,2,4]	-	-	320	$27.00	cy	$8,640		20 -12 cy haul trucks making 5 trips/day
Buttress Conc. Sand[1,2,4]	-	-	320	$26.16	cy	$8,371		20 -12 cy haul trucks making 5 trips/day
Buttress Construction[3,5]	-	-	-	$2,123.84	day	$44,601	21	2 - 1.5 cy tracked dozers at 60 lf of buttress/day
Isolation Berm Engineered Fill[4]	-	-	11,481	$10.00	cy	$114,815		
Isolation Berm Coarse Aggregate[1,4]	-	-	172	$27.00	cy	$4,650		
Dewatering System Construction[3]				$2,123.84	day	$21,238	10	2 - 1.5 cy tracked dozers
Mobilization/Demobilization	-	-	-	$5,973.24	LS	$5,973		
Clearing and Grubbing	-	0.25	-	$2,000.00	ac	$500		
Storm Water E&S Control Mgmt	1,260	-	-	$5.00	ft	$6,300		
Seeding/Restoration	-	0.25	-	$1,000.00	ac	$250		
Working in Saturated Areas	-	-	-	$6,583.90	LS	$6,584		
					Sub Total	$241,232		
					20% contingency	$48,246		
					Total Cost	**$289,479**		

Notes:
1. Sand and rock pricing were obtained from phone conversation with Trans-Ash on 5-27-2010. Price includes hauling to site, but excludes construction.
2. Current estimate is 20 trucks based on hauling material for 40 days.
3. Construction unit rate estimate from RS Means pg. 463 - 2 1.5 cy tracked dozers.
4. Buttress and isolation berm volumes obtained from previously submitted (7/22/2010) Work Plan 24.
5. Buttress construction rate of 60 lf/day is based on the average construction rate of the gypsum stack buttress.
6. The rock toe buttress has been optimized for implementation in conjunction with a reduction in the Main Ash Pond normal pool elevation, as documented in the Decision Matrix Summary Memorandum, dated May 3, 2010. This cost is inherent for every alternative as lowering the normal pool elevation would be assumed necessary to reach the minimum factor of safety

* Statements of Probable Construction Cost prepared by URS represent URS's judgment as a design professional familiar with the construction industry. It is recognized, however, that neither URS nor the Owner has control over the cost of labor, materials or equipment nor over the contractor's methods of determining the bid price or other competitive bidding, market, or negotiating conditions. Accordingly, URS cannot and does not warrant or represent that proposals, bids or actual construction costs will not vary from any statement of Probable Construction Cost or other estimates or evaluations prepared by URS.

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TVA Widows Creek Fossil Plant
Conceptual Cost Estimate
Alternative 3: Deep-Soil Mixing

Calculated By: AGH
Checked By: JJH
Date: 9/24/2010

Item	Columns ea.	Diameter ft	Depth ft	Unit Rate	Unit	Cost	Duration
Deep Soil Mixing	4,098	3	25	$100.00	cy	$2,682,132	205 days at 4 rigs producing 20 columns/day
Mobilization	-	-	-	$100,000.00	LS	$100,000	-
Item	**Length ft**	**Area ft^2**	**Volume cy**	**Unit Rate**	**Unit**	**Cost**	**Duration**
Temporary Clay/Rock Berm	5,460	400	80,889	$12.00	cy	$970,667	-
Temporary Berm Placement	-	-	-	$1,651.74	day	$127,184	68 days with 20-12 cy haul trucks making 5 trips/day
Seepage Control - sand	5,460	59	12,000	$26.16	cy	$313,917	-
Seepage Control - rock No. 7	5,460	62	12,500	$27.00	cy	$337,510	
Seepage Control - rock Rip-Rap	5,460	99	20,000	$27.00	cy	$539,994	-
Sand & Rock Placement	-	-	-	$2,123.84	day	$84,954	40 days with 20 -12 cy haul trucks making 5 trips/day
					Total	$4,818,847	
					20% contingency	$963,769	
					Total Cost	**$5,782,617**	

Wetlands Only (BW04):

Item	Columns ea.	Diameter ft	Depth ft	Unit Rate	Unit	Cost	Duration
Deep Soil Mixing	948	3	25	$100.00	cy	$620,464	64 days at 3 rigs producing 15 columns/day
Mobilization	-	-	-	$50,000.00	LS	$50,000	-
Item	**Length ft**	**Area ft^2**	**Volume cy**	**Unit Rate**	**Unit**	**Cost**	**Duration**
Temporary Clay/Rock Berm	1,260	400	18,667	$12.00	cy	$224,000	-
Temporary Berm Placement	-	-	-	$1,651.74	day	$33,035	16 days with 20-12 cy haul trucks making 5 trips/day
Seepage Control - sand	1,260	-	540	$26.16	cy	$14,126	-
Seepage Control - rock No. 7		-	320	$37.00	cy	$11,840	
Seepage Control - rock Rip-Rap	1,260	-	320	$27.00	cy	$8,640	-
Sand & Rock Placement	-	-	-	$2,123.84	day	$14,867	2 days with 10 -12 cy haul trucks making 5 trips/day
					Total	$965,132	
					20% contingency	$193,026	
					Total Cost	**$1,158,159**	

Notes:
1. All alternatives presented are in comparison to IRRP-URSAC001 (clay dike buttress).
2. Alternatives are located at the same stationing as IRRP-URSAC001 and act as a replacement for the clay dike buttress.
3. Sand and rock pricing were obtained from phone conversation with Trans-Ash on 5-27-2010. Prices include hauling to site.
4. Duration of seepage control placement and temporary berm placement can be shortened if number of trucks and total trips per day are increased.
5. Seepage control placement unit rate estimate from RS Means pg. 224 - select granular fill, wheel mounted front loader, 1.5 CY bucket.
6. Deep soil mixing rates were based on prior URS project experience, including the Deep Mixing in Wilson Bridge and Baby Creek projects in New Orleans, LA.
7. Duration of deep-soil mixing can be decreased if additional rigs are used.
8. Temporary berm placement unit rate estimate from RS Means pg. 463 - 3 cy tracked dozers for 77 days
9. Sand and rock placement unit rate estimate from RS Means pg. 463 - 2 1.5 cy tracked dozers for 27 days
10. Deep mixing replacement ratio is 30%.
11. Deep mixing column layout and seepage control cross-sectional areas obtained from SEEP/W and SLOPE/W analyses.

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TVA Widows Creek Fossil Plant
Conceptual Cost Estimate
Alternative 4: Vibro-Stone Columns

Calculated By: AGH
Checked By: JJH
Date: 9/24/2010

Item	Columns ea.	Diameter ft	Length ft	Unit Rate	Unit	Cost	Duration
Stone Columns	8,494	3	18	$28.00	ft	$4,280,976	102 days with 3 rigs producing 500/ft/day/rig
Mobilization	-	-	-	$50,000.00	LS	$50,000	-
Item	**Length ft**	**Area ft^2**	**Volume cy**	**Unit Rate**	**Unit**	**Cost**	**Duration**
Temporary Clay/Rock Berm	5,460	400	80,889	$12.00	cy	$970,667	-
Temporary Berm Placement	-	-	-	$1,651.74	day	$127,184	68 days with 20-12 cy haul trucks making 5 trips/day
Seepage Control - sand	5,460	59	12,000	$26.16	cy	$313,917	-
Seepage Control - rock No. 7	5,460	62	12,500	$27.00	cy	$337,510	
Seepage Control - rock Rip-Rap	5,460	99	20,000	$27.00	cy	$539,994	-
Sand & Rock Placement	-	-	-	$2,123.84	day	$84,954	40 days with 20 -12 cy haul trucks making 5 trips/day
					Total	$6,705,201	
					20% contingency	$1,341,040	
					Total Cost	**$8,046,241**	

Wetlands Only (BW04):

Item	Columns ea.	Diameter ft	Length ft	Unit Rate	Unit	Cost	Duration
Stone Columns	1,960	3	18	$28.00	ft	$987,840	24 days with 3 rigs producing 500/ft/day/rig
Mobilization	-	-	-	$25,000.00	LS	$25,000	-
Item	**Length ft**	**Area ft^2**	**Volume cy**	**Unit Rate**	**Unit**	**Cost**	**Duration**
Temporary Clay/Rock Berm	1,260	400	18,667	$12.00	cy	$224,000	-
Temporary Berm Placement	-	-	-	$1,651.74	day	$33,035	16 days with 20-12 cy haul trucks making 5 trips/day
Seepage Control - sand	1,260	-	540	$26.16	cy	$14,126	-
Seepage Control - rock No. 7		-	320	$27.00	cy	$8,640	
Seepage Control - rock Rip-Rap	1,260	-	320	$27.00		$8,640	-
Sand & Rock Placement	-	-	-	$2,123.84	day	$14,867	2 days with 10 -12 cy haul trucks making 5 trips/day
					Total	$1,316,148	
					20% contingency	$263,230	
					Total Cost	**$1,579,378**	

Notes:
1. All alternatives presented are in comparison to IRRP-URSGS001 (clay dike buttress).
2. Alternatives are located at the same stationing as IRRP-URSGS001 and act as a replacement for the clay dike buttress.
3. Sand and rock pricing were obtained from phone conversation with Trans-Ash on 5-27-2010. Prices include hauling to site.
4. Stone column unit costs and construction rates obtained from phone conversation with Hayward Baker on 5-18-2010
5. Duration of seepage control placement and temporary berm placement can be shortened if number of trucks and total trips per day are increased.
6. Sand and rock placement unit rate estimate from RS Means pg. 463 - 2 1.5 cy tracked dozers for 27 days
7. Temporary berm placement unit rate estimate from RS Means pg. 463 - 3 cy tracked dozers for 77 days
8. Stone column replacement ratio is 40%
9. Vibro-stone column layout and seepage control cross-sectional areas obtained from SEEP/W and SLOPE/W analyses.

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TVA Widows Creek Fossil Plant
Conceptual Cost Estimate
Alternative 5: Soil nails

Calculated By: AGH
Checked By: JJH
Date: 9/24/2010

Item	Nails ea.	Length ft	Spacing ft	Unit Rate	Unit	Cost	Duration
Soil Nails	7,644	40	5	$730.00	nail	$5,580,120	250 days with 3 crews each producing 10.2 nails/day
Mobilization	-	-	-	$50,000.00	LS	$50,000	-
Item	Length ft	Area ft^2	Volume cy	Unit Rate	Unit	Cost	Duration
Temporary Facing (2" Thick)	-	273,000	-	$6.10	sf	$1,665,300	79 days with 3 crews each producing 1160 sq. ft/day
Permanent Facing (3" Thick)	-	273,000	-	$7.35	sf	$2,006,550	152 days with 3 crews each producing 600 sq. ft/day
Drainage Strips	-	54,600	-	$0.54	sf	$29,605	-
					Total	$9,331,575	
					20% contingency	$1,866,315	
					Total Cost	**$11,197,890**	

Wetlands Only (BW04):

Item	Nails ea.	Length ft	Spacing ft	Unit Rate	Unit	Cost	Duration
Soil Nails	1,764	40	5	$730.00	nail	$1,287,720	58 days with 3 crews each producing 10.2 nails/day
Mobilization	-	-	-	$25,000.00	LS	$25,000	-
Item	Length ft	Area ft^2	Volume cy	Unit Rate	Unit	Cost	Duration
Temporary Facing (2" Thick)	-	63,000	-	$6.10	sf	$384,300	19 days with 3 crews each producing 1160 sq. ft/day
Permanent Facing (3" Thick)	-	63,000	-	$7.35	sf	$463,050	35 days with 3 crews each producing 600 sq. ft/day
Drainage Strips	-	12,600	-	$0.54	sf	$6,832	-
					Total	$2,166,902	
					20% contingency	$433,380	
					Total Cost	**$2,600,282**	

Notes:
1. All alternatives presented are in comparison to IRRP-URSGS001 (clay dike buttress).
2. Alternatives are located at the same stationing as IRRP-URSGS001 and act as a replacement for the clay dike buttress.
3. Soil nail cost and construction duration obtained from RS Means pg. 237 - difficult nailing conditions, 90 ft/hr drilling, grade 75 steel nails, 20 min setup
4. Shotcrete facing cost and construction duration obtained from RS Means pg. 67 (3" thick temp. facing) and pg. 68 (2" thick permanent facing)
5. Soil nail layout and seepage control cross-sectional areas obtained from SEEP/W and SLOPE/W analyses.
6. Drainage strips assumed to cover 20% of wall facing area. Cost obtained from RS Means pg. 317 - 0.44" thick vertical soil drainage mat.

TVA Widows Creek Fossil Plant
Conceptual Cost Estimate
Alternative 6: French drains

Calculated By: AGH
Checked By: JJH
Date: 9/24/2010

Item	Quantity	Length ft	Unit Rate	Unit	Cost	Duration
Mobilization/Demobilization	-	-	$65,000	LS	$65,000	-
Installation of Trench	-	5,460	$145	LF	$791,700	55 days with 100 ft installed per day
Installation of Sumps	6	-	$10,000	ea.	$60,000	-
Submersible Pumps	6	-	$2,000	ea.	$12,000	-
Pressure Pipe Installation	-	5,460	$10.70	LF	$58,422	18 days with 310 ft installed per day
Pressure Pipe Install. Labor	20	-	$1,172	day	$23,448	-
Pump Labor (1 hr/day for 5 years)	1825	-	$32.50	hour	$59,313	-
Pump O&M (10% of pump cost over 5 years)	-	-	-	LS	$6,000	-
Power Hookup to Pumps	-	-	$200,000	LS	$200,000	
Power to pumps	-	-	$48,376	LS	$48,376	-

Item	Length	Area ft^2	Volume cy	Unit Rate	Unit	Cost	Duration
Spoil Removal (Excavation)	5,460	40	8,089	$1.98	cy	$16,016	-
Sand Placement in Trencher (Fill)	5,460	-	-	$918.04	day	$56,918	62 days, wheel loader supporting trencher
Drain Backfill Sand	5,460	40	8,089	$26.16	cy	$211,605	-
Construction Buttress - Rock	5,460	100	20,222	$27.00	cy	$546,000	-
Construction Buttress Placement	-	-	-	$2,123.84	day	$148,669	70 days with 6 - 12 cy haul trucks making 5 trips/day
Clay Cover	5,460	2	404	$12.00	cy	$4,853	-
Clay Cover Placement	-	-	-	$918.04	day	$7,344	8 days with 1 - 12 cy haul truck making 5 trips/day
					Total	$2,315,665	
					20% contingency	$463,133	
					Total Cost	**$2,778,798**	

Wetlands Only (BW04):

Item	Quantity	Length ft	Unit Rate	Unit	Cost	Duration
Mobilization/Demobilization	-	-	$32,500	LS	$32,500	-
Installation of Trench	-	1,260	$145	LF	$182,700	13 days with 100 ft installed per day
Installation of Sumps	2	-	$10,000	ea.	$20,000	-
Submersible Pumps	2	-	$2,000	ea.	$4,000	-
Pressure Pipe Installation	-	1,260	$10.70	LF	$13,482	5 days with 310 ft installed per day
Pressure Pipe Install. Labor	8	-	$1,172	day	$9,379	-
Pump Labor (1 hr/day for 5 years)	912.5	-	$32.50	hour	$29,656	-
Pump O&M (10% of pump cost over 5 years)	-	-	-	LS	$2,000	-
Power Hookup to Pumps	-	-	$100,000	LS	$100,000	
Power to pumps	-	-	$24,188	LS	$24,188	-

Item	Length	Area ft^2	Volume cy	Unit Rate	Unit	Cost	Duration
Spoil Removal (Excavation)	1,260	40	1,867	$1.98	cy	$3,696	-
Sand Placement in Trencher (Fill)	-	-	-	$918.04	day	$22,951	25 days, wheel loader supporting trencher
Drain Backfill Sand	1,260	40	1,867	$26.16	cy	$48,832	-
Construction Buttress - Rock	1,260	100	4,667	$27.00	cy	$126,000	-
Construction Buttress Placement	-	-	-	$2,123.84	day	$27,610	13 days with 6 - 12 cy haul trucks making 5 trips/day
Clay Cover	1,260	2	93	$12.00	cy	$1,120	-
Clay Cover Placement	-	-	-	$918.04	day	$1,836	2 days with 1 - 12 cy haul truck making 5 trips/day
					Total	$649,950	
					20% contingency	$129,990	
					Total Cost	**$779,941**	

Notes:
1. All alternatives presented are in comparison to IRRP-URSGS001 (clay dike buttress).
2. Alternatives are located at the same stationing as IRRP-URSGS001 and act as a replacement for the clay dike buttress.
3. Sand and rock pricing were obtained from phone conversation with Trans-Ash on 5-27-2010. Prices include hauling to site.
4. Duration of construction buttress placement can be shortened if number of trucks and total trips per day are increased.
5. Construction buttress placement unit rate estimate from RS Means pg. 463 - 2 1.5 cy tracked dozers for 70 days
6. French drain and construction buttress layout and cross-sectional areas obtained from SEEP/W and SLOPE/W analyses
7. One-pass trenching estimate is project-specific and was provided by DeWind One-Pass Trenching, Inc.
8. Pump rental costs for 2" diameter submersible pumps, per prior URS project experience.
9. Power supply estimate based on TVA BRF project. Length of power line for WCF is similar (1.5 miles).
10. Pressure Pipe Installation Labor includes a 4 person crew with a laborer, plumber, plumber's apprentice, and foreman working 8 hour days.
11. Pipe is 6" PVC plastic pressure pipe. Slotted French drain pipe is included in trench cost.
12. Duration of Trench Installation based on phone conversation with DeWind On-pass Trenching. 400 feet/day is lowest estimate. Actual rate may be faster.
13. Clay Cover placement unit rate based on RS Means pg. 463 - 1 3/4 CY wheel loader for 8 days
14. Spoil Removal unit rate based on RS Means Pg 217 - hydraulic crawler with 1 CY capacity.
15. Sand placement in trencher unit rate based on RS Means pg. 463 - 3/4 CY wheel loader and select granular fill for 62 days.
16. French drain should be installed along entire length of the south side of the gypsum stack, as installing in sections would be impractical.

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TVA Widows Creek Fossil Plant
Conceptual Cost Estimate
Alternative 7: Bentonite Slurry Seepage Cutoff Wall

Calculated By: AGH
Checked By: JJH
Date: 9/24/2010

Item	Length	Wall Depth	Wall Area	Unit Rate	Unit	Cost	Duration
	ft	ft	sf				
Slurry Wall (Materials + Placement)	5,460	50	273,000	$10.00	sf	$2,730,000	100 ft/day
Contractor Mobilization/Demobilization	-	-	-	$80,000.00	LS	$160,000	
Misc. Construction Support Equipment front end loader to supply trench, ect...	-	-	-	$15,000.00	month	$825,000	55 days
					Total	$3,715,000	
					20% contingency	$743,000	
					Total Cost	**$4,458,000**	

Wetlands Only (BW04):

Item	Length	Wall Depth	Wall Area	Unit Rate	Unit	Cost	Duration
	ft	ft	sf				
Slurry Wall (Materials + Placement)	1,260	50	63,000	$10.00	sf	$630,000	100 ft/day
Contractor Mobilization/Demobilization	-	-	-	$80,000.00	LS	$160,000	
Misc. Construction Support Equipment; front end loader to supply trench, ect...	-	-	-	$15,000.00	month	$825,000	55 days
					Total	$1,615,000	
					20% contingency	$323,000	
					Total Cost	**$1,938,000**	

Notes:
1. All alternatives presented are in comparison to IRRP-URSGS001 (clay dike buttress).
2. Estimate assumes One-Pass Trenching with a wall thickness of 2-ft.
3. Unit rates are based on 2010 pricing estimate from an independent consultant.

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Attachment B

Ash Pond Complex Decision Matrix Memorandum

URS
1375 Euclid Avenue
Cleveland, OH 44115-1808
Tel: 216.622.2400
Fax: 216.622.2428
Architectural and Engineering Services

MEMORANDUM

TO:	Jim Blackburn, TVA	DATE:	4/12/10
BY:	Jay Mokotoff, PE Jeremy Heyerly, PE	PROJECT:	TVA WCF Main Ash Complex
		URS JOB NO.:	31852111.40150
		RE:	Decision Matrix Summary

URS has developed a matrix (spreadsheet) to assist in rating of the various combinations of dredging and rock toe buttress concepts developed for the Main Ash Pond to date. The weighting system (weighted average) is based on **risk, schedule, and cost**. Risk is assessed based on the calculated factor of safety. Preliminary weighting of 60% risk, 10% schedule, and 30% cost have been assigned to the matrix. The matrix is most sensitive to risk; if risk is assigned a weighting less than 50%, the recommended solution may not meet minimum safety factor requirements of 1.5. Refer to the attached Decision Matrix. A summary and recommendations are provided below.

Summary

While final input on the decision matrix values and weighting system will clearly come from TVA, based on URS's evaluation, the outcome of the scoring suggests that the decision may be either "big IRRP #1" with no reduction in normal pool elevation or "little IRRP #1" with a 5 foot reduction in normal pool elevation. We have defined "little IRRP #1" as a toe buttress with an approximate 4 foot total thickness and "big IRRP #1" as a toe buttress with an approximate 7 foot total thickness. Refer to the attached figures indicating preliminary details of both the proposed "little" and "big" IRRP #1, taken at cross-sections J and L.

The relatively high cost of dredging ($10/CY) and the operation buffer volume of 250,000 CY (in addition to the minimum free water volume of 535,000 CY) limit the feasibility of significant reductions in the normal pool elevation. However, dredging to produce some reduction in normal pool elevation warrants additional consideration to:

- Maintain adequate free water and operation buffer volumes to facilitate by-passing the Upper and Lower Stilling Ponds during spillway reconstruction; and,
- Provide a contingency if IRRP #1 construction is delayed by:
    - Potential wetlands or other environmental impacts at the toe of slopes;
    - Excessive schedule for implementation of "big IRRP #1"; and,
    - Potential coordination issues with rock buttress construction at the Gypsum Stack.

On-going evaluations of potentiometric surfaces in the perimeter dikes may result in an increase of existing safety factors, reducing the required size of the IRRP to achieve the target safety factor.

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1375 Euclid Avenue
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Architectural and Engineering Services

Recommendations

1. URS should proceed with development of the IRRP #1 design with alternates for construction of "little IRRP #1", "big IRRP #1", or little IRRP #1 with subsequent construction of the big IRRP #1 (by adding an additional 3 foot thickness), if necessary.

   The alternate IRRP #1 designs ("big or little") have identical footprints (length and width), from Sta. 57+00 to Sta. 118+00 of the main ash complex perimeter. The alternate IRRP #1 designs would also include reverse graded filters (1 foot of well-graded sand overlain by 1 foot of AASHTO No. 89 fine aggregate). The thickness of the overlying armor stone will be varied to develop the design alternatives. Receipt of bid costs, schedules, and other input from TVA's Contractors can be utilized in making a final decision regarding the necessity of additional dredging.

2. A reassessment of immediate dredging needs is in order to maintain the minimum free water volume (535,000 CY) and the additional operational buffer volume (250,000 CY, as estimated by TVA) should be finalized to re-establish the final grades for spillway reconstruction.

   An inlet invert elevation (bottom of the lowest stoplog) between 627.0- and 628.0-feet is likely based on: a) anticipated structure height limitations of 7.0 ft. and b) the use of a maximum of eight (8) 6-inch stop logs. Any immediate reduction in normal pool elevation not requiring dredging should be implemented to improve overall stability and available freeboard.

Attachments:
- Decision Matrix Summary
- Preliminary IRRP #1 Details at Cross-Sections J and L

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Attachment C

Filter Design

(b) STORAGE.

1. Attention is directed to the requirements of Article 106.05 and the following:

The Contractor shall prepare the storage area as needed; any stockpiled material that cannot be removed without including dirt or other foreign matter shall be rejected.

2. Stockpiling shall be as provided by ALDOT-175.

3. Different sizes of aggregate and aggregate from different sources shall be stored in separate stockpiles sufficiently separated from each other so that the material will not become intermixed. Any material which segregates so that the grading no longer conforms to that specified shall be rejected for use.

(c) USE.

1. At the time of their use, the aggregates shall be free from all foreign materials.

2. When more than one size of aggregate is required, the various sizes shall be combined in proper proportions at the mixer or plant.

3. Aggregates stored in proportioning bins shall be protected from rain by waterproof coverings.

(d) COARSE AGGREGATE GRADATION TABLE.

TABLE OF ALDOT COARSE AGGREGATE SIZES *																
PERCENT PASSING BY WEIGHT {MASS}, EACH LABORATORY SIEVE (U.S.A. STANDARD SERIES)																
Size > Number ∨	4 inch {100 mm}	3.5 inch {90 mm}	3 inch {75 mm}	2.5 inch {63 mm}	2 inch {50 mm}	1.5 inch {37.5 mm}	1 inch {25.0 mm}	3/4 inch {19.0 mm}	1/2 inch {12.5 mm}	3/8 inch {9.5 mm}	# 4 {4.75 mm}	# 8 {2.36 mm}	# 16 {1.18 mm}	# 50 {300 μm}	# 100 {150 μm}	# 200 {75 μm}
1	100	90-100		25-60		0-15		0-5								
2			100	90-100	35-70	0-15		0-5								
24			100	90-100		25-60		0-10	0-5							
3				100	90-100	35-70	0-15		0-5							
357				100	95-100		35-70		10-30		0-5					
4					100	90-100	20-55	0-15		0-5						
467					100	95-100		35-70		10-30	0-5					
410					100	85-100	60-85		30-60		18-30	11-20	8-15	5-9		2-6
5						100	90-100	20-55	0-10	0-5						
56						100	90-100	40-85	10-40	0-15	0-5					
57						100	95-100		25-60		0-10	0-5				
6							100	90-100	20-55	0-15	0-5					
67							100	90-100		20-55	0-10	0-5				
68							100	90-100		30-65	5-25	0-10	0-5			
610							100	90-100		25-60		7-30		0-15		
7								100	90-100	40-70	0-15	0-5				
78								100	90-100	40-75	5-25	0-10	0-5			
710								100	90-100	50-85		12-35		0-15		
8									100	85-100	10-30	0-10	0-5			
89									100	90-100	20-55	5-30	0-10	0-5		
810									100		70-90	50-74	38-62	20-42		9-24
8910									100	90-100	60-85	40-70		10-25		1-5
9										100	85-100	10-40	0-10	0-5		
10										100	85-100				10-30	

* Explanation of Table

1. Tabulated figures are percentages by weight {mass} of material finer than each laboratory sieve.

2. Exclusive of lightweight aggregates, the minimum dry rodded weight per cubic foot {mass per cubic meter} shall be 65 pounds {1040 kg} for Sizes 1, 3, and 4, and 70 pounds {1120 kg} for other sizes. See Article 801.12 for weight {mass} of lightweight aggregate.

The following coarse aggregate (gravel only) gradation may be substituted for use in concrete types 2, 3, and 4 for those coarse aggregate size numbers designated in the Master Proportion Table:

Sieve Size (Square Openings)	Percent Passing By Weight {Mass}
1.5 inches {37.5 mm}	100
1 inch {25.0 mm}	80 – 100
3/4 inch {19.0 mm}	70 – 100
1/2 inch {12.5 mm}	25 - 80
# 4 {4.75 mm}	0 - 15
# 8 {2.36 mm}	0 - 10

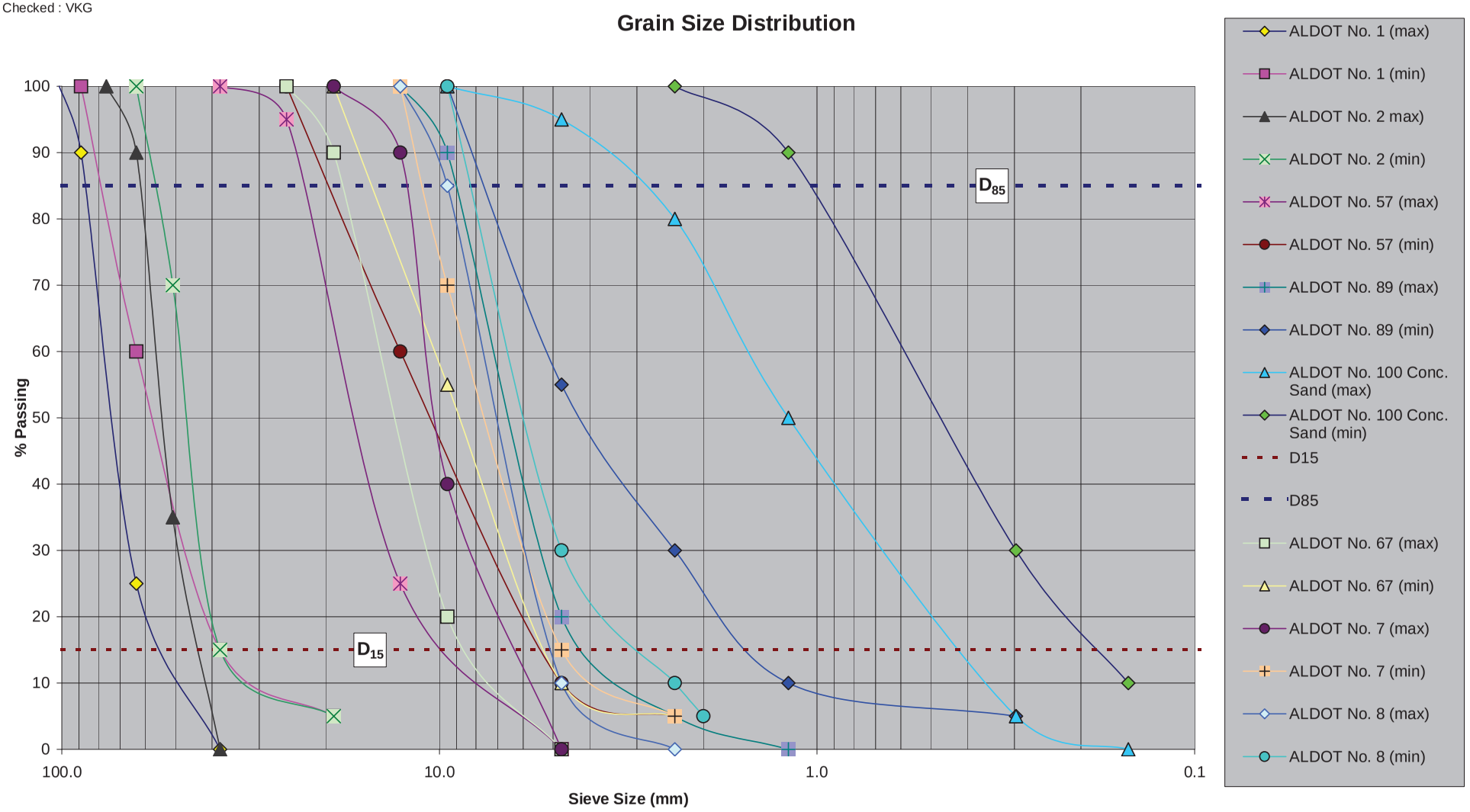
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WCF AC Proposed Buttress Design
Check Reverse Graded Filter Capacity

5/20/10

Computed: AGH
Checked : VKG



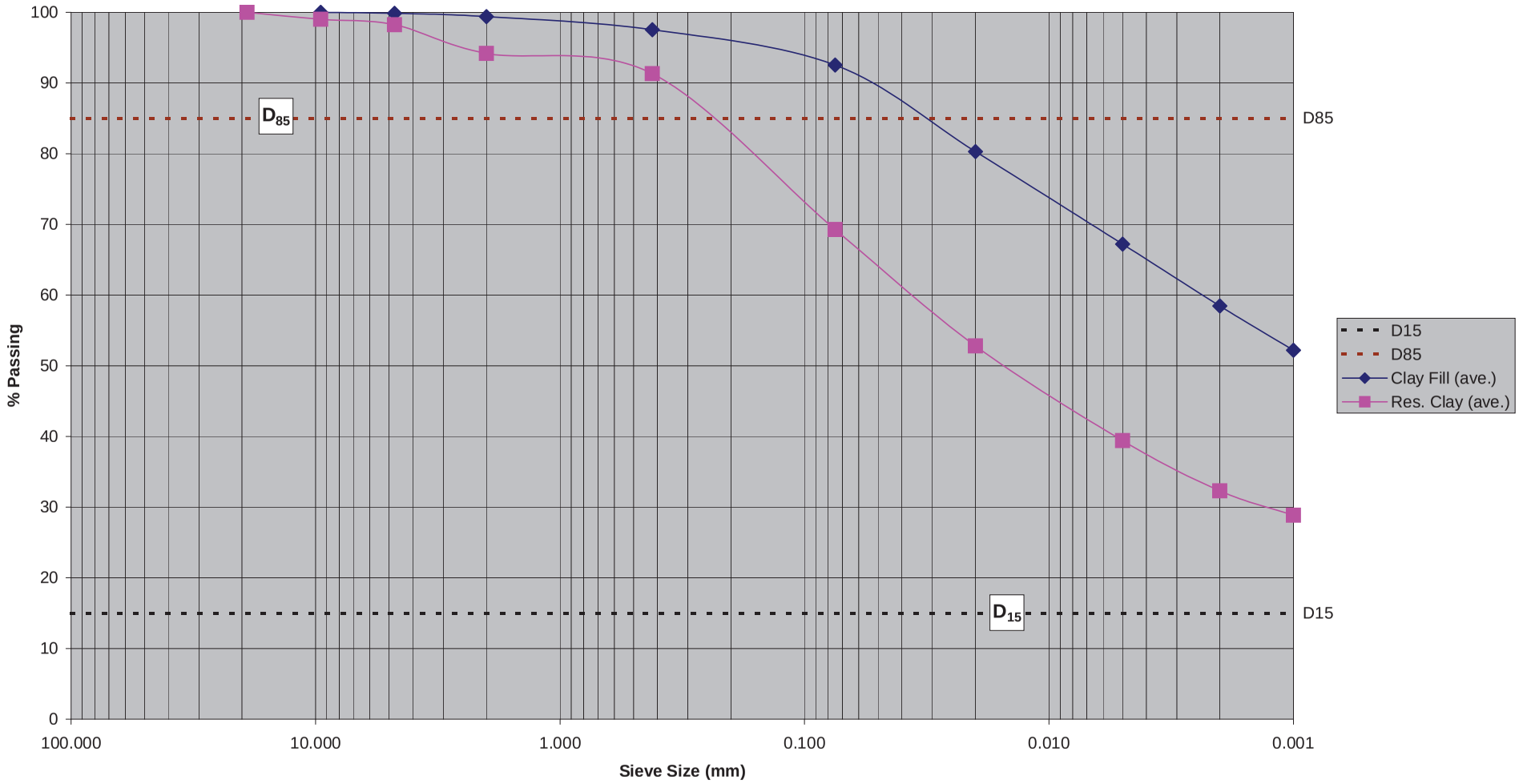
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WCF AC Proposed Buttress Design
Check Reverse Graded Filter Capacity

5/20/10

Computed: AGH
Checked : VKG

Grain Size Distribution



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WCF AC Proposed Buttress Design
Check Reverse Graded Filter Capacity

5/20/10

Computed: AGH
Checked : VKG

		% Passing																							
Ref.	**Stone Type**	**4 in**	**3.5 in**	**3 in**	**2.5 in**	**2 in**	**1.5 in**	**1 in**	**3/4 in**	**1/2 in**	**3/8 in**	**# 4**	**# 8**	**#10**	**# 16**	**#40**	**# 50**	**# 100**	**# 200**	**0.02 mm**	**0.005 mm**	**0.002 mm**	**0.001 mm**	**d_{85}**	**D_{15}**
	Sieve Size (mm)	**101.6**	**88.9**	**76.2**	**63.5**	**50.8**	**38.1**	**25.4**	**19.1**	**12.7**	**9.5**	**4.7**	**2.4**	**2.0**	**1.2**	**0.4**	**0.3**	**0.1**	**0.075**	**0.020**	**0.005**	**0.002**	**0.001**		
ALDOT	ALDOT No. 1 (max)	100	90		25.00		0.00																	SEE CHARTS	SEE CHARTS
	ALDOT No. 1 (min)		100		60.00		15.00		5.00																
	ALDOT No. 2 max)			100.00	90.00	35.00	0.00																		
	ALDOT No. 2 (min)				100.00	70.00	15.00		5.00																
	ALDOT No. 57 (max)						100.00	95.00		25.00		0.00													
	ALDOT No. 57 (min)							100.00		60.00		10.00	5.00												
	ALDOT No. 67 (max)							100.00	90.00		20.00	0.00													
	ALDOT No. 67 (min)								100.00		55.00	10.00	5.00												
	ALDOT No. 7 (max)								100.00	90.00	40.00	0.00													
	ALDOT No. 7 (min)									100.00	70.00	15.00	5.00												
	ALDOT No. 8 (max)									100.00	85.00	10.00	0.00												
	ALDOT No. 8 (min)										100.00	30.00	10.00	5.00											
	ALDOT No. 89 (max)									100.00	90.00	20.00	5.00		0.00										
	ALDOT No. 89 (min)										100.00	55.00	30.00		10.00		5.00								
	ALDOT No. 100 Conc. Sand (max)										100.00	95.00	80.00		50.00		5.00	0.00							
	ALDOT No. 100 Conc. Sand (min)												100.00		90.00		30.00	10.00							
Ave. Lab Data	Res. Clay (ave.)								100.00		99.03	98.25		94.18		91.33			69.28	52.80	39.40	32.30	28.85	0.560	n/a
	Clay Fill (ave.)										100.00	99.85		99.40		97.53			92.53	80.30	67.20	58.47	52.20	0.033	n/a

WCF AC Proposed Buttress Design
Check Reverse Graded Filter Capacity

5/20/10

Computed: AGH
Checked : VKG

Clay Dike (Fill) to Conc. Sand				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
0.033	0.410	1	$D_{15} \le 9 \times d_{85}$	OK

Redidual Clay to Conc. Sand				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
0.560	0.410	1	$D_{15} \le 9 \times d_{85}$	OK

Conc. Sand to 57				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
1.300	10.000	4	$D_{15} \le 4to5 \times d_{85}$	FAILS

Conc. Sand to 67				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
1.300	8.800	4	$D_{15} \le 4to5 \times d_{85}$	FAILS

Conc. Sand to 7				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
1.300	6.200	4	$D_{15} \le 4to5 \times d_{85}$	OK

Conc. Sand to 8				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
1.300	5.100	4	$D_{15} \le 4to5 \times d_{85}$	OK

89 to 2				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
2.900	44.000	4	$D_{15} \le 4to5 \times d_{85}$	FAILS

89 to 1				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
2.900	56.000	4	$D_{15} \le 4to5 \times d_{85}$	FAILS

57 to 2				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
19.000	44.000	4	$D_{15} \le 4to5 \times d_{85}$	OK

57 to 1				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
19.000	56.000	4	$D_{15} \le 4to5 \times d_{85}$	OK

8 to 1				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
9.500	56.000	4	$D_{15} \le 4to5 \times d_{85}$	FAILS

8 to 2				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
9.500	44.000	4	$D_{15} \le 4to5 \times d_{85}$	OK

7 to 1				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
12.500	56.000	4	$D_{15} \le 4to5 \times d_{85}$	OK

7 to 2				
$d_{85\ (min)}$	$D_{15\ (max)}$	Category	USACE Gradaiton Requirement	Design Check
12.500	44.000	4	$D_{15} \le 4to5 \times d_{85}$	OK

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WCF AC Proposed Buttress Design
Check Reverse Graded Filter Capacity

5/20/10

Computed: AGH
Checked : VKG

	% Passing																					
Boring No.	**4 in**	**3.5 in**	**3 in**	**2.5 in**	**2 in**	**1.5 in**	**1 in**	**3/4 in**	**1/2 in**	**3/8 in**	**# 4**	**# 8**	**#10**	**# 16**	**#40**	**# 50**	**# 100**	**# 200**	**0.02 mm**	**0.005 mm**	**0.002 mm**	**0.001 mm**	**d_{85}**
Sieve Size (mm)			**75**		**50**	**37.5**	**25**	**19**		**9.5**	**4.75**		**2**		**0.425**			**0.075**	**0.02**	**0.005**	**0.002**	**0.001**	
66-Res. Clay (Sec C)										100.00	99.90		99.90		99.70			94.00	72.80	49.50	39.30	33.00	0.0380
108-Res. Clay (Sec C)										100.00	99.90		99.90		99.60			45.40	25.20	15.70	11.20	10.00	0.2500
73-Res. Clay (Sec J)							100.00	97.50		91.70	76.50		57.00		43.40			32.50	27.00	20.10	16.10	14.20	7.0000
81-Res. Clay (Sec M)								100.00		97.50	95.70		90.30		84.60			65.00	49.90	39.80	31.10	28.40	0.4500
84- Res. Clay (Sec O)								100.00		98.60	97.50		86.60		81.40			72.70	63.30	52.60	47.60	44.00	1.5000
85-Clay Fill (Sec O)										100.00	99.90		98.90		97.10			91.70	81.30	73.40	65.80	59.00	0.0310
80-Clay Fill (Sec M)													100.00		99.20			96.00	83.30	66.60	56.00	49.60	0.0270
64-Clay Fill (Sec A)										100.00	99.80		99.30		96.30			89.90	76.30	61.60	53.60	48.00	0.0400
101-Clay Fill (Sec D)							100.00	97.30		95.90	93.70		91.30		87.50			66.60	44.40	30.40	23.70	20.00	0.3100

Data not included in analysis

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Attachment D

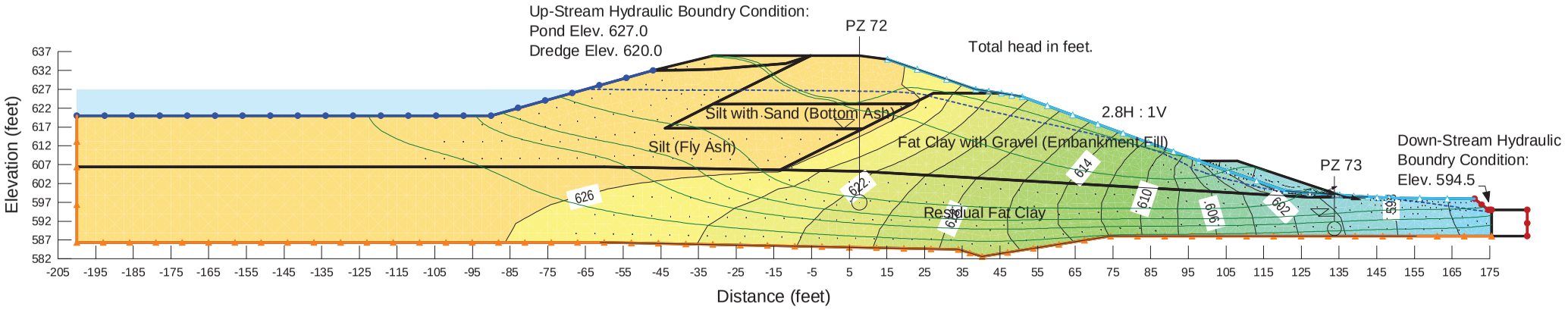
GeoStudio Output Files and Stability Figures

US EPA ARCHIVE DOCUMENT

Widows Creek Ash Pond
Section J - Main Ash Pond
Seepage Analysis; After Buttress Construction
Total Stress

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

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Name: Stability Butress Model: Saturated / Unsaturated K-Function: Stability Butress Vol. WC. Function: Stability Butress K-Ratio: 1 K-Direction: 0 °

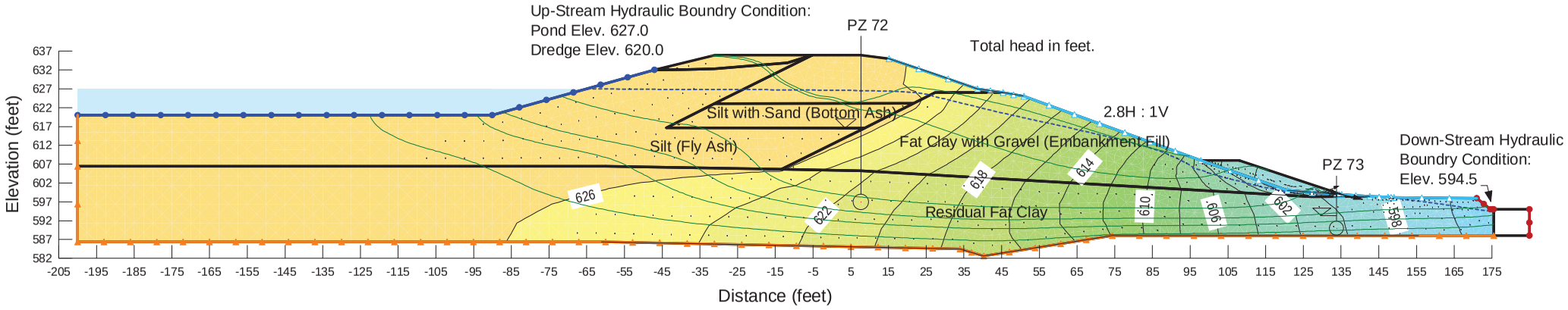


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Widows Creek Ash Pond
Section J - Main Ash Pond
Seepage Analysis; After Buttress Construction

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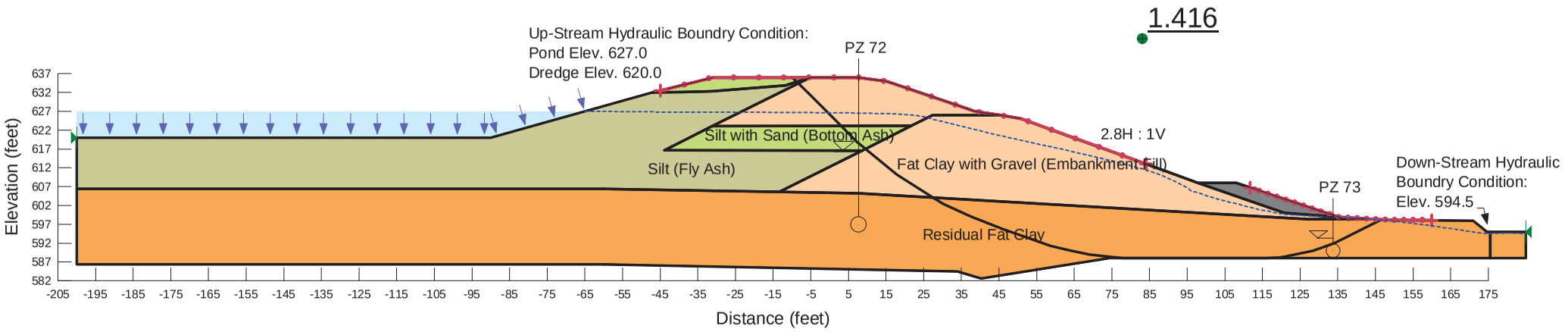


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Widows Creek Ash Pond
Section J - Main Ash Pond
Slope Stability; After IRRP Construction
Exit & Entry Optimized
Total Stress

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Name: Silt (Fly Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 27.5 ° Phi-B: 0 °
Name: Stability Butress Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion: 0 psf Phi: 40 ° Phi-B: 0 °

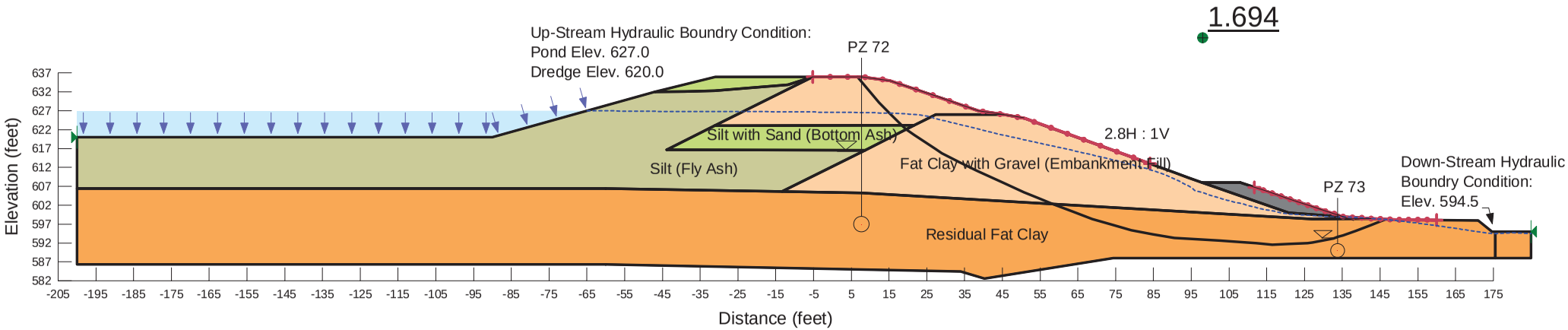


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Widows Creek Ash Pond
Section J - Main Ash Pond
Slope Stability; After IRRP Construction
Exit & Entry Optimized

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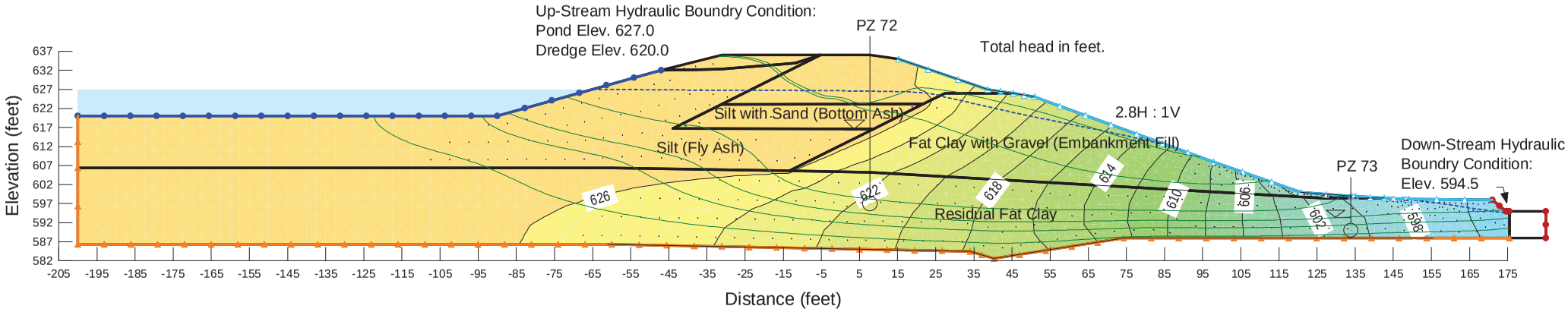


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Widows Creek Ash Pond
Section J - Main Ash Pond
Seepage Analysis; After Dredging

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Name: Residual Fat Clay Model: Saturated / Unsaturated K-Function: Residual Fat Clay Vol. WC. Function: Residual Fat Clay K-Ratio: 0.1 K-Direction: 0 °
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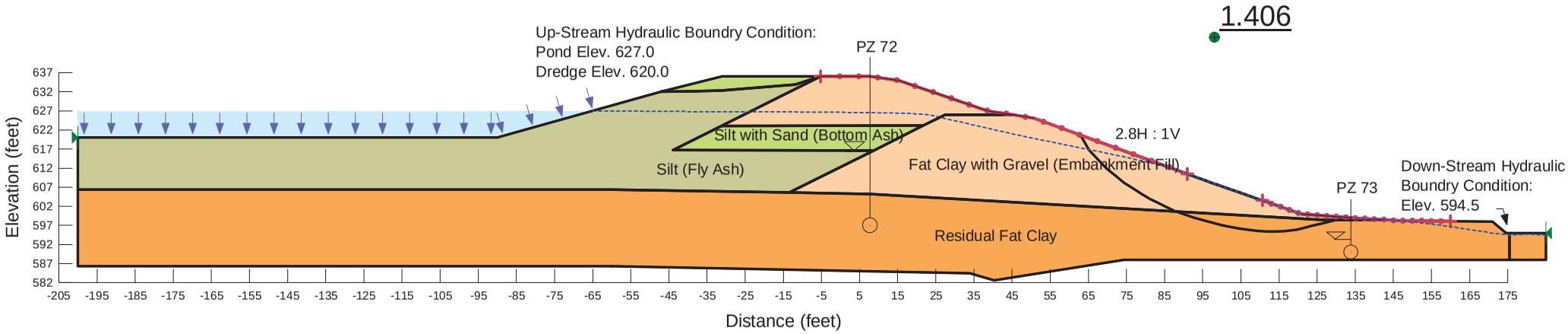


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Widows Creek Ash Pond
Section J - Main Ash Pond
Slope Stability Analysis; After Dredging
Exit & Entry Optimized

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Name: Silt (Fly Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 27.5 ° Phi-B: 0 °

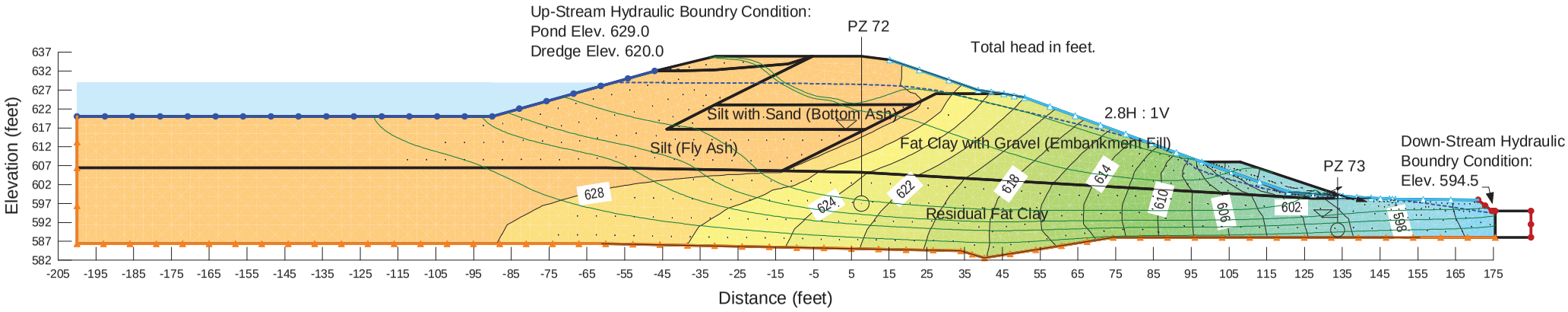


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Widows Creek Ash Pond
Section J - Main Ash Pond
Seepage Analysis; After Buttress Construction

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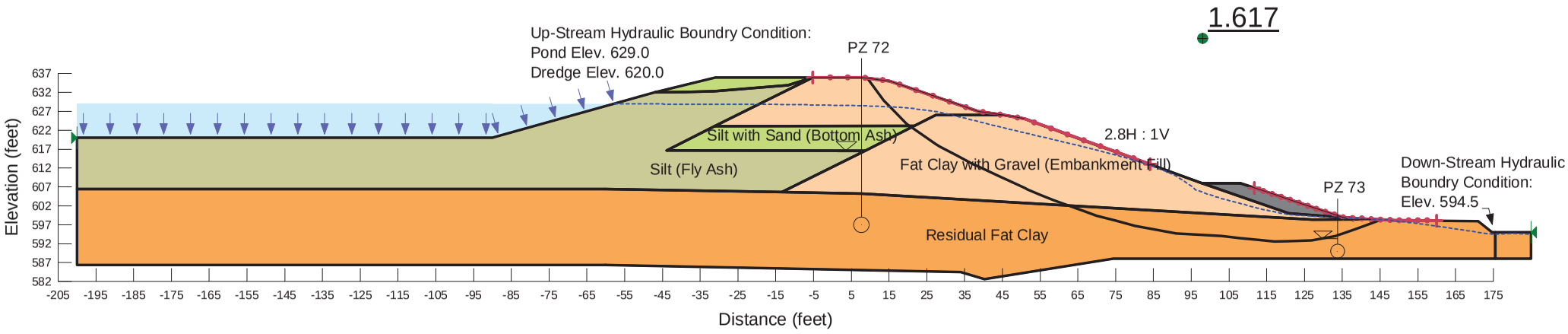


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Widows Creek Ash Pond
Section J - Main Ash Pond
Slope Stability; After Buttress Construction
Exit & Entry Optimized

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Name: Stability Butress Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion: 0 psf Phi: 40 ° Phi-B: 0 °

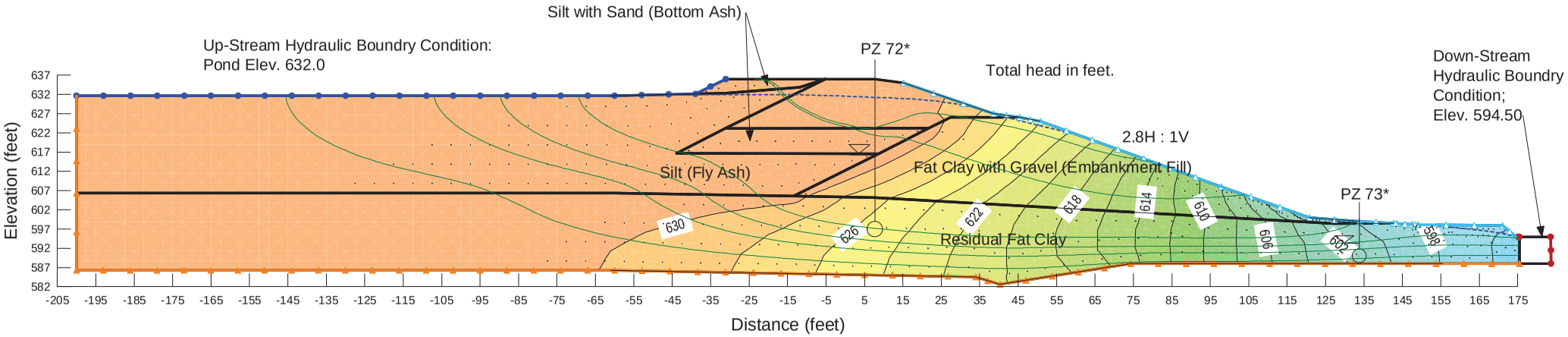


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Widows Creek Ash Pond
Section J - Main Ash Pond
Seepage Analysis; Existing Conditions

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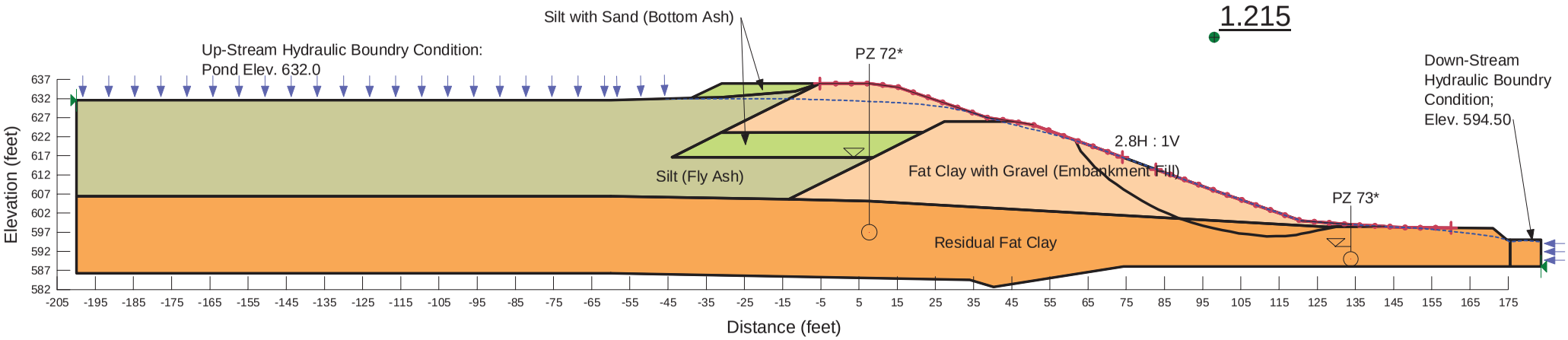
* (Stantec, Report of Phase II
Geotechnical Exploration, 2/4/10)

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Widows Creek Ash Pond
Section J - Main Ash Pond
Slope Stability Analysis; Existing Conditions
Exit & Entry Optimized

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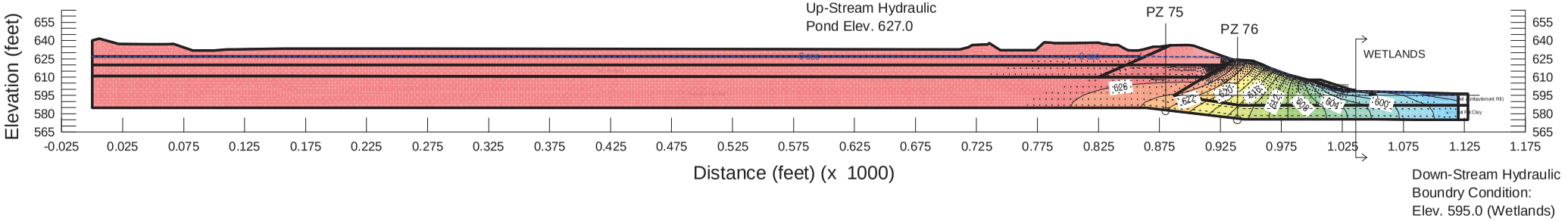
* (Stantec, Report of Phase II Geotechnical Exploration, 2/4/10)

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Widows Creek Ash Pond
Section K - Main Ash Pond
Seepage Analysis; After IRRP Construction
Total Stress

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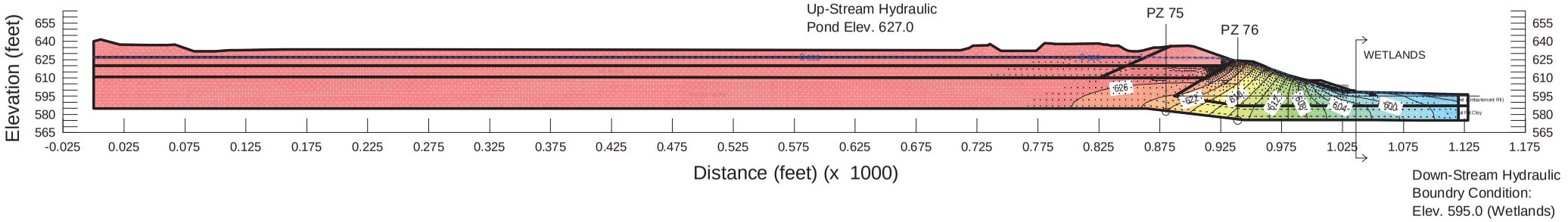
Down-Stream Hydraulic
Boundry Condition:
Elev. 595.0 (Wetlands)

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Widows Creek Ash Pond
Section K - Main Ash Pond
Seepage Analysis; After IRRP Construction

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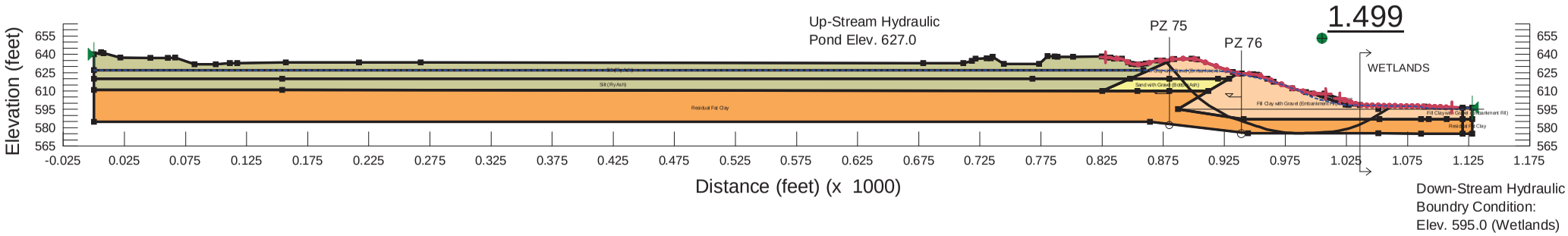


US EPA ARCHIVE DOCUMENT

Widows Creek Ash Pond
Section K - Main Ash Pond
Slope Stability; After IRRP Construction
Exit & Entry Optimized
Total Stress

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Name: Stability Butress Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion: 0 psf Phi: 40 ° Phi-B: 0 °

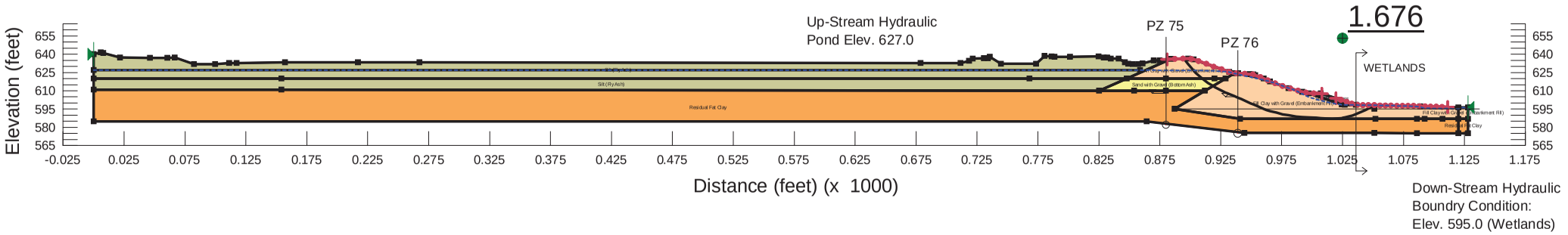


US EPA ARCHIVE DOCUMENT

Widows Creek Ash Pond
Section K - Main Ash Pond
Slope Stability; After IRRP Construction
Exit & Entry Optimized

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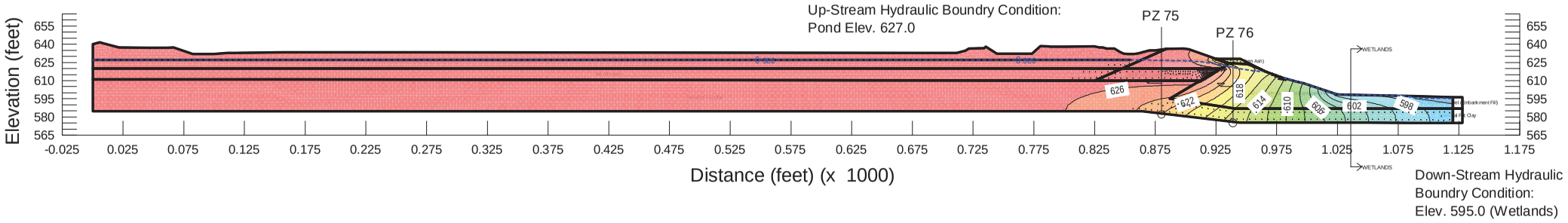


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Widows Creek Ash Pond
Section K - Main Ash Pond
Seepage Analysis; After Drawdown Temporary Access Road

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Name: Fill Clay with Gravel (Embankment Fill) Model: Saturated / Unsaturated K-Function: Fat Clay with Gravel (Embankment Fill) Vol. WC. Function: Fat Clay with Gravel (Embankment Fill) K-Ratio: 0.25 K-Direction: 0 °
Name: Residual Fat Clay Model: Saturated / Unsaturated K-Function: Residual Fat Clay Vol. WC. Function: Residual Fat Clay K-Ratio: 0.1 K-Direction: 0 °
Name: Silt (Fly Ash) Model: Saturated / Unsaturated K-Function: Silt (Fly Ash) Vol. WC. Function: Silt (Fly Ash) K-Ratio: 0.1 K-Direction: 0 °

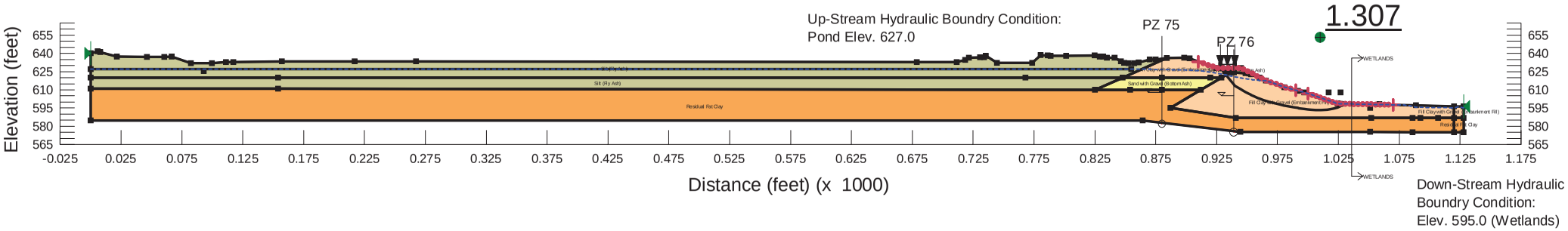


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Widows Creek Ash Pond
Section K - Main Ash Pond
Slope Stability; After Drawdown Temporary Access Road Exit & Entry Optimized

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Mohr-Coulomb Unit Weight: 119 pcf Cohesion: 0 psf Phi: 33 ° Phi-B: 0 °
Name: Silt with Sand (Bottom Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 33 ° Phi-B: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 150 psf Phi: 29 ° Phi-B: 0 °
Name: Residual Fat Clay Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 50 psf Phi: 33 ° Phi-B: 0 °
Name: Silt (Fly Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 27.5 ° Phi-B: 0 °

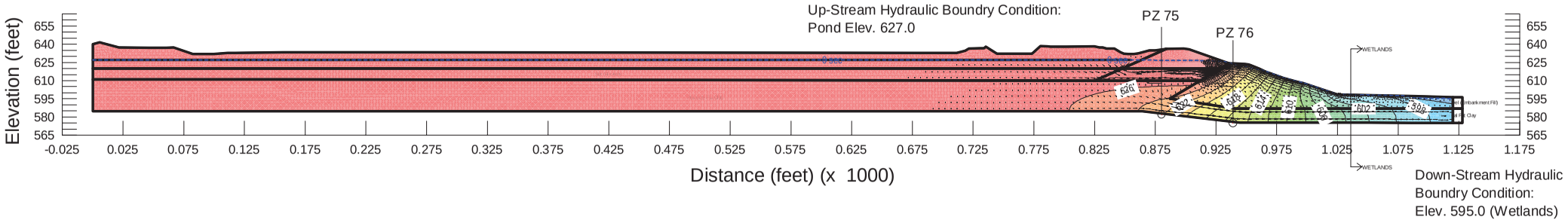


US EPA ARCHIVE DOCUMENT

Widows Creek Ash Pond
Section K - Main Ash Pond
Seepage Analysis; After Drawdown

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Saturated / Unsaturated K-Function: Sand with Gravel (Bottom Ash) Vol. WC. Function: Sand with Gravel (Bottom Ash) K-Ratio: 0.1 K-Direction: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Saturated / Unsaturated K-Function: Fat Clay with Gravel (Embankment Fill) Vol. WC. Function: Fat Clay with Gravel (Embankment Fill) K-Ratio: 0.25 K-Direction: 0 °
Name: Residual Fat Clay Model: Saturated / Unsaturated K-Function: Residual Fat Clay Vol. WC. Function: Residual Fat Clay K-Ratio: 0.1 K-Direction: 0 °
Name: Silt (Fly Ash) Model: Saturated / Unsaturated K-Function: Silt (Fly Ash) Vol. WC. Function: Silt (Fly Ash) K-Ratio: 0.1 K-Direction: 0 °

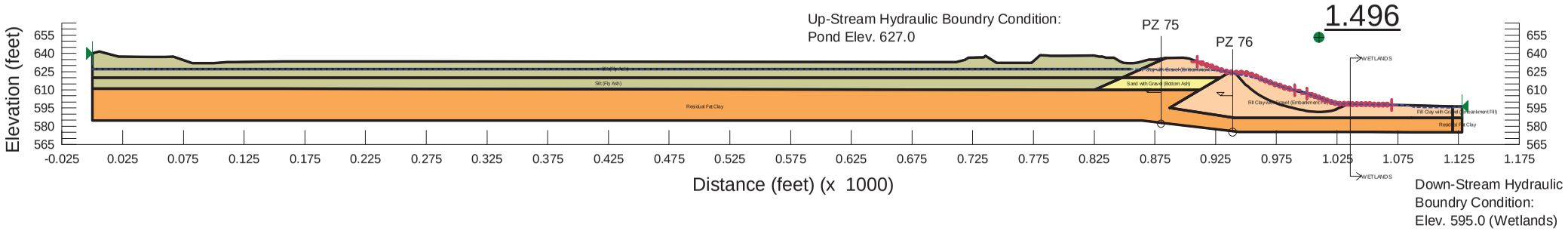


US EPA ARCHIVE DOCUMENT

Widows Creek Ash Pond
Section K - Main Ash Pond
Slope Stability; After Drawdown
Exit & Entry Optimized

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Mohr-Coulomb Unit Weight: 119 pcf Cohesion: 0 psf Phi: 33 ° Phi-B: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 150 psf Phi: 29 ° Phi-B: 0 °
Name: Residual Fat Clay Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 50 psf Phi: 33 ° Phi-B: 0 °
Name: Silt (Fly Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 27.5 ° Phi-B: 0 °

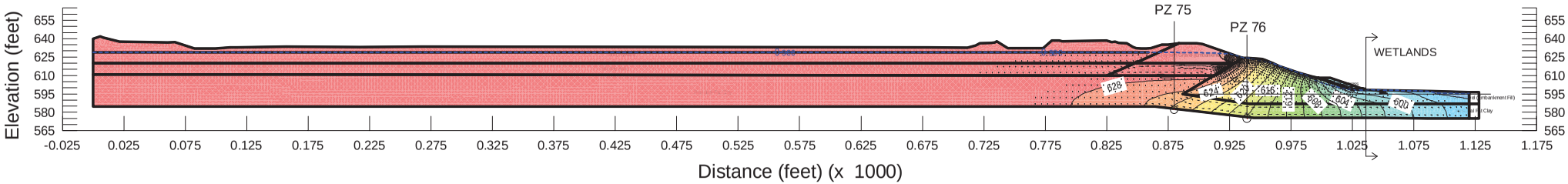


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Widows Creek Ash Pond
Section K - Main Ash Pond
Seepage Analysis; After Burtress Construction

Name: Sand with Gravel (Bottom Ash) Model: Saturated / Unsaturated K-Function: Sand with Gravel (Bottom Ash) Vol. WC. Function: Sand with Gravel (Bottom Ash) K-Ratio: 0.1 K-Direction: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Saturated / Unsaturated K-Function: Fat Clay with Gravel (Embankment Fill) Vol. WC. Function: Fat Clay with Gravel (Embankment Fill) K-Ratio: 0.25 K-Direction: 0 °
Name: Residual Fat Clay Model: Saturated / Unsaturated K-Function: Residual Fat Clay Vol. WC. Function: Residual Fat Clay K-Ratio: 0.1 K-Direction: 0 °
Name: Silt (Fly Ash) Model: Saturated / Unsaturated K-Function: Silt (Fly Ash) Vol. WC. Function: Silt (Fly Ash) K-Ratio: 0.1 K-Direction: 0 °
Name: Stability Butress Model: Saturated / Unsaturated K-Function: Stability Butress Vol. WC. Function: Stability Butress K-Ratio: 1 K-Direction: 0 °

This analysis is based on electronic SEEP/W & SLOPE/W files provided to URS by the Tennessee Valley Authority. The electronic files were origionally produced by STANTEC. The models were modified to add additional SLOPE/W stability analysis with updated geometry to show recommended repairs.

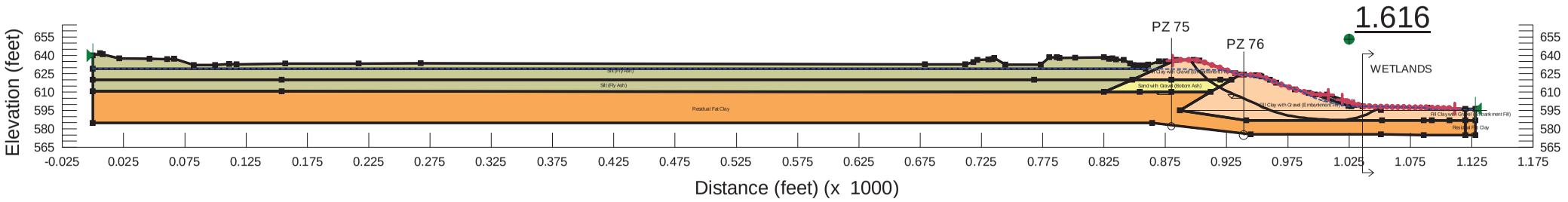


US EPA ARCHIVE DOCUMENT

Widows Creek Ash Pond
Section K - Main Ash Pond
Slope Stability Analysis; After Burtress Construction
Exit & Entry Optimized

Name: Sand with Gravel (Bottom Ash) Model: Mohr-Coulomb Unit Weight: 119 pcf Cohesion: 0 psf Phi: 33 ° Phi-B: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 150 psf Phi: 29 ° Phi-B: 0 °
Name: Residual Fat Clay Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 50 psf Phi: 33 ° Phi-B: 0 °
Name: Silt (Fly Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 27.5 ° Phi-B: 0 °
Name: Stability Butress Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion: 0 psf Phi: 40 ° Phi-B: 0 °

This analysis is based on electronic SEEP/W & SLOPE/W files provided to URS by the Tennessee Valley Authority. The electronic files were origionally produced by STANTEC. The models were modified to add additional SLOPE/W stability analysis with updated geometry to show recommended repairs.

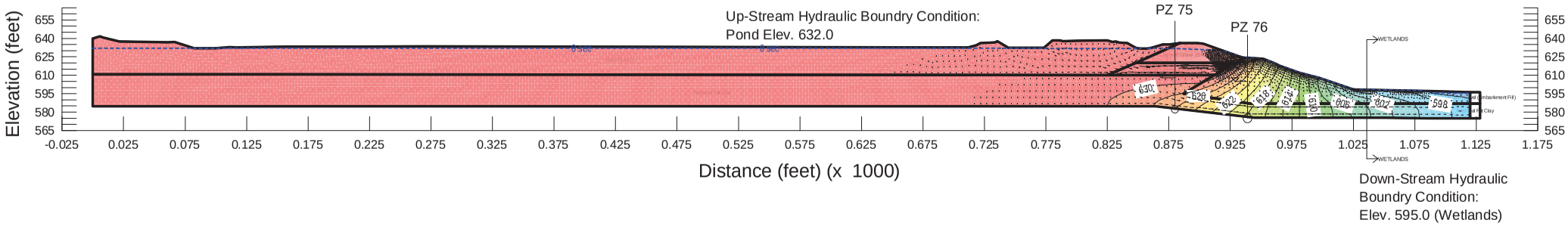


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Widows Creek Ash Pond
Section K - Main Ash Pond
Seepage Analysis; Existing Conditions

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Saturated / Unsaturated K-Function: Sand with Gravel (Bottom Ash) Vol. WC. Function: Sand with Gravel (Bottom Ash) K-Ratio: 0.1 K-Direction: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Saturated / Unsaturated K-Function: Fat Clay with Gravel (Embankment Fill) Vol. WC. Function: Fat Clay with Gravel (Embankment Fill) K-Ratio: 0.25 K-Direction: 0 °
Name: Residual Fat Clay Model: Saturated / Unsaturated K-Function: Residual Fat Clay Vol. WC. Function: Residual Fat Clay K-Ratio: 0.1 K-Direction: 0 °
Name: Silt (Fly Ash) Model: Saturated / Unsaturated K-Function: Silt (Fly Ash) Vol. WC. Function: Silt (Fly Ash) K-Ratio: 0.1 K-Direction: 0 °

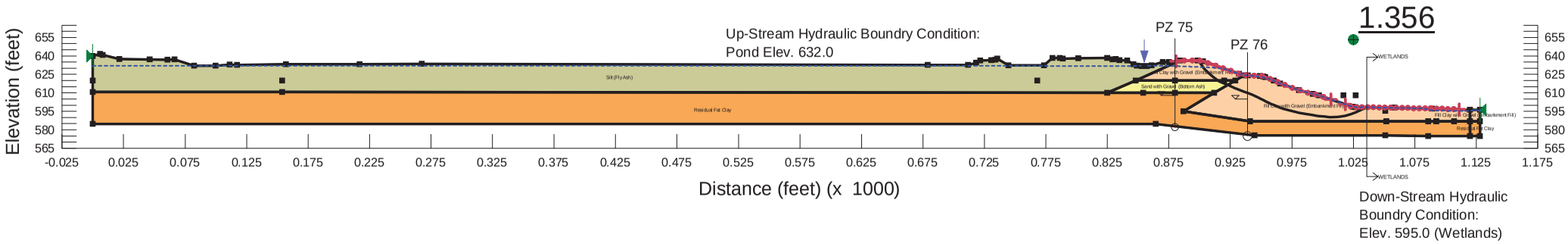


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Widows Creek Ash Pond
Section K - Main Ash Pond
Slope Stability; Existing Conditions
Exit & Entry Optimized

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Mohr-Coulomb Unit Weight: 119 pcf Cohesion: 0 psf Phi: 33 ° Phi-B: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 150 psf Phi: 29 ° Phi-B: 0 °
Name: Residual Fat Clay Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 50 psf Phi: 33 ° Phi-B: 0 °
Name: Silt (Fly Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 27.5 ° Phi-B: 0 °

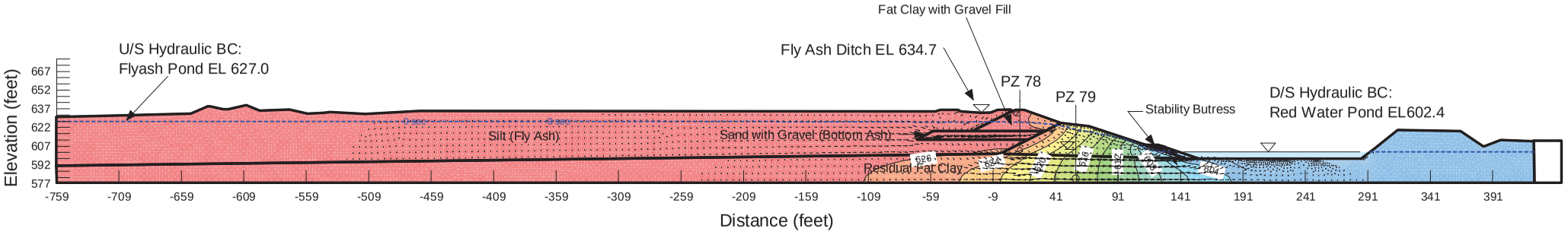


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Widows Creek Ash Pond
Section L - Main Ash Pond
Seepage Analysis; After IRRP Construction

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Saturated / Unsaturated K-Function: Sand with Gravel (Bottom Ash) Vol. WC. Function: Sand with Gravel (Bottom Ash) K-Ratio: 0.1 K-Direction: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Saturated / Unsaturated K-Function: Fat Clay with Gravel (Embankment Fill) Vol. WC. Function: Fat Clay with Gravel (Embankment Fill) K-Ratio: 0.25 K-Direction: 0 °
Name: Residual Fat Clay Model: Saturated / Unsaturated K-Function: Residual Fat Clay Vol. WC. Function: Residual Fat Clay K-Ratio: 0.1 K-Direction: 0 °
Name: Silt (Fly Ash) Model: Saturated / Unsaturated K-Function: Silt (Fly Ash) Vol. WC. Function: Silt (Fly Ash) K-Ratio: 0.1 K-Direction: 0 °
Name: Stability Butress Model: Saturated / Unsaturated K-Function: Stability Butress Vol. WC. Function: Stability Butress K-Ratio: 1 K-Direction: 0 °

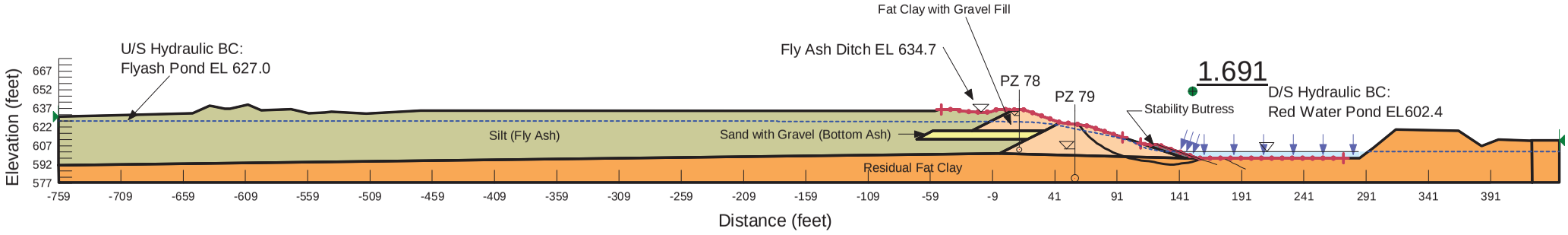


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Widows Creek Ash Pond
Section L - Main Ash Pond
Slope Stability; After IRRP Construction
Exit & Entry Optimized

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Mohr-Coulomb Unit Weight: 119 pcf Cohesion: 0 psf Phi: 33 ° Phi-B: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 150 psf Phi: 29 ° Phi-B: 0 °
Name: Residual Fat Clay Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 50 psf Phi: 33 ° Phi-B: 0 °
Name: Silt (Fly Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 27.5 ° Phi-B: 0 °
Name: Stability Butress Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion: 0 psf Phi: 40 ° Phi-B: 0 °

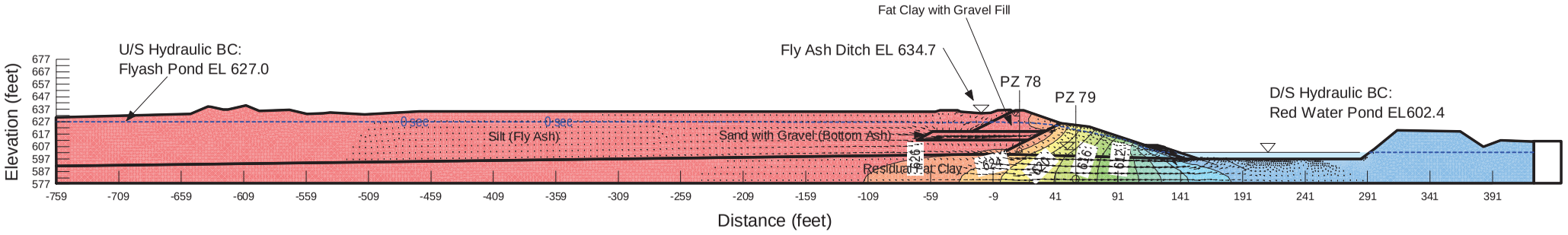


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Widows Creek Ash Pond
Section L - Main Ash Pond
Seepage Analysis; After IRRP Construction

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Saturated / Unsaturated K-Function: Sand with Gravel (Bottom Ash) Vol. WC. Function: Sand with Gravel (Bottom Ash) K-Ratio: 0.1 K-Direction: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Saturated / Unsaturated K-Function: Fat Clay with Gravel (Embankment Fill) Vol. WC. Function: Fat Clay with Gravel (Embankment Fill) K-Ratio: 0.25 K-Direction: 0 °
Name: Residual Fat Clay Model: Saturated / Unsaturated K-Function: Residual Fat Clay Vol. WC. Function: Residual Fat Clay K-Ratio: 0.1 K-Direction: 0 °
Name: Silt (Fly Ash) Model: Saturated / Unsaturated K-Function: Silt (Fly Ash) Vol. WC. Function: Silt (Fly Ash) K-Ratio: 0.1 K-Direction: 0 °
Name: Stability Butress Model: Saturated / Unsaturated K-Function: Stability Butress Vol. WC. Function: Stability Butress K-Ratio: 1 K-Direction: 0 °

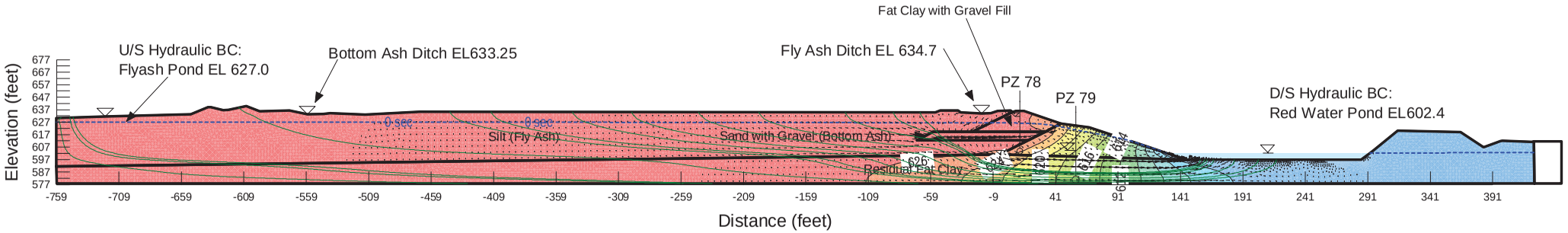


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Widows Creek Ash Pond
Section L - Main Ash Pond
Seepage Analysis; Existing Conditions

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Saturated / Unsaturated K-Function: Sand with Gravel (Bottom Ash) Vol. WC. Function: Sand with Gravel (Bottom Ash) K-Ratio: 0.1 K-Direction: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Saturated / Unsaturated K-Function: Fat Clay with Gravel (Embankment Fill) Vol. WC. Function: Fat Clay with Gravel (Embankment Fill) K-Ratio: 0.25 K-Direction: 0 °
Name: Residual Fat Clay Model: Saturated / Unsaturated K-Function: Residual Fat Clay Vol. WC. Function: Residual Fat Clay K-Ratio: 0.1 K-Direction: 0 °
Name: Silt (Fly Ash) Model: Saturated / Unsaturated K-Function: Silt (Fly Ash) Vol. WC. Function: Silt (Fly Ash) K-Ratio: 0.1 K-Direction: 0 °

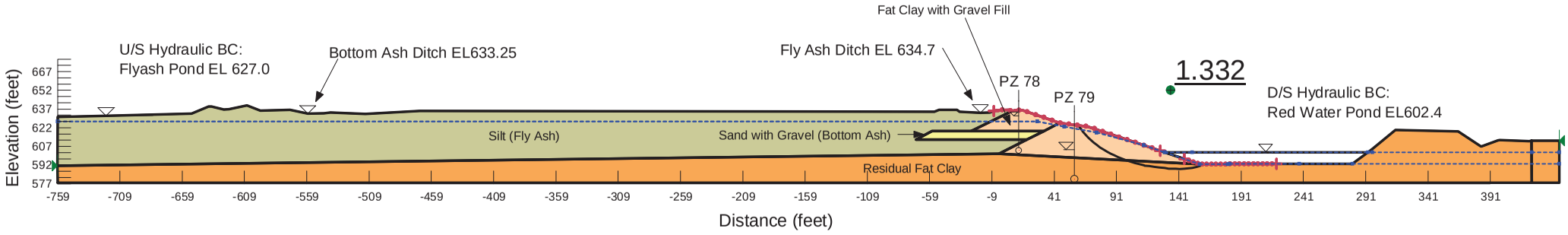


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Widows Creek Ash Pond
Section L - Main Ash Pond
Slope Stability; Drawdown of Red Water Pond
Exit & Entry Optimized

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Mohr-Coulomb Unit Weight: 119 pcf Cohesion: 0 psf Phi: 33 ° Phi-B: 0 ° Drawdown Total Cohesion: 0 psf Drawdown Total Phi: 0 ° Piezometric Line: 1 Piezometric Line After Drawdown: 2
Name: Fill Clay with Gravel (Embankment Fill) Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 150 psf Phi: 29 ° Phi-B: 0 ° Drawdown Total Cohesion: 723 psf Drawdown Total Phi: 14.1 ° Piezometric Line: 1 Piezometric Line After Drawdown: 2
Name: Residual Fat Clay Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 50 psf Phi: 33 ° Phi-B: 0 ° Drawdown Total Cohesion: 211 psf Drawdown Total Phi: 19.6 ° Piezometric Line: 1 Piezometric Line After Drawdown: 2
Name: Silt (Fly Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 27.5 ° Phi-B: 0 ° Drawdown Total Cohesion: 0 psf Drawdown Total Phi: 0 ° Piezometric Line: 1 Piezometric Line After Drawdown: 2

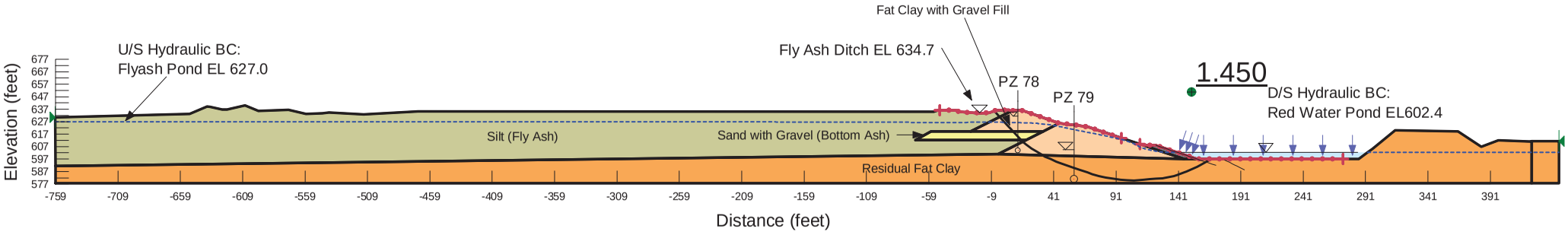


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Widows Creek Ash Pond
Section L - Main Ash Pond
Slope Stability; After IRRP Construction
Exit & Entry Optimized

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Mohr-Coulomb Unit Weight: 119 pcf Cohesion: 0 psf Phi: 33 ° Phi-B: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 500 psf Phi: 15.5 ° Phi-B: 0 °
Name: Residual Fat Clay Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 211 psf Phi: 19.6 ° Phi-B: 0 °
Name: Silt (Fly Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 27.5 ° Phi-B: 0 °
Name: Stability Butress Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion: 0 psf Phi: 40 ° Phi-B: 0 °

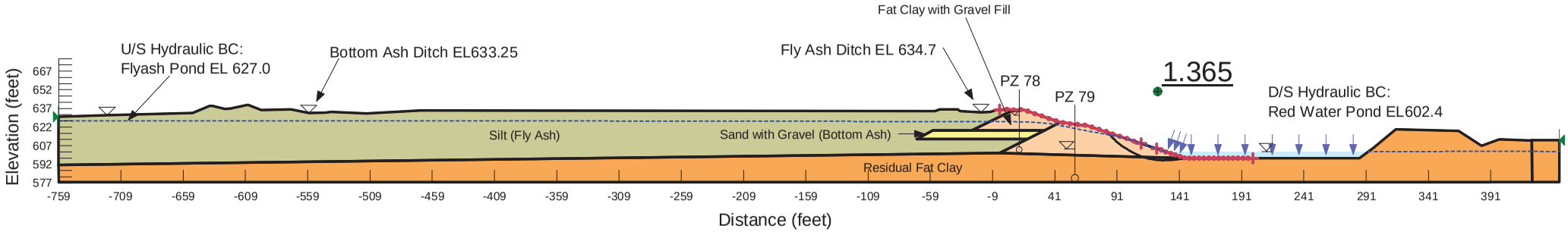


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Widows Creek Ash Pond
Section L - Main Ash Pond
Slope Stability; After Drawdown
Exit & Entry Optimized

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Mohr-Coulomb Unit Weight: 119 pcf Cohesion: 0 psf Phi: 33 ° Phi-B: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 150 psf Phi: 29 ° Phi-B: 0 °
Name: Residual Fat Clay Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 50 psf Phi: 33 ° Phi-B: 0 °
Name: Silt (Fly Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 27.5 ° Phi-B: 0 °

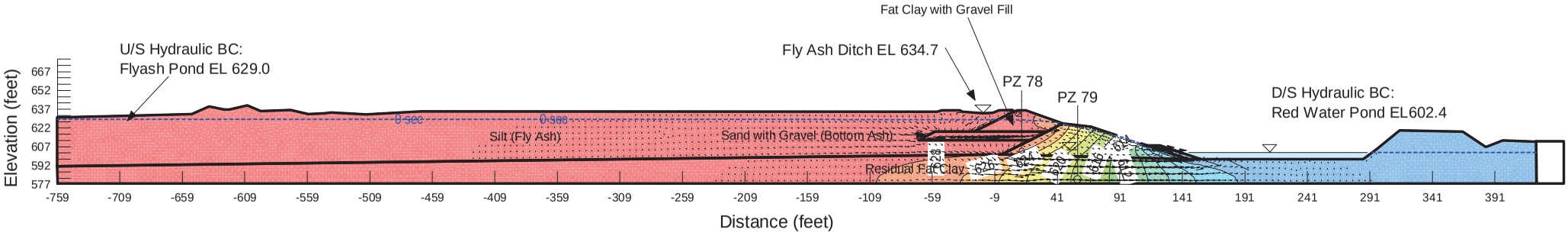


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Widows Creek Ash Pond
Section L - Main Ash Pond
Seepage Analysis; After IRRP Construction

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Saturated / Unsaturated K-Function: Sand with Gravel (Bottom Ash) Vol. WC. Function: Sand with Gravel (Bottom Ash) K-Ratio: 0.1 K-Direction: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Saturated / Unsaturated K-Function: Fat Clay with Gravel (Embankment Fill) Vol. WC. Function: Fat Clay with Gravel (Embankment Fill) K-Ratio: 0.25 K-Direction: 0 °
Name: Residual Fat Clay Model: Saturated / Unsaturated K-Function: Residual Fat Clay Vol. WC. Function: Residual Fat Clay K-Ratio: 0.1 K-Direction: 0 °
Name: Silt (Fly Ash) Model: Saturated / Unsaturated K-Function: Silt (Fly Ash) Vol. WC. Function: Silt (Fly Ash) K-Ratio: 0.1 K-Direction: 0 °
Name: Stability Butress Model: Saturated / Unsaturated K-Function: Stability Butress Vol. WC. Function: Stability Butress K-Ratio: 1 K-Direction: 0 °

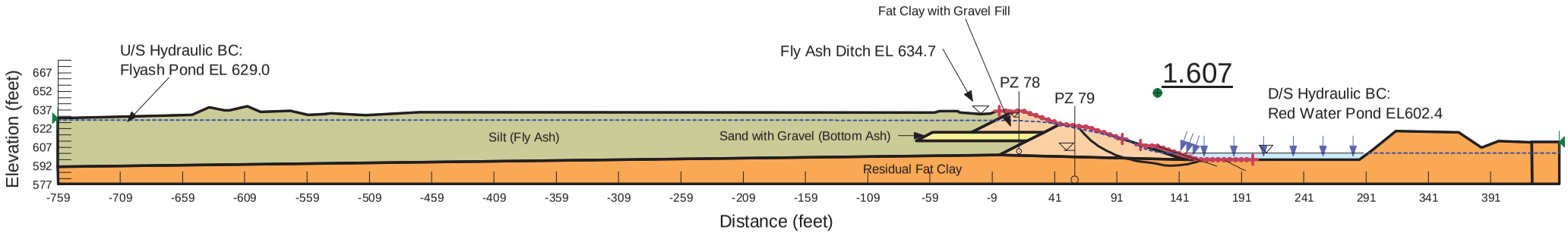


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Widows Creek Ash Pond
Section L - Main Ash Pond
Slope Stability; After IRRP Construction
Exit & Entry Optimized

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Mohr-Coulomb Unit Weight: 119 pcf Cohesion: 0 psf Phi: 33 ° Phi-B: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 150 psf Phi: 29 ° Phi-B: 0 °
Name: Residual Fat Clay Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 50 psf Phi: 33 ° Phi-B: 0 °
Name: Silt (Fly Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 27.5 ° Phi-B: 0 °
Name: Stability Butress Model: Mohr-Coulomb Unit Weight: 115 pcf Cohesion: 0 psf Phi: 40 ° Phi-B: 0 °

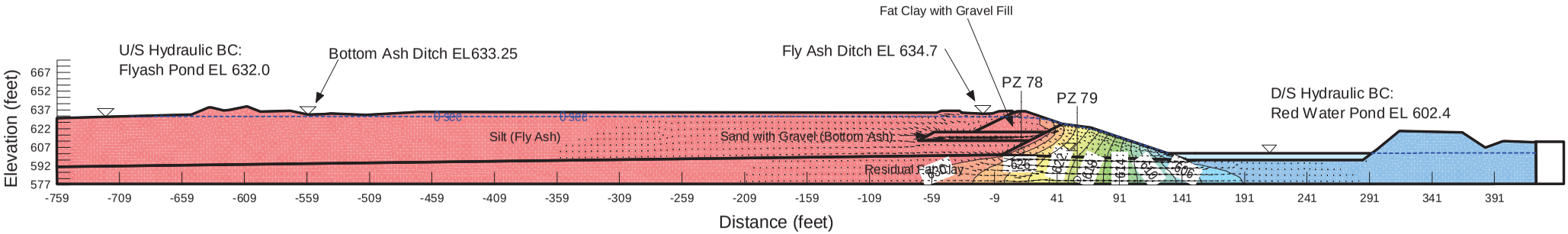


US EPA ARCHIVE DOCUMENT

Widows Creek Ash Pond
Section L - Main Ash Pond
Seepage Analysis; Existing Conditions

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Saturated / Unsaturated K-Function: Sand with Gravel (Bottom Ash) Vol. WC. Function: Sand with Gravel (Bottom Ash) K-Ratio: 0.1 K-Direction: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Saturated / Unsaturated K-Function: Fat Clay with Gravel (Embankment Fill) Vol. WC. Function: Fat Clay with Gravel (Embankment Fill) K-Ratio: 0.25 K-Direction: 0 °
Name: Residual Fat Clay Model: Saturated / Unsaturated K-Function: Residual Fat Clay Vol. WC. Function: Residual Fat Clay K-Ratio: 0.1 K-Direction: 0 °
Name: Silt (Fly Ash) Model: Saturated / Unsaturated K-Function: Silt (Fly Ash) Vol. WC. Function: Silt (Fly Ash) K-Ratio: 0.1 K-Direction: 0 °

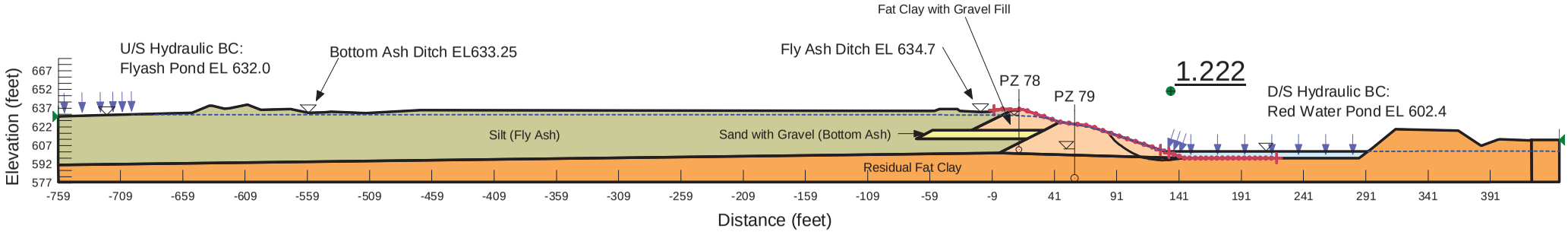


US EPA ARCHIVE DOCUMENT

Widows Creek Ash Pond
Section L - Main Ash Pond
Slope Stability; Existing Conditions
Exit & Entry Optimized

This analysis is based on electronic GeoStudio files provided to URS by the Tennessee Valley Authority. The electronic files were originally produced by STANTEC. The models were modified by URS with updated material properties and geometry based on additional information and detailed design concepts.

Name: Sand with Gravel (Bottom Ash) Model: Mohr-Coulomb Unit Weight: 119 pcf Cohesion: 0 psf Phi: 33 ° Phi-B: 0 °
Name: Fill Clay with Gravel (Embankment Fill) Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 150 psf Phi: 29 ° Phi-B: 0 °
Name: Residual Fat Clay Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 50 psf Phi: 33 ° Phi-B: 0 °
Name: Silt (Fly Ash) Model: Mohr-Coulomb Unit Weight: 112 pcf Cohesion: 0 psf Phi: 27.5 ° Phi-B: 0 °



US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 16

Stantec Consulting Services Inc. Correspondence Dated February 15, 2012, Re: Results of Pseudostatic Slope Stability Analysis, Active CCP Disposal Facilities, BRF, COF, GAF, JSF, KIF, PAF, and WCF

Stantec Consulting Services Inc.
10509 Timberwood Circle Suite 100
Louisville, KY 40223-5301
Tel: (502) 212-5000
Fax: (502) 212-5055

February 15, 2012 ltr_002_175551015

Mr. Michael S. Turnbow
Tennessee Valley Authority
1101 Market Street, LP 2G-C
Chattanooga, Tennessee 37402-2801

Re: Results of Pseudostatic Slope Stability Analysis
Active CCP Disposal Facilities
BRF, COF, GAF, JSF, JOF, KIF, PAF, and WCF

Dear Mr. Turnbow:

As requested, Stantec Consulting Services Inc. (Stantec) has conducted pseudostatic slope stability analyses for ground motion levels corresponding to a return period of 2,500 years to support the U.S. Environmental Protection Agency's assessment of TVA's CCP disposal facilities. The results for Bull Run (BFR), Colbert (COF), Gallatin (GAF), John Sevier (JSF), Johnsonville (JOF), Kingston (KIF), Paradise (PAF), and Widows Creek (WCF) are provided in this letter.

Approach

The analyses were performed for current conditions using pseudostatic stability methods, where the added inertial load from an earthquake is assumed to be represented by a simple horizontal pseudostatic coefficient. Specifics related to the analyses/approach are as follows:

- Subsurface data was obtained from the Stantec's recent geotechnical studies performed in 2009 and 2010 time frame.
- SLOPE/W software (from GEO-SLOPE International, Inc.) was used to perform the calculations.
- One existing SLOPE/W cross-section model per disposal facility was selected from the previous studies for analysis. For simplicity and conservatism, the selected sections represent the facility's lowest current static (long-term) factor of safety. The SLOPE/W models were updated to reflect any significant mitigations or operational changes that have occurred since completion of Stantec's geotechnical studies.
- Undrained shear strength parameters were used.
- Ground motion levels corresponding to a return period of 2,500 years (or approximate exceedance probability of 2% in 50 years) was used for selection of a horizontal seismic coefficient. For simplicity, the horizontal seismic coefficient was selected to equal the total hazard peak ground acceleration (rock) for 2,500 year return periods as shown in plant-

US EPA ARCHIVE DOCUMENT

specific tables (Tables 13 through 23) of TVA's March 28, 2011 region-specific seismic hazard study performed by AMEC Geomatrix, Inc.

- A target factor of safety (FS) of 1.0 was considered for comparing results.

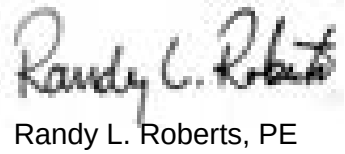
Results

The results of the pseudostatic stability analyses are enclosed (summary spreadsheet, SLOPE/W cross-sections, and plan views showing cross-section locations). The results indicate factors of safety greater than or equal to the target of 1.0.

Stantec appreciates the opportunity to provide these services. If you have questions, or if we can provide additional information, please let us know.

Sincerely,

STANTEC CONSULTING SERVICES INC.



Randy L. Roberts, PE
Principal

Enclosures

/cdm

US EPA ARCHIVE DOCUMENT

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Pseudostatic Stability Analysis Summary - TVA Active CCP Disposal Facilities

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

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

US EPA ARCHIVE DOCUMENT

Widows Creek Fossil Plant (WCF)

US EPA ARCHIVE DOCUMENT

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

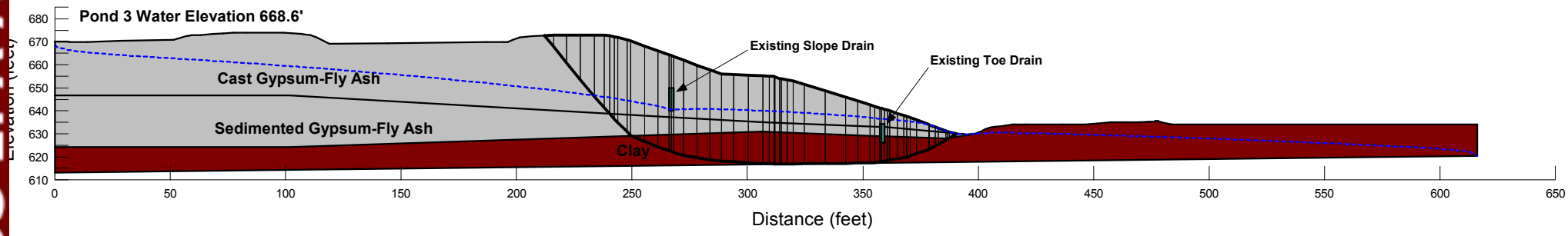
Section F - Gypsum Stack
Widows Creek Fossil Plant
Stevenson, Alabama

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Material Type	Unit Weight	Cohesion	Friction Angle
Cast Gypsum-Fly Ash	113 pcf	0 psf	40 °
Sedimented Gypsum-Fly Ash	112 pcf	0 psf	41 °
Sand Drains	110 pcf	0 psf	33 °
Clay	123 pcf	650 psf	15.7 °

Factor of Safety: 1.5

Horizontal Sesmic Coefficient Kh = 0.1 g
2500 year Return Period Event



te of Assessment - 11/4/2011

Project No. 175551015

US EPA ARCHIVE DOCUMENT

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

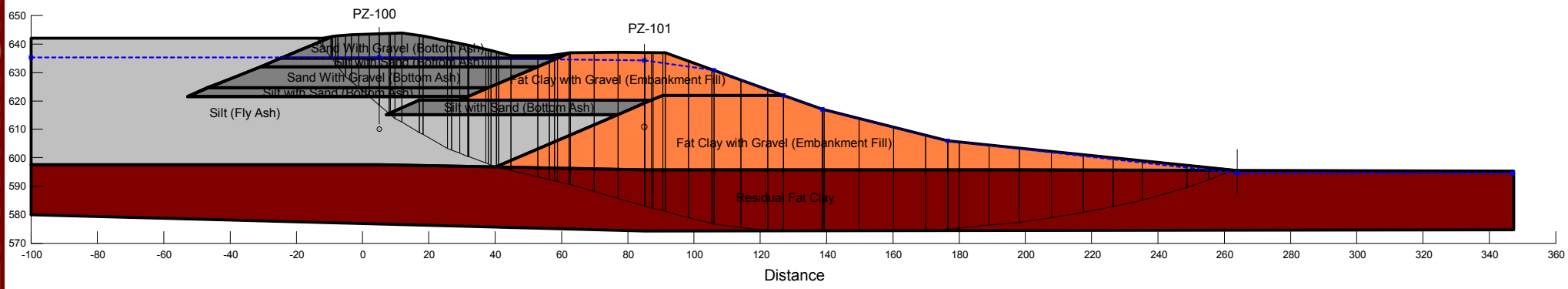
Section D - Dredge Cell (Old Scrubber Sludge Pond)
Widows Creek Fossil Plant
Stevenson, Alabama

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Material Type	Unit Weight	Cohesion	Friction Angle
Residual Fat Clay	125 pcf	650 psf	15.7 °
Silt with Sand (Bottom Ash)	112 pcf	0 psf	33 °
Silt (Fly Ash)	112 pcf	0 psf	21.8 °
Sand With Gravel (Bottom Ash)	119 pcf	0 psf	33 °
Fat Clay with Gravel (Embankment Fill)	125 pcf	1375 psf	14.2 °

Factor of Safety: 1.1

Horizontal Sesmic Coefficient Kh = 0.1 g
2500 year Return Period Event



Date of Assessment - 11/04/2011

Project No. 175551015

US EPA ARCHIVE DOCUMENT

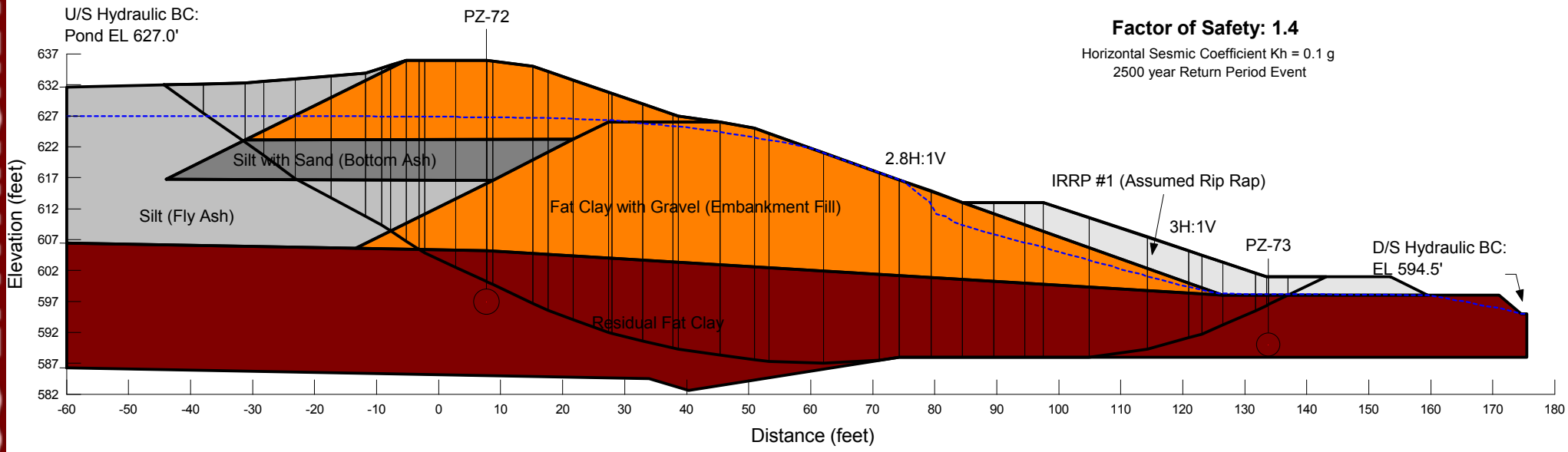
Pseudostatic Slope Stability Analysis
CCP Storage Facilities - Existing Conditions
Tennessee Valley Authority Fossil Plants

Section J - Main Ash Pond
Widows Creek Fossil Plant
Stevenson, Alabama

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Additional remediation measures taken from URS plans dated 8/14/2010.

Material Type	Unit Weight	Cohesion	Friction Angle
Sand with Gravel (Bottom Ash)	119 pcf	0 psf	33 °
Rip Rap	115 pcf	0 psf	40 °
Silt (Fly Ash)	112 pcf	0 psf	21.8 °
Residual Fat Clay	125 pcf	650 psf	15.7 °
Fat Clay with Gravel (Embankment Fill)	125 pcf	1375 psf	14.2 °



Assessment - 11/4/2011

Project No. 175551015

BORING LOCATION TABLE			
Boring	Northing	Easting	Elev. (ft.)
***STN-28	1,600,650.1	527,741.0	651.31
***STN-29	1,600,590.1	527,790.4	623.86
***STN-31	1,601,984.7	527,265.3	672.62
***STN-32	1,602,037.1	528,528.5	656.24
***STN-33	1,602,037.0	528,597.9	640.20
***STN-34	1,603,809.7	528,724.1	674.11
***STN-35	1,603,831.0	528,927.7	656.13
***STN-36	1,603,833.8	529,007.1	631.94
***STN-37	1,603,319.9	527,464.9	626.95
***STN-38	1,602,372.3	526,925.2	675.50
***STN-39	1,602,374.5	526,839.7	655.19
***STN-40	1,602,379.2	526,750.0	621.38
***STN-41	1,601,812.5	527,159.8	657.76
***STN-42	1,601,813.4	527,067.0	659.04
***STN-43	1,601,979.2	527,240.0	672.69
***STN-44	1,601,640.7	527,161.8	655.78
***STN-45	1,601,630.6	527,063.0	655.20
***STN-46	1,601,239.7	527,133.1	654.70
***STN-47	1,601,297.4	527,118.9	655.53
***STN-48	1,601,416.0	527,060.8	654.92
***STN-49	1,601,258.4	527,058.8	655.03
***STN-50	1,601,244.1	527,059.0	654.94
***STN-51	1,601,651.7	526,940.3	621.13
***STN-52	1,601,757.5	526,939.5	621.65
***CPT-10	1,601,250.6	526,991.4	629.0
***CPT-11	1,601,253.2	527,098.3	655.0
***CPT-12	1,601,629.8	526,995.7	629.0
***CPT-13	1,601,650.8	527,105.0	655.0
***V-9	1,601,630.2	527,103.0	655.0
***V-10	1,601,235.3	527,092.3	655.0
*TH-1	1,600,610.0	527,279.3	643.43
*TH-2	1,600,584.4	527,243.0	650.48
*TH-3	1,601,322.1	527,292.9	643.55
*TH-4	1,601,303.2	527,810.5	639.89
*TH-5	1,601,646.1	528,409.3	642.09
*TH-6	1,600,568.4	527,615.1	648.88
*TH-7	1,601,553.5	528,589.9	638.64
**SPT-1	1,601,472.4	527,068.5	649.90
**SPT-2	1,602,279.9	527,011.4	661.89
**SPT-3	1,603,928.6	528,728.1	659.79
**SPT-4	1,602,691.0	528,555.7	663.34
**SPT-5	1,603,183.6	527,640.9	660.18
**SPT-6	1,601,092.3	527,910.1	660.50

* Boring drilled in 1991.
** Boring drilled in 2004.
*** Boring drilled in 2009.

NOTES:

1. Topographic mapping was developed from a composite of a LIDAR survey which was originally performed in NAD 83 and later shifted to NAD 27 Datum by Tuck Mapping Solutions, Inc. on January 9, 2009 (post-event) and surveys performed by TVA on October 30, 2006, January 14, 2009 and February 23, 2009.
2. The geotechnical information and data furnished herein are not intended as representation or warranties but are furnished for information only. It shall be distinctly understood that the Owner or Engineer will not be responsible for any deduction, interpretation or conclusion drawn therefrom. The information is made available in order that the Contractor may have ready access to the same information available to the Owner and the Engineer and is not part of this contract.
3. See sheet 2 for baseline curve data and table of baseline coordinates.

RECORD DRAWING

FOR INFORMATION ONLY
This Record Drawing which has been previously submitted to TVA is provided for Information Only.

Widows Creek Fossil Plant, Gypsum Stack. Cross Section F used to perform pseudostatic slope stability analysis.

STILLING POND
POND 3
POND 2B
POND 2A
POND 1
Project Baseline
GRAPHIC SCALE: 1" = 100'
CONTOUR INTERVAL = 2'

LEGEND

- STN-47 Soil Boring with Undisturbed (Shelby) Tube Samples and Rock Core
- STN-52 Rock Sounding
- STN-34 Soil Boring with Standard Penetration Tests and/or Undisturbed (Shelby) Tube Samples
- CPT-10 Cone Penetration Test with Piezometer
- V-10 Vane Shear Test Location
- *TH-7 Existing Test Boring, 1991
- **SPT-3 Existing Test Boring, 2004

For Supporting Design Calculations see FPGWCFFESCDX00000020100005

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST
R 0	02/05/10	GKA	PJB	RDF	HRA	JSM	JSM	TJ		
ISSUED FOR REVIEW										

SCALE: 1"=100' EXCEPT AS NOTED

YARD
–

GYPSUM STACK GEOTECHNICAL EXPLORATION BORING LAYOUT

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
K. ANDERSON	P. BOND	R. FULLER	H. APARICIO	J. MONTGOMERY	J. MONTGOMERY	T. JOHNSON

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

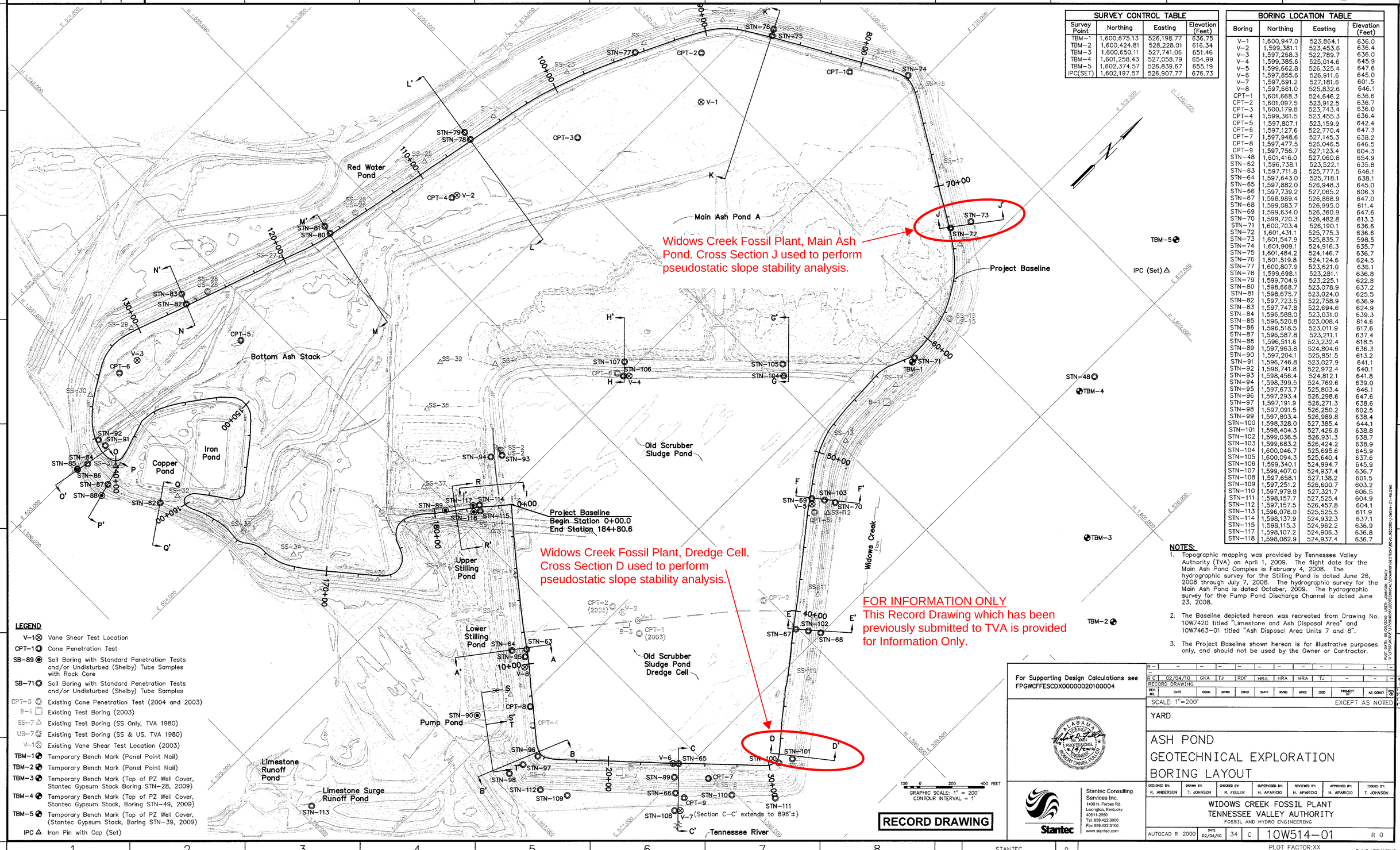
Stantec Consulting Services Inc.
1409 N. Forbes Rd.
Lexington, Kentucky
40511-2050
Tel. 859.422.3000
Fax 859.422.3100
www.stantec.com

AUTOCAD R 2000 | DATE 02/05/10 | 34 | C | 10W515-04 | R 0

STANTEC | 0
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:100
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY



SURVEY CONTROL TABLE

Survey Point	Northing	Easting	Elevation (Feet)
TBM–1	1,600,675.13	526,198.77	636.75
TBM–2	1,600,424.81	528,228.01	616.34
TBM–3	1,600,650.11	527,741.06	651.46
TBM–4	1,601,258.43	527,058.79	654.99
TBM–5	1,602,374.57	526,839.67	655.19
IPC(SET)	1,602,197.57	526,907.77	676.73

BORING LOCATION TABLE

Boring	Northing	Easting	Elevation (Feet)
V–1	1,600,947.0	523,864.1	636.0
V–2	1,599,381.1	523,453.6	636.4
V–3	1,597,268.3	522,789.7	636.0
V–4	1,599,385.6	525,014.6	645.9
V–5	1,599,662.8	526,325.4	647.6
V–6	1,597,855.6	526,911.6	645.0
V–7	1,597,691.2	527,181.6	601.5
V–8	1,597,661.0	525,832.6	646.1
CPT–1	1,601,668.3	524,646.2	636.6
CPT–2	1,601,097.5	523,912.5	636.7
CPT–3	1,600,179.8	523,743.4	636.0
CPT–4	1,599,361.5	523,455.3	636.4
CPT–5	1,597,807.1	523,159.9	642.4
CPT–6	1,597,127.6	522,770.4	647.3
CPT–7	1,597,948.6	527,145.3	638.2
CPT–8	1,597,477.5	526,046.5	646.5
CPT–9	1,597,756.7	527,123.4	604.3
STN–48	1,601,416.0	527,060.8	654.9
STN–62	1,596,738.1	523,522.1	635.8
STN–63	1,597,711.8	525,777.5	646.1
STN–64	1,597,643.0	525,718.1	638.1
STN–65	1,597,882.0	526,948.3	645.0
STN–66	1,597,739.2	527,065.2	606.3
STN–67	1,598,989.4	526,868.9	647.0
STN–68	1,599,083.7	526,995.0	611.4
STN–69	1,599,634.0	526,360.9	647.6
STN–70	1,599,720.3	526,482.8	613.3
STN–71	1,600,703.4	526,190.1	636.6
STN–72	1,601,431.1	525,775.3	636.6
STN–73	1,601,547.9	525,835.7	598.5
STN–74	1,601,909.1	524,916.3	635.7
STN–75	1,601,484.2	524,146.7	636.7
STN–76	1,601,519.8	524,124.6	624.5
STN–77	1,600,807.9	523,621.0	636.1
STN–78	1,599,698.1	523,281.1	636.8
STN–79	1,599,704.9	523,225.1	622.8
STN–80	1,598,668.7	523,078.9	637.2
STN–81	1,598,675.7	523,024.0	625.5
STN–82	1,597,723.5	522,758.9	636.9
STN–83	1,597,747.8	522,694.6	624.9
STN–84	1,596,588.0	523,031.0	639.3
STN–85	1,596,520.8	523,008.4	614.6
STN–86	1,596,516.5	523,011.9	617.6
STN–87	1,596,587.8	523,211.1	637.4
STN–88	1,596,511.6	523,232.4	618.5
STN–89	1,597,963.8	524,804.6	636.3
STN–90	1,597,204.1	525,851.5	613.2
STN–91	1,596,746.8	523,027.9	641.1
STN–92	1,596,741.8	522,972.4	640.1
STN–93	1,598,456.4	524,812.1	641.8
STN–94	1,598,399.5	524,769.6	639.0
STN–95	1,597,673.7	525,803.4	646.1
STN–96	1,597,293.4	526,298.6	647.6
STN–97	1,597,191.9	526,271.3	638.6
STN–98	1,597,091.5	526,250.2	602.5
STN–99	1,597,803.4	526,989.8	638.4
STN–100	1,598,328.0	527,385.4	644.1
STN–101	1,598,404.3	527,426.8	638.8
STN–102	1,599,036.5	526,931.3	638.7
STN–103	1,599,683.2	526,424.2	638.9
STN–104	1,600,046.7	525,695.6	645.9
STN–105	1,600,094.3	525,640.4	637.6
STN–106	1,599,340.1	524,994.7	645.9
STN–107	1,599,407.0	524,937.4	636.7
STN–108	1,597,658.1	527,138.2	601.5
STN–109	1,597,251.2	526,600.7	603.2
STN–110	1,597,979.8	527,321.7	606.5
STN–111	1,598,157.7	527,525.4	604.9
STN–112	1,597,157.5	526,457.8	604.1
STN–113	1,596,076.0	525,525.5	611.9
STN–114	1,598,137.9	524,932.3	637.1
STN–115	1,598,115.3	524,962.2	636.9
STN–117	1,598,107.2	524,906.3	636.8
STN–118	1,598,082.9	524,937.4	636.7

NOTES:

1. Topographic mapping was provided by Tennessee Valley Authority (TVA) on April 1, 2009. The flight date for the Main Ash Pond Complex is February 4, 2008. The hydrographic survey for the Stilling Pond is dated June 26, 2008 through July 7, 2008. The hydrographic survey for the Main Ash Pond is dated October, 2009. The hydrographic survey for the Pump Pond Discharge Channel is dated June 23, 2008.
2. The Baseline depicted hereon was recreated from Drawing No. 10W7420 titled "Limestone and Ash Disposal Area" and 10W7463–01 titled "Ash Disposal Area Units 7 and 8".
3. The Project Baseline shown hereon is for illustrative purposes only, and should not be used by the Owner or Contractor.

LEGEND

- V–1⊗ Vane Shear Test Location
- CPT–1 Cone Penetration Test
- SB–89 Soil Boring with Standard Penetration Tests and/or Undisturbed (Shelby) Tube Samples with Rock Core
- SB–71 Soil Boring with Standard Penetration Tests and/or Undisturbed (Shelby) Tube Samples
- CPT–3 Existing Cone Penetration Test (2004 and 2003)
- B–1 Existing Test Boring (2003)
- SS–7 Existing Test Boring (SS Only, TVA 1980)
- US–7 Existing Test Boring (SS & US, TVA 1980)
- V–1⊗ Existing Vane Shear Test Location (2003)
- TBM–1 Temporary Bench Mark (Panel Point Nail)
- TBM–2 Temporary Bench Mark (Panel Point Nail)
- TBM–3 Temporary Bench Mark (Top of PZ Well Cover, Stantec Gypsum Stack Boring STN–28, 2009)
- TBM–4 Temporary Bench Mark (Top of PZ Well Cover, Stantec Gypsum Stack, Boring STN–49, 2009)
- TBM–5 Temporary Bench Mark (Top of PZ Well Cover, (Stantec Gypsum Stack, Boring STN–39, 2009)
- IPC △ Iron Pin with Cap (Set)

FOR INFORMATION ONLY
This Record Drawing which has been previously submitted to TVA is provided for Information Only.

For Supporting Design Calculations see FPGWCFFESCDX00000020100004

RECORD DRAWING

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV. NO.
R 0	02/04/10	GKA	TJ	RDF	HRA	HRA	HRA	TJ	–	–	R 0

RECORD DRAWING

SCALE: 1"=200' EXCEPT AS NOTED

YARD

ASH POND GEOTECHNICAL EXPLORATION BORING LAYOUT

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
K. ANDERSON	T. JOHNSON	R. FULLER	H. APARICIO	H. APARICIO	H. APARICIO	T. JOHNSON

WIDOWS CREEK FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000 | DATE 02/04/10 | 34 | C | 10W514–01 | R 0

Stantec Consulting Services Inc.
1409 N. Forbes Rd.
Lexington, Kentucky
40511-2050
Tel. 859.422.3000
Fax 859.422.3100
www.stantec.com

STANTEC | 0
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:XX
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

PLOT DATE: 05/04/2010 USER: JOHNSON, TRACY
V:\1750\active\172566808\gdb\geotechnical\drawings\gdb\geotechdwg_sec09d\10w514-01-rd.dwg

US EPA ARCHIVE DOCUMENT

APPENDIX B

Document 17

Main Ash Pond – Dam Inspection Check List Form

Site Name:	**Widows Creek Fossil Plant**	Date:	**13 September 2011**
Unit Name:	**Main Ash Pond**	Operator's Name:	**Tennessee Valley Authority**
Unit I.D.:	**Main Ash Pond**	Hazard Potential Classification:	**High** ☐ **Significant** ☒ **Low** ☐
	Inspector's Name:	Joseph P. Klein, III, P.E. and Frank B. Lockridge, P.E.	

Check the appropriate box below. Provide comments when appropriate. If not applicable or not available, record "N/A". Any unusual conditions or construction practices that should be noted in the comments section. For large diked embankments, separate checklists may be used for different embankment areas. If separate forms are used, identify approximate area that the form applies to in comments.

	Yes	No		Yes	No
1. Frequency of Company's Dam Inspections?	X		18. Sloughing or bulging on slopes?		X
2. Pool elevation (operator records)?	627.5		19. Major erosion or slope deterioration?		X
3. Decant inlet elevation (operator records)?	N/A		20. Decant Pipes:		
4. Open channel spillway elevation (operator records)?	N/A		Is water entering inlet, but not exiting outlet?		X
5. Lowest dam crest elevation (operator records)?	635		Is water exiting outlet, but not entering inlet?		X
6. If instrumentation is present, are readings recorded (operator records)?	X		Is water exiting outlet flowing clear?	X	
7. Is the embankment currently under construction?	X		21. Seepage (specify location, if seepage carries fines, and approximate seepage rate below):		
8. Foundation preparation (remove vegetation, stumps, topsoil in area where embankment fill will be placed)?	Documentation Pending		From underdrain?		X
9. Trees growing on embankment? (If so, indicate largest diameter below)	X		At isolated points on embankment slopes?	X	
10. Cracks or scarps on crest?		X	At natural hillside in the embankment area?		X
11. Is there significant settlement along the crest?		X	Over widespread areas?	X	
12. Are decant trashracks clear and in place?	X		From downstream foundation area?		X
13. Depressions or sinkholes in tailings surface or whirlpool in the pool area?		X	"Boils" beneath stream or ponded water?		X
14. Clogged spillways, groin or diversion ditches?		X	Around the outside of the decant pipe?		X
15. Are spillway or ditch linings deteriorated?		X	22. Surface movements in valley bottom or on hillside?		X
16. Are outlets of decant or underdrains blocked?		X	23. Water against downstream toe?	X	
17. Cracks or scarps on slopes?		X	24. Were Photos taken during the dam inspection?	X	

Major adverse changes in these items could cause instability and should be reported for further evaluation. Adverse conditions noted in these items should normally be described (extent, location, volume, etc.) in the space below and on the back of this sheet.

Issue #	Comments
1	Next TVA quarterly inspection scheduled for 9/22/2100. Recent prior inspection reports provided for review.
7	Inverted filter drain and lower slope buttress being constructed on northeast embankment
8	Documentation of original site preparation being researched by TVA. Geotechnical data indicates Phase 2 embankment constructed by widening original embankment to the interior of the impoundment, resulting in a portion of the base of the new dike supported on CCR.
9	Trees present on lower embankment of west dike between Main Ash Pond and Red Water Pond
21	Inverted filter drains installed and multiple points in the embankments. Other embankment seepage areas have been identified, marked with signs and are being monitored pending construction of additional filter drains.
23	Widows Creek flows along east dike

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Coal Combustion Waste (CCW)

Impoundment Inspection

Impoundment NPDES Permit	AL0003875	**INSPECTOR**	Joseph P. Klein, III, P.E. and Frank B. Lockridge, P.E.

Date	Permit Modification Effective Date January 7, 2008
Impoundment Name	DNS001a: Ash pond discharge to units 7 and 8 intake structures DSN001b and DSN001c: Ash pond consisting of low volume wastes from various sources DSN005: Constructed wetlands receiving stormwater runoff from ash disposal area and other sources DSN006 Constructed wetland discharge consisting of inactive ash disposal seepage and site storm runoff Discharge from other non-ash related sources.
Impoundment Company	Tennessee Valley Authority
EPA Region	4
State Agency (Field Office) Address	Alabama Department of Environmental Management 1400 Coliseum Blvd. Montgomery, AL 36130
Name of Impoundment	Main Ash Pond – Widows Creek Fossil Plant

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

New ☐ **Update** ☒

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

IMPOUNDMENT FUNCTION: Storage of coal combustion residual material

Nearest Downstream Town Name: Stevenson, AL

Distance from the impoundment: 4 miles

Location:

Latitude	34	Degrees	53	Minutes	32.71	Seconds	**N**
Longitude	85	Degrees	45	Minutes	01.39	Seconds	**W**

State Alabama **County** Jackson

	Yes	No

Does a state agency regulate this impoundment?	☒	☐
If So Which State Agency?	Alabama Department of Environmental Management	

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HAZARD POTENTIAL *(In the event the impoundment should fail, the following would occur):*

☐ **LESS THAN LOW HAZARD POTENTIAL:** Failure or misoperation of the dam results in no probable loss of human life or economic or environmental losses.

☐ **LOW HAZARD POTENTIAL:** Dams assigned the low hazard potential classification are those where failure or misoperation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.

☒ **SIGNIFICANT HAZARD POTENTIAL:** Dams assigned the significant hazard potential classification are those dams where failure or misoperation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns. Significant hazard potential classification dams are often located in predominantly rural or agricultural areas but could be located in areas with population and significant infrastructure.

☐ **HIGH HAZARD POTENTIAL:** Dams assigned the high hazard potential classification are those where failure or misoperation will probably cause loss of human life.

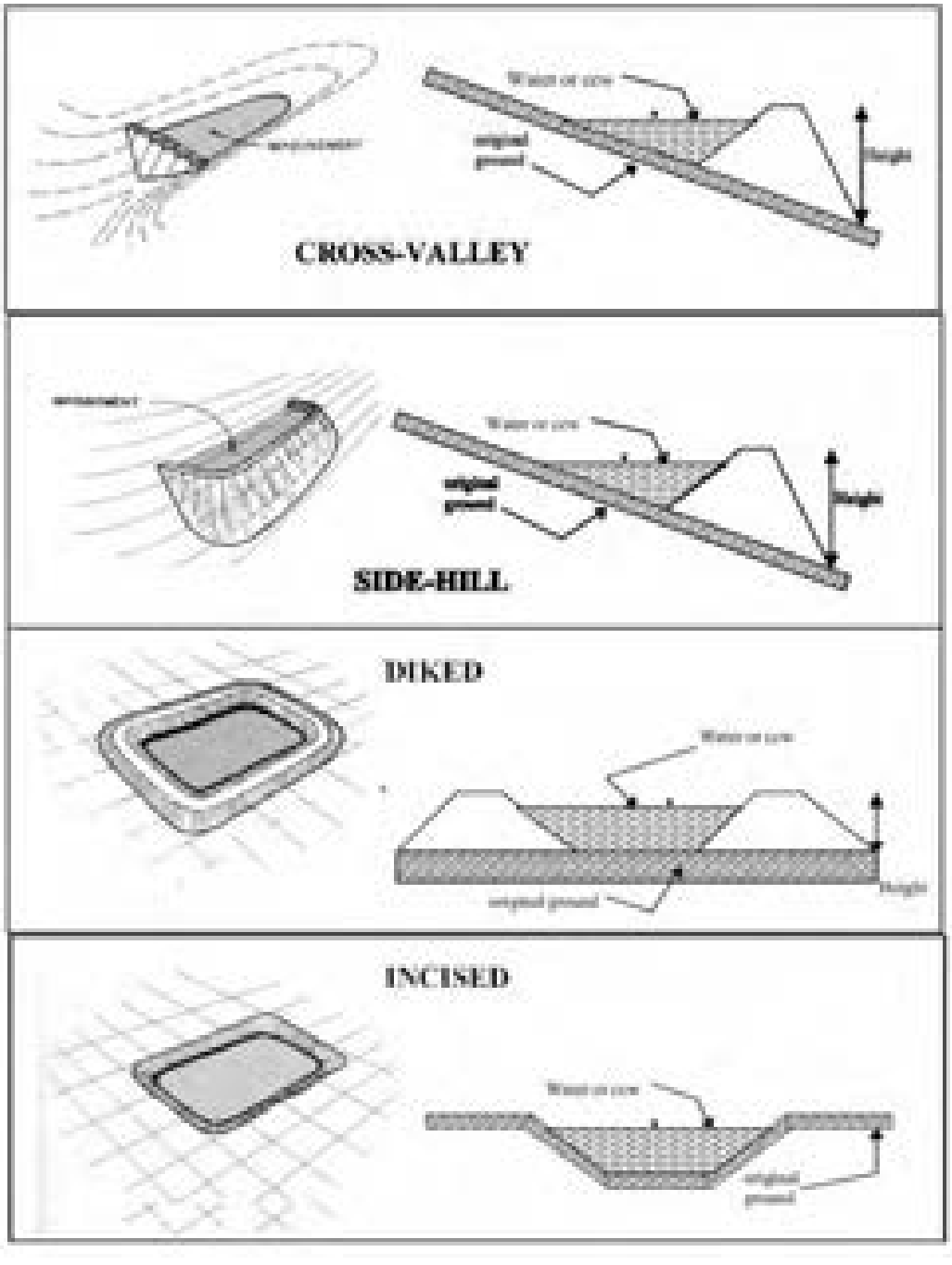
DESCRIBE REASONING FOR HAZARD RATING CHOSEN:

Based on the absence of housing or established public gathering places near the Widows Creek Plant indicates that loss of human life is not probable in the event of a failure or misoperation of the Widows Creek Main Ash Impoundment. However, a failure or misoperation is expected to cause economic loss and damage to the environment.

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CONFIGURATION:



☐ Cross-Valley ☐ Side-Hill ☒ Diked

☐ Incised (form completion optional) ☐ Combination Incised/Diked

Embankment Height (ft)	30	**Embankment Material**	Clay
Pool Area (ac)	304	**Liner**	None
Current Freeboard (ft)	8	**Liner Permeability**	N/A

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TYPE OF OUTLET (Mark all that apply)

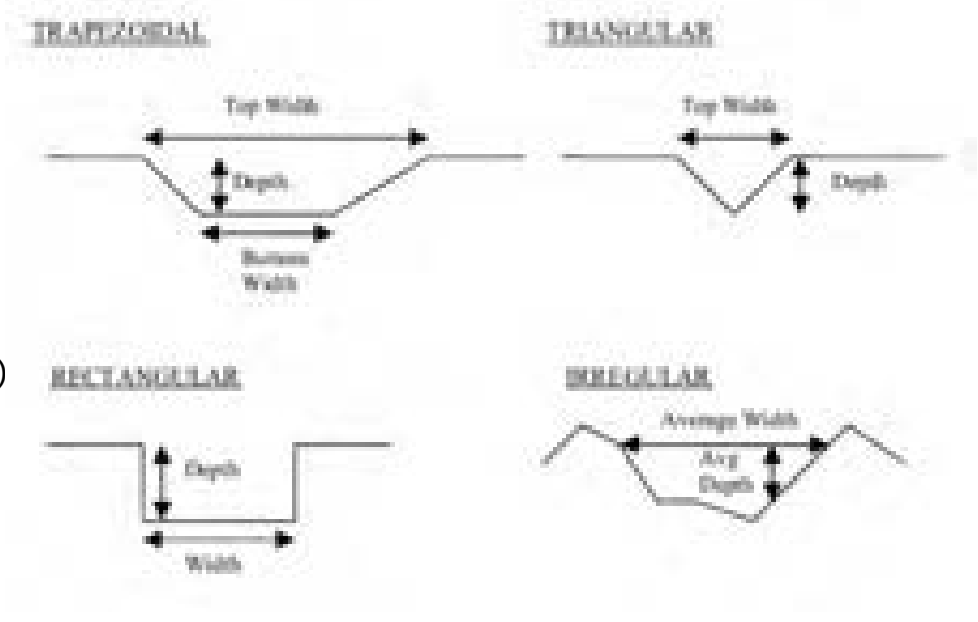
☐ **Open Channel Spillway**

- ☐ Trapezoidal
- ☐ Triangular
- ☐ Rectangular
- ☐ Irregular

depth (ft)

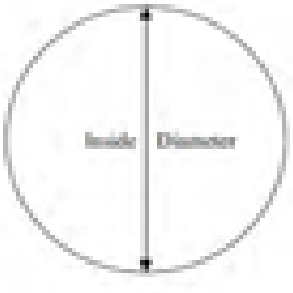
average bottom width (ft)

top width (ft)



☒ **Outlet (Three)**

36” inside diameter
(SDR 17 – smooth lined – 19.5” OD)



Material

- ☒ corrugated metal
- ☐ welded steel
- ☐ concrete
- ☐ plastic (hdpe, pvc, etc.)
- ☐ other (specify):

	Yes	No
Is water flowing through the outlet?	☒	☐

☐ **No Outlet**

☐ **Other Type of Outlet**
(specify):

The Impoundment was Designed By Documentation Pending

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

If So When?

If So Please Describe :

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

If So When? 2009 and continuing

If So Please Describe : Several inverted filter drains have been installed along the Main Ash Pond embankment. Additional seepage areas have been identified, marked in the field, and construction plans are being developed for drains in those areas as well.

	Yes	No
Has there ever been any measures undertaken to monitor/lower Phreatic water table levels based on past seepages or breaches at this site?	☒	☐

If so, which method (e.g., piezometers, gw pumping,...)? Piezometers

If So Please Describe : About 39 piezometers have been installed to monitor phreatic water levels in the Main Ash Pond Embankment. The piezometers have been installed along 16 embankment transects spaced along the perimeter of the embankment.

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ADDITIONAL INSPECTION QUESTIONS

Concerning the embankment foundation, was the embankment construction built over wet ash, slag, or other unsuitable materials? If there is no information just note that.

Discussion during the site visit indicated that TVA was reviewing their files for documentation on site preparation for the initial embankment foundation sub-grade.

The embankment height was subsequently raised be extending the exterior slope upward toward the interior of the impoundment. The 2009 geotechnical exploration data indicate that widening of the embankment toward the interior of the impoundment resulted in a portion of the new embankment being supported on coal combustion residue material.

Did the dam assessor meet with, or have documentation from, the design Engineer-of-Record concerning the foundation preparation?

No

From the site visit or from photographic documentation, was there evidence of prior releases, failures, or patchwork on the dikes?

No evidence of prior releases or failures if the dike. The site visit identified several inverted filter seepage drains on the exterior slope of the Main Ash Pond embankment.

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APPENDIX B

Document 18

Gypsum Stack – Dam Inspection Check List Form



Site Name:	**Widows Creek Fossil Plant**	Date:	**13 September 2011**
Unit Name:	**Gypsum Stack**	Operator's Name:	**Tennessee Valley Authority**
Unit I.D.:	**Gypsum Stack**	Hazard Potential Classification:	**High** ☐ **Significant** ☒ **Low** ☐
	Inspector's Name:	Joseph P. Klein, III, P.E. and Frank B. Lockridge, P.E.	

Check the appropriate box below. Provide comments when appropriate. If not applicable or not available, record "N/A". Any unusual conditions or construction practices that should be noted in the comments section. For large diked embankments, separate checklists may be used for different embankment areas. If separate forms are used, identify approximate area that the form applies to in comments.

	Yes	No		Yes	No
1. Frequency of Company's Dam Inspections?	X		18. Sloughing or bulging on slopes?		X
2. Pool elevation (operator records)?	614.2		19. Major erosion or slope deterioration?		X
3. Decant inlet elevation (operator records)?	N/A		20. Decant Pipes:		
4. Open channel spillway elevation (operator records)?	N/A		Is water entering inlet, but not exiting outlet?		X
5. Lowest dam crest elevation (operator records)?	624		Is water exiting outlet, but not entering inlet?		X
6. If instrumentation is present, are readings recorded (operator records)?	X		Is water exiting outlet flowing clear?	X	
7. Is the embankment currently under construction?		X	21. Seepage (specify location, if seepage carries fines, and approximate seepage rate below):		
8. Foundation preparation (remove vegetation, stumps, topsoil in area where embankment fill will be placed)?	Documentation Pending		From underdrain?		X
9. Trees growing on embankment? (If so, indicate largest diameter below)	X		At isolated points on embankment slopes?	X	
10. Cracks or scarps on crest?		X	At natural hillside in the embankment area?		X
11. Is there significant settlement along the crest?		X	Over widespread areas?	X	
12. Are decant trashracks clear and in place?		X	From downstream foundation area?		X
13. Depressions or sinkholes in tailings surface or whirlpool in the pool area?	N/A		"Boils" beneath stream or ponded water?		X
14. Clogged spillways, groin or diversion ditches?		X	Around the outside of the decant pipe?		X
15. Are spillway or ditch linings deteriorated?		X	22. Surface movements in valley bottom or on hillside?		X
16. Are outlets of decant or underdrains blocked?	N/A		23. Water against downstream toe?	X	
17. Cracks or scarps on slopes?		X	24. Were Photos taken during the dam inspection?	X	

Major adverse changes in these items could cause instability and should be reported for further evaluation. Adverse conditions noted in these items should normally be described (extent, location, volume, etc.) in the space below and on the back of this sheet.

Issue #	Comments
1	Next quarterly inspection scheduled for 9/22/2011. Recent prior inspection report provided for review.
7	Intermediate Risk Reduction Plan being implemented: Construction of inverted filter drains in seepage areas and stone buttresses being installed along bottom of impoundment dike. Riprap being installed in interior dike of Gypsum Stack Stilling Pond for wave protection.
12	Decant riser does not have a trash rack
21	Prior inspection identified several seepage areas. Corrective actions underway. See Issue 7.
23	Small drainage ditch along south end of east embankment.

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Coal Combustion Waste (CCW)

Impoundment Inspection

Impoundment NPDES Permit	AL0003875	**INSPECTOR**	Joseph P. Klein, III, P.E. and Frank B. Lockridge, P.E.

Date	Permit Modification Effective Date January 7, 2008
Impoundment Name	DNS001a: Ash pond discharge to units 7 and 8 intake structures DSN001b and DSN001c: Ash pond consisting of low volume wastes from various sources DSN005: Constructed wetlands receiving stormwater runoff from ash disposal area and other sources DSN006 Constructed wetland discharge consisting of inactive ash disposal seepage and site storm runoff Discharge from other non-ash related sources.
Impoundment Company	Tennessee Valley Authority
EPA Region	4
State Agency (Field Office) Address	Alabama Department of Environmental Management 1400 Coliseum Blvd. Montgomery, AL 36130
Name of Impoundment	Main Ash Pond – Widows Creek Fossil Plant

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

New ☐ **Update** ☒

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

IMPOUNDMENT FUNCTION:	Dry storage of gypsum and storage of stormwater runoff and seepage from gypsum stack.
Nearest Downstream Town Name:	Stevenson, AL
Distance from the impoundment:	4 miles

Location:

Latitude	34	Degrees	53	Minutes	59.21	Seconds	**N**
Longitude	85	Degrees	44	Minutes	36.06	Seconds	**W**

State Alabama **County** Jackson

Yes	No

Does a state agency regulate this impoundment?	☒	☐
If So Which State Agency?	Alabama Department of Environmental Management	

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HAZARD POTENTIAL *(In the event the impoundment should fail, the following would occur):*

☐ **LESS THAN LOW HAZARD POTENTIAL:** Failure or misoperation of the dam results in no probable loss of human life or economic or environmental losses.

☐ **LOW HAZARD POTENTIAL:** Dams assigned the low hazard potential classification are those where failure or misoperation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.

☒ **SIGNIFICANT HAZARD POTENTIAL:** Dams assigned the significant hazard potential classification are those dams where failure or misoperation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns. Significant hazard potential classification dams are often located in predominantly rural or agricultural areas but could be located in areas with population and significant infrastructure.

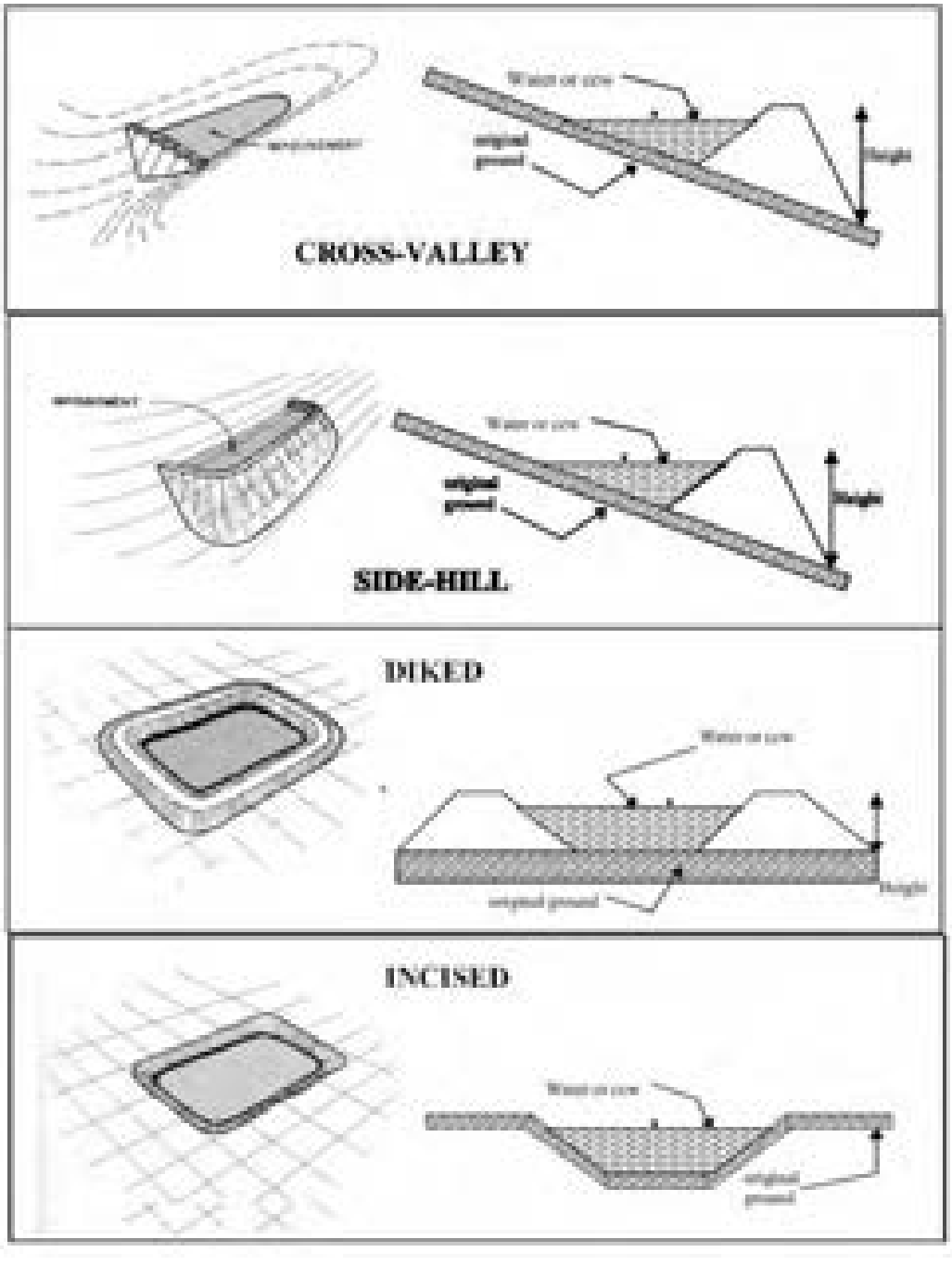
☐ **HIGH HAZARD POTENTIAL:** Dams assigned the high hazard potential classification are those where failure or misoperation will probably cause loss of human life.

DESCRIBE REASONING FOR HAZARD RATING CHOSEN:

Based on the absence of housing or established public gathering places near the Widows Creek Plant indicates that loss of human life is not probable in the event of a failure or misoperation of the Widows Creek Main Ash Impoundment. However, a failure or misoperation is expected to cause economic loss and damage to the environment.

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CONFIGURATION:



☐ Cross-Valley ☒ Side-Hill ☐ Diked

☐ Incised (form completion optional) ☐ Combination Incised/Diked

Embankment Height (ft)	35	**Embankment Material**	Clay
Pool Area (ac)	104	**Liner**	None
Current Freeboard (ft)	10 (Stilling Pond)	**Liner Permeability**	N/A

US EPA ARCHIVE DOCUMENT

TYPE OF OUTLET (Mark all that apply)

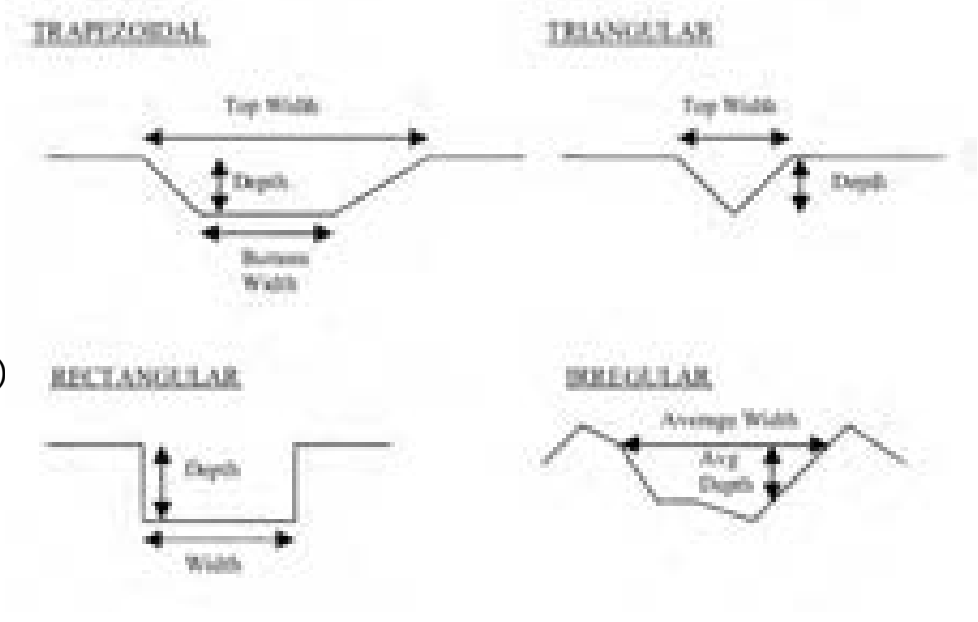
☐ **Open Channel Spillway**

- ☐ Trapezoidal
- ☐ Triangular
- ☐ Rectangular
- ☐ Irregular

depth (ft)

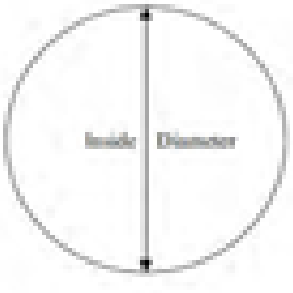
average bottom width (ft)

top width (ft)



☒ **Outlet**

36” inside diameter
(SDR 17 – smooth lined – 19.5” OD)



Material

- ☐ corrugated metal
- ☒ welded steel
- ☐ concrete
- ☐ plastic (hdpe, pvc, etc.)
- ☐ other (specify):

	Yes	No
Is water flowing through the outlet?	☒	☐

☐ **No Outlet**

☐ **Other Type of Outlet**
(specify):

The Impoundment was Designed By Documentation Pending

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

If So When? 2009

If So Please Describe : Failure of gypsum stack internal drainage system led to overtopping the east dike and spillage into Widows Creek.

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

If So When? 2009 and continuing

If So Please Describe : Several inverted filter drains have been installed along the gypsum stack and the impoundment embankment. Additional seepage areas have been identified, marked in the field, and construction plans are being developed for drains in those areas as well.

	Yes	No
Has there ever been any measures undertaken to monitor/lower Phreatic water table levels based on past seepages or breaches at this site?	☒	☐

If so, which method (e.g., piezometers, gw pumping,...)? Piezometers

If So Please Describe: About 20 piezometers have been installed to monitor phreatic water levels in the Main Ash Pond Embankment. The piezometers have been installed along 6 embankment transects spaced along the perimeter of the embankment.

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ADDITIONAL INSPECTION QUESTIONS

Concerning the embankment foundation, was the embankment construction built over wet ash, slag, or other unsuitable materials? If there is no information just note that.

Discussion during the site visit indicated that TVA was reviewing their files for documentation on site preparation for the initial embankment foundation sub-grade. Studies received after the site visit showed the subgrade was built over suitable materials.

Did the dam assessor meet with, or have documentation from, the design Engineer-of-Record concerning the foundation preparation?

No

From the site visit or from photographic documentation, was there evidence of prior releases, failures, or patchwork on the dikes?

A failure of the gypsum stack internal drainage system resulted in overtopping the impoundment embankment and a release into Widows Creek. The site visit identified several inverted filter seepage drains on the exterior slope of the gypsum stack and the impoundment embankment.

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APPENDIX B

Document 19

Dewberry Memorandum dated May 25, 2012, Regarding Qualitative Assessment

Memorandum

To: Stephen Hoffman, USEPA
Through: Jerry Strauss

From: Joe Klein

Date: May 25, 2012

Re: Qualitative Assessment
Liquefaction Potential
TVA Fossil Plant CCR Impoundments
Dewberry Project No, 50047151

This memorandum provides the results of a qualitative assessment of CCR impoundment embankment susceptibility to liquefaction at eight of the TVA fossil fuel plants assessed by Dewberry. The plants are: Bull Run; Colbert; Cumberland; Gallatin; John Sevier; Johnsonville; Kingston, and Widows Creek. We have not included Watts Bar (small pond, inactive for 30 years, minimal potential ash release), and Allen (TVA continuing deformation analyses, awaiting data and report)

TVA has indicated that a formal assessment of liquefaction susceptibility is underway; a completion date has not been provided. In prior rounds of the EPA CCR program, Dewberry has provided a preliminary indication of the presence of soils susceptible to liquefaction based on the geotechnical data provided with the slope stability analysis. The purpose of this assessment is to include similar information as a component of our reports to EPA, and to provide a uniform approach to the remaining plant sites.

Generally the geotechnical review looks at the soil stratification beneath both the embankments and impoundments to identify soil types considered susceptible to liquefaction; i.e., fine to medium grain sands, and some silts with Standard Penetration Resistance, or N-Values of less than 15 blows per foot[1]. That criterion, is an accepted industry standard for first level reviews.

Because several of the embankments had been constructed to their current configuration in stages, and because the raised sections were typically constructed by extending embankments in the *upstream* direction, most of TVA raised dikes are supported in part on stored bottom ash and/or fly ash. As bottom ash and fly ash are both known to be somewhat susceptible to liquefaction, an assessment of the potential impact on loss of subgrade support to the raised dike sections is a key consideration in the assessments.

For most of the other management units I have visited, the impoundments were expanded by *building out* on the downstream side of the dikes, eliminating the situation of building on the existing ash layer. The one site that did expand inward conducted a liquefaction analysis which indicated a potential for liquefaction in the ash at certain groundwater elevations. In that case the utility combined a groundwater monitoring system and construction schedule in an effort to prevent groundwater elevation

[1] Winterkorn, H.F., and Fang, H., *Foundation Engineering Handbook. Van Nostrand Reinhold, Ltd., New York, NY, 1975, pg. 268*

increases. If the approach proved to be unsuccessful, the utility had a drainage system design ready to be installed to stabilize the embankment against a potential liquefaction failure.

Because the assessments are qualitative rather than quantitative, I elected not to consider the results as indicative of either SATISFACTORY or UNSATISFACTORY. The assessed liquefaction condition at each impoundment is presented as either NO CONCERN or CONCERN. Each impoundment is assessed based on the natural foundation soils at the site, and the supporting material of raised dike sections. A composite rating is provided as described below.

The evaluations are based on the embankment cross-sections used in the recent (February 2012 and April 2012) pseudo static slope stability analyses conducted by Stantec Consulting Services for TVA.

Foundation Rating

Foundation soils are rated not only on the presence of liquefaction susceptible soils, but also the depth and thickness of the stratum, the slope of the base of the stratum, and whether the stratum extends beneath the base dike, or is restricted to the impoundment area. A CONCERN rating indicates the presence of soils susceptible to liquefaction at a relatively shallow depth below the embankment, and sufficiently thick to result in substantial deformations to the embankment in the event liquefaction occurs.

Dike Rating

Dikes were rated based on the presence of bottom ash, fly ash or other CCR material underlying raised dike sections. If the CCR material supported 50 percent or more of the raised dike, the dike received a CONCERN rating.

Composite Ratings

Composite ratings are based on a judgment of deformations that may occur to the embankments in the event of liquefaction of materials supporting the initial and/or raised dikes, The rating reflects the potential volume of material released in the event of an embankment failure, and the nature of the adjoining area expected to receive the outflow. In most cases, the controlling parameter for each perimeter dike is the potential failure of raised dikes supported in part by CCR material. Conversely, the controlling factor for interior dikes is the foundation rating.

Results

Table 1 presents a summary of the results of this assessment.

Plant	Impoundment	Liquefaction Stability Rating		
		Foundation	Dikes	Composite
Bull Run	Disposal Area 2A	NO CONCERN	CONCERN	**CONCERN**
	Disposal Area 2	NO CONCERN	NO CONCERN	NO CONCERN
	Bottom Ash Disposal Area 1	NO CONCERN	CONCERN	**CONCERN**
Colbert	Ash Pond 4	CONCERN	CONCERN	**CONCERN**
	Ash Pond 5	NO CONCERN	NO CONCERN	NO CONCERN
Cumberland	Ash Pond	NO CONCERN	NO CONCERN	NO CONCERN
Gallatin	Ash Pond A	NO CONCERN	CONCERN	NO CONCERN
	Ash Pond E	NO CONCERN	CONCERN	NO CONCERN
John Sevier	Bottom Ash Pond	NO CONCERN	NO CONCERN	NO CONCERN
	Ash Disposal Area J	NO CONCERN	NO CONCERN	NO CONCERN
Johnsonville	Ash Disposal Area 2	NO CONCERN	CONCERN	**CONCERN**
Kingston	Ash Pond Dike C	CONCERN	CONCERN	**CONCERN**
	Gypsum Stack	NO CONCERN	NO CONCERN	NO CONCERN
Widows Creek	Main Ash Pond Complex	NO CONCERN	CONCERN	**CONCERN**
	Gypsum Stack	NO CONCERN	NO CONCERN	NO CONCERN

The embankment composite ratings at Gallatin Fossil Plant are the exception to the general case of the dike rating being the controlling factor. Gallatin Ash Pond A embankment is an interior dike separating Ash Pond A and Stilling Pond B. Failure of the embankment due to liquefaction of the supporting ash would result in an intermingling of ash and decant water within the impoundment, a release from the impoundment would not be expected to occur.

Gallatin Ash Pond E is supported on an underlying layer of ash that extends beyond the toe of the embankment to a natural slope, expected to be the excavation limits for the original impoundment area. Failure of the Ash Pond E due to liquefaction of the underlying material is not expected to result in a significant release beyond the boundaries of the current impoundment,

Conclusions

Based on the results of this review, the stability of six impoundments is rated as CONCERN relative to potential liquefaction during a seismic event.

As previously discussed, the embankment stability ratings are based on a qualitative review of the current geotechnical data. More rigorous analytical assessments may arrive at different results. Such analyses should evaluate both the likelihood of liquefaction occurring from susceptible soils in the event of the design earthquake, and the effects of liquefaction on the embankments. The second phase of analyses is important to assess the risk posed by potential liquefaction of (or beneath) the CCR impoundment embankments.

Limitations

Our assessment of the stability of CCR impoundment embankments includes evaluation of many variables, including liquefaction potential. Most of the other variables have data developed with significantly more technical rigor than this qualitative assessment. *Therefore, I caution against using the results of this assessment as a primary determinant on the overall rating of a CCR impoundment.* Although reasonable judgment was used throughout the evaluation, uncertainties were evaluated using the most conservation assumptions.

Further, it is likely that the geotechnical data provided by TVA is "inconsistent" with the data (i.e., procedure) used in the Foundation Engineering Handbook (Footnote 1) to develop correlations with liquefaction susceptibility and N-values. That is, information in the TVA geotechnical reports indicate that the Standard Penetration Tests were conducted using an automatic hammer to drive the sampler. Research has shown that automatic hammers impart a significantly higher percentage of the theoretical maximum hammer to the drive anvil energy than achieved by traditional manual methods using a rope and cathead to raise and release the hammer. The result is that TVA's recorded N-values can be expected to be lower than those achieved by manual hammers in use at the time the industry-practice (i.e., Handbook) liquefaction correlations were developed.

Further, the sand strata encountered at TVA sites were below the ground water level. The boring logs indicated borings were advanced using a hollow stem auger. Hollow stem augers are a standard method for advancing soil borings, and comply with ASTM requirements. However, it is difficult to maintain the required hydrostatic head inside the augers while inserting and removing the sampler. If the hydrostatic head is not maintained, an upward gradient can develop at the tip of the auger which also reduces the N-value below the theoretical value.

It is for these reasons that the results of this assessment should not be used as the primary determinate of the overall rating for an embankment.