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Coal Combustion Residue Impoundment

Round 11 - Dam Assessment Report

Kingston Fossil Plant
Tennessee Valley Authority
Harriman, Tennessee

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 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. We must marshal our best efforts to prevent such catastrophic failure and damage. 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 two Kingston Fossil Plant Coal Combustion Residuals (CCR) management units: Ash Pond C and Flue Gas Desulfurization System Landfill (Gypsum Disposal Facility), is based on a review of available documents and on the site assessment conducted by Dewberry personnel on Wednesday, September 21, 2011. We found the supporting technical documentation inadequate (Section 1.1.3). As detailed in Section 1.2.1, this report recommends that a quantitative liquefaction analysis be performed for the Ash Pond C management unit.

In summary, the Ash Pond C is rated FAIR and the Gypsum Disposal Unit is SATISFACTORY for continued safe and reliable operation, with no recognized existing or potential management unit safety deficiencies.

PURPOSE AND SCOPE

The U.S. Environmental Protection Agency (EPA) is investigating 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 early 2009, the EPA sent a first wave of 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

DRAFT

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.

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 management units and to determine the 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 Ash Pond C and the Gypsum Disposal 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).

Note: The terms "embankment", "berm", "dike", and "dam" are used interchangeably within this report, as have the terms "pond", "basin", and "impoundment".

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.

US EPA ARCHIVE DOCUMENT

US EPA ARCHIVE DOCUMENT

DRAFT

Table of Contents

Page

INTRODUCTION, SUMMARY CONCLUSIONS AND RECOMMENDATIONS II

PURPOSE AND SCOPE II

1.0 CONCLUSIONS AND RECOMMENDATIONS 1-1

1.1 CONCLUSIONS 1-1
1.1.1 Conclusions Regarding the Structural Soundness of the Management Unit(s) 1-1
1.1.2 Conclusions Regarding the Hydrologic/Hydraulic Safety of the Management Unit(s) 1-1
1.1.3 Conclusions Regarding the Adequacy of Supporting Technical Documentation 1-2
1.1.4 Conclusions Regarding the Description of the Management Unit(s) 1-2
1.1.5 Conclusions Regarding the Field Observations 1-2
1.1.6 Conclusions Regarding the Adequacy of Maintenance and Methods of Operation 1-2
1.1.7 Conclusions Regarding the Adequacy of the Surveillance and Monitoring Program 1-3
1.1.8 Classification Regarding Suitability for Continued Safe and Reliable Operation 1-3
1.2 RECOMMENDATIONS 1-3
1.2.1 Recommendations Regarding Continued Safe and Reliable Operation 1-3
1.3 PARTICIPANTS AND ACKNOWLEDGEMENT 1-4
1.3.1 List of Participants 1-4
1.3.2 Acknowledgement and Signature 1-4

2.0 DESCRIPTION OF THE COAL COMBUSTION RESIDUE MANAGEMENT UNIT(S) 2-1

2.1 LOCATION AND GENERAL DESCRIPTION 2-1
2.2 COAL COMBUSTION RESIDUE HANDLING 2-3
2.2.1 Fly Ash 2-3
2.2.2 Bottom Ash 2-3
2.2.3 Boiler Slag 2-4
2.2.4 Flue Gas Desulfurization Sludge 2-4
2.3 SIZE AND HAZARD CLASSIFICATION 2-4
2.4 AMOUNT AND TYPE OF RESIDUALS CURRENTLY CONTAINED IN THE UNIT(S) AND MAXIMUM CAPACITY 2-6
2.5 PRINCIPAL PROJECT STRUCTURES 2-6
2.5.1 Earth Embankment 2-6
2.5.2 Outlet Structures 2-7
2.6 CRITICAL INFRASTRUCTURE WITHIN FIVE MILES DOWN GRADIENT 2-8

3.0 SUMMARY OF RELEVANT REPORTS, PERMITS, AND INCIDENTS 3-1

3.1 SUMMARY OF LOCAL, STATE, AND FEDERAL ENVIRONMENTAL PERMITS 3-1
3.2 SUMMARY OF SPILL/RELEASE INCIDENTS 3-2

4.0 SUMMARY OF HISTORY OF CONSTRUCTION AND OPERATION 4-1

4.1 SUMMARY OF CONSTRUCTION HISTORY 4-1
4.1.1 Original Construction 4-1
4.1.2 Significant Changes/Modifications in Design since Original Construction 4-1
4.1.3 Significant Repairs/Rehabilitation since Original Construction 4-2

DRAFT

4.2 SUMMARY OF OPERATIONAL PROCEDURES 4-3
4.2.1 Original Operational Procedures 4-3
4.2.2 Significant Changes in Operational Procedures and Original Startup 4-3
4.2.3 Current Operational Procedures 4-4
4.2.4 Other Notable Events since Original Startup 4-5

5.0 FIELD OBSERVATIONS 5-1

5.1 PROJECT OVERVIEW AND SIGNIFICANT FINDINGS 5-1
5.2 ASH POND C 5-1
5.2.1 Crest 5-1
5.2.2 Upstream/Inside Slope 5-2
5.2.3 Downstream/Outside Slope and Toe 5-3
5.2.4 Abutments and Groin Areas 5-4
5.3 GYPSUM DISPOSAL FACILITY 5-4
5.3.1 Crest 5-4
5.3.2 Upstream/Inside Slope 5-5
5.3.3 Downstream/Outside Slope and Toe 5-5
5.3.4 Abutments and Groin Areas 5-7
5.4 OUTLET STRUCTURES 5-7
5.4.1 Overflow Structure 5-7
5.4.2 Outlet Conduit 5-9
5.4.3 Emergency Spillway 5-11
5.4.4 Low Level Outlet 5-11

6.0 HYDROLOGIC/HYDRAULIC SAFETY 6-1

6.1 SUPPORTING TECHNICAL DOCUMENTATION 6-1
6.1.1 Flood of Record 6-1
6.1.2 Inflow Design Flood 6-1
6.1.3 Spillway Rating 6-3
6.1.4 Downstream Flood Analysis 6-3
6.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION 6-4
6.3 ASSESSMENT OF HYDROLOGIC/HYDRAULIC SAFETY 6-4

7.0 STRUCTURAL STABILITY 7-1

7.1 SUPPORTING TECHNICAL DOCUMENTATION 7-1
7.1.1 Stability Analyses and Load Cases Analyzed 7-1
7.1.2 Design Parameters and Dam Materials 7-2
7.1.3 Uplift and/or Phreatic Surface Assumptions 7-4
7.1.4 Factors of Safety and Base Stresses 7-4
7.1.5 Liquefaction Potential 7-6
7.1.6 Critical Geological Conditions 7-7
7.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION 7-7
7.3 ASSESSMENT OF STRUCTURAL STABILITY 7-7

US EPA ARCHIVE DOCUMENT

DRAFT

8.0 ADEQUACY OF MAINTENANCE AND METHODS OF OPERATION 8-1

8.1 OPERATING PROCEDURES 8-1
8.2 MAINTENANCE OF THE DAM AND PROJECT FACILITIES 8-1
8.3 ASSESSMENT OF MAINTENANCE AND METHODS OF OPERATIONS 8-2
8.3.1 Adequacy of Operating Procedures 8-2
8.3.2 Adequacy of Maintenance 8-2

9.0 ADEQUACY OF SURVEILLANCE AND MONITORING PROGRAM 9-1

9.1 SURVEILLANCE PROCEDURES 9-1
9.2 INSTRUMENTATION MONITORING 9-1
9.3 ASSESSMENT OF SURVEILLANCE AND MONITORING PROGRAM 9-1
9.3.1 Adequacy of Inspection Program 9-1
9.3.2 Adequacy of Instrumentation Monitoring Program 9-1

APPENDIX A

Doc 01: *Report of Geotechnical Exploration and Slope Stability for Dike C, Kingston Fossil Plant, Harriman, Tennessee,* Stantec Consulting Services, August 3, 2009

Doc 02: Construction drawings "Coal Combustion By-Products (Gypsum) Disposal Facility, Kingston Fossil Plant – Peninsula Site" TVA Drawing Numbers 10W428-1 through 10W428-22

Doc 03: TVA Spreadsheet "CCR Generation and Handling," dated September 27, 2011

Doc 04: *Tennessee Valley Authority (TVA) Routine Handling Operations and Maintenance (RHO&M) Operations Support Document, Kingston Fossil Plant,* July 2011

Doc 05: Stantec Consulting Services correspondence dated October 4, 2011 "Reference: Dam Safety Hazard Classification Review, Ash Disposal Area 2, Kingston Fossil Plant, Roane County, Tennessee"

Doc 06: "Plans for Construction, Diffuser Replacement Project, Stilling Pond Work Plan 6 (KIF-110503-WP-6) Kingston Fossil Plant" Drawing Numbers 81W403-01 through 81W403-07

Doc 07: Doc *2010 Annual Inspection of CCP Facilities and Ponds, Kingston Fossil Plant, Harriman, Tennessee*, Stantec Consulting Services, June 1, 2010

Doc 08: NPDES Permit No. TN0005452, TVA – Kingston Fossil Plant, Harriman, Roane County, Tennessee

Doc 09: NPDES Permit No. TN0080870, TVA – Kingston Fossil Plant – Flue Gas Desulfurization System Discharges, Harriman, Roane County, Tennessee

Doc 10: "Construction Drawings, Phase 1A Area, Gypsum Disposal Facility, Kingston Fossil Plant – Peninsula Site," Drawing Nos. 10WXXX-1 through 10WXXX-16, dated July, 2011, prepared by Geosyntec Consultants

US EPA ARCHIVE DOCUMENT

DRAFT

Doc 11: *Report of Hydrologic and Hydraulic Analysis, Ash Pond and Stilling Pond, TVA Kingston Fossil Plant, Roans County, Tennessee,* Stantec Consulting Services, September 30, 2010

Doc 12: Stantec Consulting Services correspondence dated October 11, 2011 "Re: Addendum, September 30, 2010 H&H Report – KIF Ash Pond/Stilling Basin, Dam Safety Regulation – Spillway Capacity Evaluation"

Doc 13: Geosyntec Consultants Technical Memorandum, October 14, 2011, "Subject: Evaluating Stormwater Runoff in the Gypsum Disposal Area for the One-Third Probable Maximum Precipitation Event

Doc 14: Stantec Consulting Services correspondence dated October 5, 2011 "Re: Results of Seismic Slope Stability Analysis, Main Ash Pond, Stilling Pond, Dike C, Kingston Fossil Plant"

Doc 15: Geosyntec Consultants "Site response and Seismic Deformation Analyses, Kingston Peninsula Permit Modification," November 10, 2010

Doc 16: *Design Report for Dike C Buttress, Kingston Fossil Plant, Harriman, TN*, Stantec Consulting Services, Inc., March 1, 2011

Doc 17: *Seepage Action Plan (SAP), Kingston Fossil Plant, Harriman, Roans County, Tennessee,* Stantec Consulting Services, June 10, 2010

Doc 18: Stantec Consulting Services Memo dated August 8, 2011 "Reference: Dike C Slope Inclinometer Data Weekly Review"

Doc 19: Stantec Consulting Services Memo dated August 8, 2011, "Reference" Dike C Weekly Piezometer Data Review (Data from July 22, 2011 to August 5, 2011)"

Doc 20: 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.

Doc 21: Geosyntec Consultants "Foundation Stability Analyses", Kingston Peninsula Permit Modification, October 29, 2010

Doc 22: Dewberry Memorandum, "Qualitative Assessment. Liquefaction Potential, TVA Fossil Plant CCR Impoundments, Dewberry Project No. 50047151", May 25, 2012.

APPENDIX B

Doc 23: Dam Inspection Check List Form: Ash Pond C

Doc 24 Dam Inspection Check List Form: Gypsum Disposal Facility

US EPA ARCHIVE DOCUMENT

DRAFT

1.0 CONCLUSIONS AND RECOMMENDATIONS

1.1 CONCLUSIONS

Conclusions are based on visual observations from a one-day site visit, October 21, 2011, and review of technical documentation provided by the Tennessee Valley Authority (TVA).

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

Dike C impounding Ash Pond C, and the dike impounding the Gypsum Disposal Facility appear to be structurally sound based on a review of the engineering data provided by the owner's technical staff and Dewberry's engineers' observations during the site visit. Remediation of Dike C was substantially complete in each area at the time of the site visit. However a 2009 geotechnical report indicated slope Factors of Safety less than the required minimum value of 1.5. The 2011 design report for the remediation measures includes updated slope stability analyses demonstrating the long term Factors of Safety were equal to or greater than 1.5.

No liquefaction evaluation was performed for the dikes of Ash Pond C or the Gypsum Disposal Facility. TVA stated during the site visit that they plan on performing such analyses upon closure of Ash Pond C, and Phase 1 of the Gypsum Disposal Facility. Phase 2 of the Gypsum Disposal Facility has been redesigned and is being constructed to accept dry product only. Without information concerning potential releases of CCR as a result of liquefaction under seismic conditions, for dikes that could fail, such as Ash Pond C, the dikes cannot be rated Satisfactory.

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

Hydrologic and hydraulic analyses provided to Dewberry indicate that Ash Pond C has adequate impoundment capacity to contain the 1 percent probability storm without overtopping either the Ash Pond or an adjacent Settling Pond. The analyses indicate that the 6-hour Probable Maximum Precipitation (PMP) event would result in overtopping the Ash Pond embankment. However, subsequent analyses indicate that the Ash Pond has adequate capacity to store one-half the 6-hour PMP event without overtopping. Capacity to store one-half the 6-hour PMP event meets the

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design requirements of the current Tennessee dam safety regulations for intermediate, significant hazard potential dams.

The Hydrologic and hydraulic analyses indicate the Gypsum Disposal Facility stormwater pond can retain the 1/3 – six-hour PMP event without overtopping which is the design event required by Tennessee dam safety regulations for Small, Significant hazard dams.

1.1.3 Conclusions Regarding the Adequacy of Supporting Technical Documentation

The supporting technical documentation is inadequate, due to the lack of quantitative analysis of liquefaction potential. 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 to have no signs of overstress, significant settlement, shear failure, or other signs of instability although visual observations were hampered by the presence of thick vegetation in some areas. 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 Ash Pond C and the Gypsum Disposal Facility. There was no evidence of significant embankment repairs or prior releases observed during the field inspection

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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.

The Ash Pond C embankments are monitored with piezometers and slope inclinometers. The Gypsum Disposal Facility is monitored with piezometers.

1.1.8 Classification Regarding Suitability for Continued Safe and Reliable Operation

The Ash Pond C is rated FAIR and the Gypsum Disposal Facility is rated SATISFACTORY for continued safe and reliable operation based on visual assessment and the pertinent technical documentation provided. Implementation of the recommendations described in 1.2 would help improve the ratings.

1.2 RECOMMENDATIONS

1.2.1 Recommendations Regarding Continued Safe and Reliable Operation

It is anticipated that both Ash Pond C and the Gypsum Disposal Facility will be considered SATISFACTORY for continued safe and reliable operations upon:

A determination that there is no liquefaction potential for soils and materials at the management units, particularly Ash Pond C, under the design seismic event.

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1.3 PARTICIPANTS AND ACKNOWLEDGEMENT

1.3.1 List of Participants

John Dizer, TVA
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Kathryn Nash, TVA
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Hank Julian, Geosyntec Consultants
Craig Zeller, U.S. EPA, Region 4
Jim Kohler, U.S. EPA Headquarters
Joe Klein, Dewberry
Stan Notestine, Dewberry

1.3.2 Acknowledgement and Signature

We acknowledge that the management unit referenced herein has been assessed on September 21, 2011

Stanley W. Notestine, P.E.
Registered, TN

Joseph P. Klein, III, P.E.

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

2.1 LOCATION AND GENERAL DESCRIPTION

The Kingston Fossil Plant is located on the west side of Watts Bar Lake at the confluence of the Emory and Clinch Rivers, north of Kingston, Tennessee. Construction of the 1,723 MW plant began in 1951 and the plant went into commercial production in 1954. The Emory River and Watts Bar Lake border the site on the north, east and south sides. Pine Ridge, a local topographic feature borders the site to the west. The site location and area topography are shown on Figure 2.1-1.

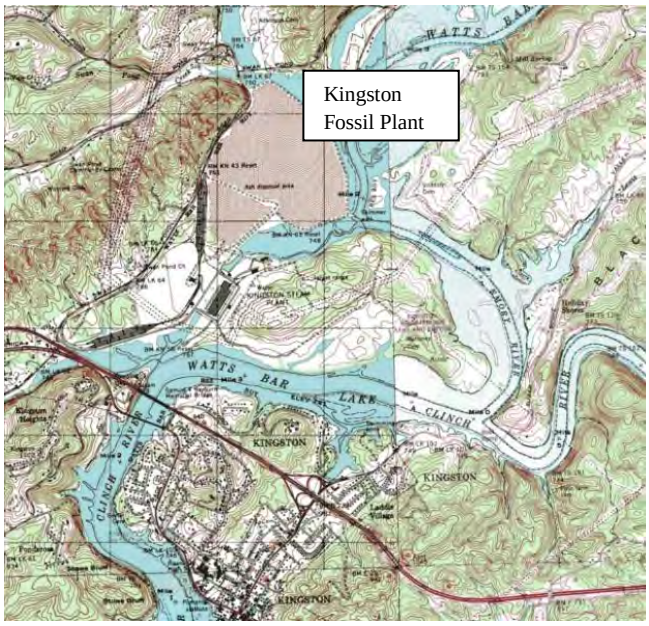


Figure 2.1-1: Kingston Fossil Plant Location Plan

At the time of Dewberry's site visit the plant had one active coal combustion residuals (CCR) impoundment and another under construction.

Ash Pond C

The active CCR management unit is designated Ash Pond C. Ash Pond C includes an ash storage area and a settling pond separated by an internal ash dike. Ash Pond C consists of the southern portion of the original 245 acre Main Ash Pond. Ash Pond C is impounded on the east, south and west sides by Dike C. Ash Pond C is separated from the northern portion of the original Main Ash Pond to the north by Dike D. The northern portion of the Main Ash Pond is designated for use as dredge

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cells but is currently out of service during major restoration due to a 2008 breach in the dredge cell eastern embankment.

Flue Gas Desulfurization System Landfill

The Flue Gas Desulfurization System landfill (i.e., Gypsum Disposal Facility) is located on a peninsula to the east of the main generation plant. The landfill, under construction at the time of Dewberry's site visit, includes an impoundment for dry stacking gypsum, and a storm water pond abutting the west end of the gypsum stack impoundment. The gypsum stack cell encompasses about 50 acres, and the storm water pond encompasses about 6 acres. The height of storm water pond embankment is about 25 feet.

Table 2.1 provides a summary of the dimensions of the embankments. Figure 2.1-2 shows the location of the CCR facilities on the plant site.

Table 2.1: Summary of Dam Dimensions and Size

	Ash Pond C[1]	Flue Gas Desulfurization Landfill[2]	
		Gypsum Stack	Storm Water Pond
Dam Height (ft)	30	30	25
Crest Width (ft)	25	25	25
Length (ft)	6,000	6,000	2,200
Side Slopes (upstream) H:V	1.5(H):1(V) to 3.0(H):1(V)	3.0(H):1(V)	2.5(H):1(V) to (3.0:1(v)
Side Slopes (downstream) H:V	1.6(H):1.0(V)	3.0(H):1(V)	2.5(H):1(V) to (3.0):1(v)

[1] Based on information from *Report of Geotechnical Exploration and Slope Stability for Dike C, Kingston Fossil Plant, Harriman, Tennessee*, Stantec Consulting Services, August 3, 2009 (See Appendix A – Doc 01)

[2] Based on information from Drawings for the "Coal Combustion By Product (Gypsum) Disposal Facility, Kingston Fossil Plant – Peninsula Site", drawing numbers 10W428 -12 through 14 (See Appendix A – Doc 02).

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Figure 2.1-2: CCR Impoundment on Kingston Plant Site

2.2 COAL COMBUSTION RESIDUE HANDLING

Dewberry was provided general information regarding the type of equipment used in the handling, transportation and disposition of various coal combustion residuals (See Appendix A – Doc 03)

2.2.1 Fly Ash

Equipment used for the collection of fly ash included selective catalytic reductions (SCR) hoppers, economizer hoppers, precipitator hoppers, jet pumps, hydroveyors, air separator tanks and piping. SCR hoppers, economizer hoppers, precipitator hoppers are located inside the plant building. Other equipment is located outside.

2.2.2 Bottom Ash

Equipment used for the collections of bottom ash includes bottom ash hoppers, jet pumps and piping. The bottom ash hoppers are located inside the plant building. Other equipment is located outside.

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DRAFT

2.2.3 Boiler Slag

Boiler slag is not produced at the Kingston Fossil Plant

2.2.4 Flue Gas Desulfurization Sludge

Equipment used for the collection, handling and disposition of Flue Gas Desulfurization Sludge at Kingston Fossil Plant included limestone preparation facilities, absorbers, recycle pumps, area sump and piping. Equipment is outside the plant building.

2.3 SIZE AND HAZARD CLASSIFICATION

Ash Pond C

Dike C, which forms the perimeter of Ash Pond C on three sides, was constructed in three stages. The first stage consisted of a starter dike constructed in 1954 to a crest elevation of 746 feet. The starter dike was constructed of residual clay soil and bottom ash. In the second stage, constructed in 1974, the crest was raised about 1.5 feet by constructing a compacted bottom ash cap over the crest of the existing embankment. In the final stage, constructed in the late 1970s, Dike C was raised to a crest elevation of 765 feet by constructing a raised dike extending upward and inward from the inside edge of the Dike C starter dike cap. The three stage configuration of Dike C impounding Ash Pond C is about 30 feet high and 2,800 feet long.

Dike C abuts Dike D to the north on both the east and west sides of Ash Pond C. Ash Pond C impounds about 120 acres.

Flue Gas Desulfurization Landfill Storm Water Pond

The Flue Gas Desulfurization Landfill Storm Water Pond is a “V” shaped impoundment formed by a clay fill embankment abutting the west side of the first phase gypsum stack cell that was under construction at the time of Dewberry’s site visit. The embankment impounding the storm water pond is 2,200 feet long and about 25 feet high. The storm water pond impounds about 6 acres.

The classification for size, based on the height of the embankments and the impoundment storage capacity is “Intermediate” for Ash Pond C, and “Small” for the Flue Gas Desulfurization Landfill Storm Water Pond according to the USACE recommended guidelines for safety inspection of dams, ER 1110-2-106, as shown in Table 2.2a.

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

[1] Ash Pond C
[2] Flue Gas Desulfurization Landfill Storm Water Pond

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

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)

TVA commissioned a hazard classification study that was completed in October, 2011. The study concluded that a release from Ash Pond C due to failure or misoperation was not expected to result in the loss of human life, or damage critical infrastructure. However, a release is expected to impact off-site waters of the United States., including Watts Bar Reservoir. Therefore the study concluded that Ash Pond C represented a "Significant" hazard (See Appendix A – Doc 05).

As the Flue Gas Desulfurization landfill and storm water pond is about 200 feet from the Watts Bar Reservoir, it is likely that release due to a failure or misoperation would impact off-site waters of the United States. However a release is not expected to cause a loss of human life, or impact critical infrastructure.

As a release from either or both the Ash Pond C and/or the Flue Gas Desulfurization ash stack or stormwater pond is not expected to cause loss of human life, or damage to critical infrastructure, but is expected to cause potential

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environmental and economic losses beyond the plant boundaries, Dewberry evaluated both facilities as being a **Significant Hazard Potential.**

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

Ash Pond C receives fly ash and bottom ash from the plant. The Gypsum Disposal Facility is currently empty and under reconstruction.

Table 2.3: Maximum Capacity of Unit

Ash Pond Name	Ash Pond C Complex[1]	Gypsum Disposal Facility[2]
Surface Area (acre)[1]	120	6
Current Storage Capacity (cubic yards)[1]	706,300	DNR[3]
Current Storage Capacity (acre-feet)	435	DNR
Total Storage Capacity (cubic yards)[1]	1,534,110	DNR
Total Storage Capacity (acre-feet)	950	DNR
Crest Elevation (feet)	765	769
Normal Pond Level (feet)[4]	760/754.4	759

[1] *Report of Hydrologic and Hydraulic Analysis, Ash Pond and Stilling Pond, Kingston Fossil Plant, Roane County, Tennessee*, Stantec Consulting Services, September 30, 2012 (See Appendix A – Doc 11)
[2] Construction drawings "Coal Combustion by Products (Gypsum) Disposal Facility, Kingston Fossil Plant Peninsula Site" (See Appendix A – Doc 02)
[3] DNR: Data Not Received
[4] Ash Pond C normal pool elevations for Ash Pond/Settling Pond

Although the data provided for review indicates that the capacity and height of Ash Pond C might allow it to be classified a "Small" impoundment, Dewberry concluded that based on the expected precision of the data it was appropriate to use the "Intermediate" classification. The Stantec report (Appendix A, Doc 05) also evaluated Ash Pond C as an "Intermediate" impoundment.

2.5 PRINCIPAL PROJECT STRUCTURES

2.5.1 Earth Embankment

Ash Pond C

The exterior perimeter of Ash Pond C is formed by Dike C. Dike C consists of a starter dike constructed in 1954 to a crest elevation of 746 feet. The starter dike was constructed of residual clay soil and bottom ash.

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In 1974 the crest was raised about 1.5 feet by construction of a compacted bottom ash cap over the crest of the existing embankment. In the late 1970s Dike C was raised to a crest elevation of 765 by constructing a raised dike extending upward and inward from the inside edge of the Dike C ash cap. Dike C abuts Dike D to the north on both the east and west sides of Ash Pond C. Dike D separates Ash Pond C from the adjacent Dredge Dike. (See Appendix A – Doc 04). Dike C impounding Ash Pond C is about 2,800 feet long and about 30 feet high. Ash Pond C encompasses about 120 acres.

Gypsum Disposal Facility Storm Water Pond

Gypsum Disposal Facility Storm Water Pond is a "V" shaped impoundment formed by a clay fill embankment abutting the west side of the first phase ash stack cell that was under construction at the time of Dewberry's site visit. The embankment impounding the storm water pond is 2,200 feet long and about 25 feet high. The storm water pond impounds about 6 acres.

2.5.2 Outlet Structures

Ash Pond C

Outlet structures for Ash Pond C are located near the southeast corner of the stilling pond. The original outlet system consists of six 48-inch diameter, reinforced concrete pipe overflow risers with corrugated metal skimmers and trash racks. Each riser is connected to a 36-inch diameter ductile iron pipe that runs through the starter dike to a headwall near the outside toe. At the headwall, the discharge pipe is reduced to a 24-inch diameter pipe that discharges into the plant water intake channel from the Emory River. The original discharge pipe was ductile iron; however it was changed to 24-inch diameter HDPE pipe in conjunction with the addition of a new siphon system in 2011 (See Appendix A doc 06).

A siphon system was added to the outlet structure in 2011. The siphon system consists of four 24-inch diameter HDPE pipes located a short distance east of the overflow risers. The siphon pipes discharge at elevation 743, about two feet above the Emory River summer pool elevation.

Gypsum Disposal Facility

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The Gypsum Disposal Facility is designed to receive sluiced gypsum from the plant into the lined gypsum cell. Sluice water and storm water seepage through the gypsum are collected in an under drain system and piped by gravity to a lift station constructed on the outside slope of the embankment near the southeast corner of the gypsum cell. Water from the lift station is pumped to the adjacent Gypsum Disposal Facility storm water pond.

As the gypsum stack increases in elevation above the crest of the embankment, surface runoff is collected in a perimeter ditch that discharges into three drop inlets located in the northwest corner of the impoundment. A 91-inch diameter reinforced concrete pipe from each drop inlet carries water through the embankment into the landfill storm water pond.

The outlet structure for the storm water pond consists of a lift station that pumps water to the main plant water intake channel for recycling.

2.6 CRITICAL INFRASTRUCTURE WITHIN FIVE MILES DOWN GRADIENT

The plant site drains generally to the south into the bordering Watts Bar Reservoir portion of the Clinch River. Land down gradient of the plant is generally undeveloped woodland. Interstate Highway 40 is about 3,500 to 4,000 feet south of the plant site. The Hazard Classification report indicates that roadways in the area are either above the elevation of the impoundment or are at a distance that would permit dissipation of a breach wave prior to reaching them (See Appendix A – Doc 05).

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3.0 SUMMARY OF RELEVANT REPORTS, PERMITS, AND INCIDENTS

TVA provided the 2010 annual inspection report prepared by Stantec Consulting Services for review (See Appendix A – Doc 07). The report does not indicate findings of significance for either Ash Pond C or the Flue Gas Desulfurization System landfill storm water pond. Recommendations presented in the report were generally related to routine maintenance issues including:

Ash Pond C

- Remove trees and bushes from embankment slopes
- Continue monitoring seepage areas in accordance with established inspection program
- Repair area of rutting and toe undercutting along a section of Dike C.

Gypsum Pond

- Repair areas of erosion along the interior of gypsum stack embankment
- Re-vegetate areas of the embankments lacking adequate ground cover.
- Check interior drainage pump station periodically for proper operation.

The 2011 annual inspection was scheduled for October, after Dewberry's site visit.

TVA provided Dewberry with copies of reports addressing the safe operation of Ash Pond C and the Gypsum Pond. These reports are provided in Appendix A.

3.1 SUMMARY OF LOCAL, STATE, AND FEDERAL ENVIRONMENTAL PERMITS

Tennessee dam safety laws and regulations are administered by the Department of Environment and Conservation. The law provides a waiver for projects approved by the federal government. As TVA is a federal entity, a state dam permit is not required.

Discharge from the impoundment is regulated by the Tennessee Department of Environment and Conservation. The Kingston Fossil Plant has been issued two National Pollutant Discharge Elimination System permits.

- Permit No. TN0005452 was issued September 1, 2003. The permitted discharges include fly ash and bottom ash sluice water. The permit expired August 31, 2008. TVA reported that it had submitted an application for renewal

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prior to February 28, 2008. Review of the renewal application by the Tennessee Department of Environment and Conservation has not been completed. (See Appendix A – Doc 08).

- Permit No. TN00080870 was issued September 22, 2009. The permitted discharge is for the Flue Gas Desulfurization System storm water pond. The permit expires September 30, 2013. (See Appendix A – Doc 09).

3.2 SUMMARY OF SPILL/RELEASE INCIDENTS

Ash Pond C

As discussed in Section 4 of this report, the original 275-acre ash pond has been subdivided several times during its operation. Between 1987 and 1988 the original configuration was changed by the construction of Dike D that divided the ash pond into north and south sections. The south section received sluiced ash from the plant and provided a stilling pond for the sluice water. The north section was subdivided into cells that received ash dredged from the southern portion of the facility.

On December 8, 2008 the northwestern portion of the dredge cell area failed. The progressive failure impacted a section of the northeast corner of Ash Pond C, abutting the east side of Dike D. The failure resulted in a major release of CCR material into the Emory River and Clinch River. A major recovery effort was undertaken, and major reconstruction of the dredge cells was under construction at the time of Dewberry's site visit.

Gypsum Stack Landfill

Excavation and construction of the gypsum stack impoundment began in 2008. On December 15, 2010 during initial operation of Phase 1, a dropout occurred resulting in halting gypsum production until the cause of the problem was identified and repaired. The cause was determined to be the development of a sinkhole and further investigation revealed other potential sinkholes at the site. The facility has been redesigned and reconstruction was in progress at the time of Dewberry's site visit.

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4.0 SUMMARY OF HISTORY OF CONSTRUCTION AND OPERATION

4.1 SUMMARY OF CONSTRUCTION HISTORY

4.1.1 Original Construction

Construction of the Kingston Fossil Plant began in 1951. In 1954 the plant went on-line with a capacity of 1,723 megawatts. At the start of plant operations, ash slurry was discharged directly to slack water created by Watts Bar Reservoir. From 1954 to 1976 ash was sluiced to a pond formed by the hydraulically placed ash (See Appendix A – Doc 04).

Ash Pond

In 1958 construction of a 275-acre ash pond was impounded by the completion of Dike C which was constructed of native clay and bottom ash. The original crest elevation of Dike C was 746 feet (See Appendix A – Doc 04).

Gypsum Impoundment

Excavation and construction of the new Gypsum Pond began in 2008 as the management unit to collect gypsum produced by new plant scrubbers. During start-up of the first phase of the facility a sinkhole was encountered that required the pond to be taken out of service (See Appendix A – Doc 04).

4.1.2 Significant Changes/Modifications in Design since Original Construction

Ash Pond C (See Appendix A – Doc 04)

In 1974 Dike C was raised about 1.5 feet to provide additional freeboard for the ash pond. The dike was raised by using bottom ash to construct a cap along the inside slope and over the crest.

Between 1976 and 1978 Dike C was raised to a crest elevation of 765 by extending the existing dike inward beginning at the inside edge of the existing crest. The raised dike section was constructed largely over deposited ash.

In 1983 an interior dike was constructed to divide the ash pond into two halves, with the northern half being the planned dredge cell. In 1984 the interior dike failed as a result of undermining during dredging operations.

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A replacement dike, designated Dike D, was constructed in its current location which is somewhat east of the original location. Dike D forms the boundary between Ash Pond C on the south and the dredge impoundment to the north.

Gypsum Disposal Facility (See Appendix A – Doc 04)

Due to the development of sinkholes in the gypsum stack impoundment during initial operations, significant design changes were made to the ash stack impoundment subgrade and interior drainage system. Construction implementing the design changes was underway at the time of Dewberry's site visit.

4.1.3 Significant Repairs/Rehabilitation since Original Construction

Ash Pond C (See Appendix A – Doc 04)

On December 22, 2008 an embankment failure occurred near the northwest corner of the dredge impoundment cell. The embankment failure impacted a large section of the north dike and the interior Dike D. A major repair/rehabilitation was begun immediately after the event and was continuing at the time of Dewberry's site visit. The work included construction of a buttress along Ash Pond C at the north end of the Dike D embankment, and repairs at the northeast corner of Ash Pond C adjacent to the failed embankment.

Gypsum Disposal Facility (See Appendix A – Doc 10)

Rehabilitation of the gypsum impoundment includes undercutting the stack impoundment subgrade by about 4 feet; proof-roll exposed subgrade; repair areas of potential sinkhole development, and replacement and re-compaction of the fill. Above the re-worked fill two layers of compacted clay and a multi-layer drainage system are being constructed. Construction of the gypsum stack impoundment drainage system was underway during Dewberry's site visit.

No need for rehabilitation of the stormwater pond has been identified.

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4.2 SUMMARY OF OPERATIONAL PROCEDURES

4.2.1 Original Operational Procedures

Main Ash Pond

At the start of operations in 1954 ash handling at the Kingston Fossil Plant was to discharge ash slurry directly into Watts Bar Reservoir. In 1958 a 275-acre ash impoundment was completed and served as the ash pond disposal site.

Gypsum Disposal Facility

As a result of a drop out encountered in the ash stack impoundment during start-up, the gypsum impoundment was taken out of services before full scale operations. The gypsum stack impoundment was out of service at the time of Dewberry's site visit with rehabilitation construction and rehabilitation work under way.

The storm water pond is in service. In addition to direct runoff, stormwater and seepage from the gypsum stack impoundment is pumped to the stormwater pond. Water from the storm water pond is pumped to the plant water intake channel for recycling.

4.2.2 Significant Changes in Operational Procedures and Original Startup

Main Ash Pond

During 1977 to 1978, as part of the project to raise the Main Ash Pond elevation, a stilling pond was created by constructing an interior dike to separate the eastern section of the main ash pond. The stilling pond was located on the east side of the facility to allow for additional settling of ash particles before water was discharged to Watts Bar Reservoir.

In 1983 an interior ash dike, designated Dike D, was constructed to divide the pond into two halves as a precursor to transition to using the north side of the pond as dredge cells, and the south side to receive sluiced ash and detain it to allow ash to settle out of suspension. In 1984 Dike D failed and was reconstructed east of the original construction.

Sluiced ash was piped into the pond to the south of Dike D. The pond south of Dike D was designated Ash Pond C. Sluiced ash flowed through the ash in an interior ditch to a stilling area. Decant water from the stilling

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pond flowed through a six pipe overflow riser system to the settling pond and then to the plant intake channel. In 2010-2011 a four pipe siphon spillway system from the stilling area to the Settling Pond was added.

Sedimented ash from Ash Pond C was dredged and stored in the north side dredge cell.

Gypsum Disposal Facility

The gypsum impoundment was undergoing subgrade reconstruction at the time of Dewberry's site visit and was out of service.

No CCR materials were stored in the impoundment at the time of site visit.

4.2.3 Current Operational Procedures

Ash Pond C

Fly ash and bottom ash are sluiced to Ash Pond C in five active spillway pipes. The sluiced ash is deposited in the northwest corner of Ash Pond C. Sluice water flows from the ash storage area of Ash Pond C into a stilling area. Decant water from the Ash Pond stilling area flows to the settling pond through spillway pipes located at the east end of an interior dike.

Gypsum Disposal Facility

The gypsum impoundment was not in operation at the time of Dewberry's site visit. Photograph 4.2.3-1 shows liner construction underway in progress at the time of Dewberry's site visit.

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Figure 4.2.3-1: Gypsum Disposal Facility Liner Construction Underway at Time of Dewberry's Site Visit

4.2.4 Other Notable Events since Original Startup

No other notable events have occurred since original startup.

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5.0 FIELD OBSERVATIONS

5.1 PROJECT OVERVIEW AND SIGNIFICANT FINDINGS

Dewberry personnel Stanley W. Notestine, P.E. and Joseph P. Klein, III, P.E. performed a site visit on Wednesday, September 21, 2011 in company with the participants.

The site visit began 9:00 AM. The weather was overcast with rain showers in the morning and clearing in the afternoon. Photographs were taken of conditions observed. Please refer to the Dam Inspection Checklist in Appendix B. Selected photographs are included here for ease of visual reference. All pictures were taken by Dewberry personnel during the site visit and provided to TVA at the conclusion of the site visit.

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

5.2 ASH POND C

5.2.1 Crest

The exterior perimeter of Ash Pond C is formed by Dike C. The crest of Dike C is a 20- to 25-foot wide gravel and ash roadway. There are no significant signs of depressions, tension cracks, settlement, or shear failure along the crest. Photograph 5.2.1-1 shows representative conditions observed along the crest of Dike C.

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Photograph 5.2.1-1: Ash Pond C Crest of South Embankment

5.2.2 Upstream/Inside Slope

The inside slopes of Dike C are vegetated with grass and weeds. Portions of the north end of the east embankment are covered with ash. There are no signs of slumps or slides in the visible portion of the slopes, and no significant erosion is visible. Photograph 5.2.2-1 shows a representative view of the inside slope of Dike C.



Photograph 5.2.2-1: Ash Pond C Inside Slope of South Embankment

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5.2.3 Downstream/Outside Slope and Toe

The west and south outside slopes of Dike C are vegetated with grass and weeds. A crushed stone buttress has been added to the outside slope of the east embankment starter dike. There are no signs of slumps or slides in the visible portion of the slopes, and no significant erosion is visible. Photograph 5.2.3-1 shows a representative view of the outside slopes of Dike C.



Photograph 5.2.3-1: Ash Pond C East Embankment Starter Dike Outside Slope. Grassed Outside Slope of Raised Dike Is Visible on Left Side

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5.2.4 Abutments and Groin Areas

There are no abutments in the portion of Dike C impounding Ash Pond C.

Exterior groins are vegetated, except for the southeast starter dike groin where a crushed stone buttress has been added. No signs of erosion or significant seepage are visible.

5.3 GYPSUM DISPOSAL FACILITY

5.3.1 Crest

The crest of the Gypsum Disposal Facility impoundment is a 20- to 25-foot wide, compacted clay and gravel roadway. There are no significant signs of depressions, tension cracks, settlement, or shear failure along the crest. Photograph 5.3.1-1 shows representative conditions observed along the crest of the east embankment of the Gypsum Disposal Facility impoundment.



Photograph 5.3.1-1: Gypsum Disposal Facility Impoundment Crest of East Embankment. East Side of Ash Stack Impoundment (Left Side of Photograph) Used as Lay Down Area for Liner Construction on West Side.

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5.3.2 Upstream/Inside Slope

Inside slopes of the Gypsum Disposal Facility ash stack impoundment not exposed for construction of the liner and subdrainage system are vegetated with grass. The inside slopes of portions of the ash stack under construction for liner installation are exposed soil subgrade.

Inside slopes of the storm water pond are armored with riprap to protect against erosion from wind driven waves. There are no signs of slumps or slides in the visible portion of the slopes, and no significant erosion is visible. Photograph 5.3.2-1 shows a representative view of the inside slopes of the Gypsum Disposal Facility.



Photograph 5.3.2-1: Gypsum Disposal Facility Storm Water Pond Inside Slopes

5.3.3 Downstream/Outside Slope and Toe

The outside slopes of the ash stack portion of the Gypsum Disposal Facility are vegetated with grass. Small areas of standing water were visible along the toe of the South slope. However, as the ash stack impoundment was empty, the wet areas are expected to be the result of rain showers earlier in the day.

The outside slope of the storm water pond is protected with stone riprap.

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There are no signs of slumps or slides in the visible portion of the slopes, and no significant erosion is visible. Photographs 5.3.3-1 and 5.3.3-2 show representative views of the outside slopes of the Gypsum Disposal Facility.



Photograph 5.3.3-1: Gypsum Disposal Facility Impoundment Outside Slope and Toe of South Embankment



Photograph 5.3.3-2: Gypsum Disposal Facility Storm Water Pond Outside Slope and Toe Area

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5.3.4 Abutments and Groin Areas

The Gypsum Disposal Facility east and west embankment abut a hillside on the north side of the impoundment. A topographic knoll near the southeast corner of the impoundment intruded into the embankment alignment creating a saddle like abutment in the south embankment. Photograph 5.3.4-1 shows the east embankment abutment.



Figure 5.3.4-1: Gypsum Disposal Facility East Embankment Abutment at North End.

Except where liner construction is underway, ash stack impoundment groins are vegetated with grass. Storm water pond groins are armored with riprap.

No signs of erosion or significant seepage are visible.

5.4 OUTLET STRUCTURES

5.4.1 Overflow Structure

Ash Pond C

The Ash Pond C overflow structure from the Ash Pond stilling area to the Settling Pond consists of five 48-inch diameter corrugated metal risers

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with corrugated metal skimmers and trash racks. Figure 5.4.1-1 shows the spillway risers.



Figure 5.4.4-1: Ash Pond Overflow Spillway Risers

The overflow structure from the Settling Pond consists of six 36-inch diameter pipe risers protected with 48-inch diameter corrugated metal skimmers and trash racks. Figure 5.4.1-2 shows the spillway risers.



Figure 5.4.1-2 Settling Pond Overflow Spillway Risers

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Gypsum Disposal

Facility

The original overflow structure from the gypsum stack to the storm water pond consists of a drop inlet located at the west side of the impoundment. The drop inlet receives surface runoff from an interior perimeter ditch. Design drawings for the construction in progress at the time of Dewberry's site visit indicate a new overflow structure consisting of three 91-inch diameter concrete risers hydraulically connected to the perimeter ditch and the new liner system (See Appendix A – Doc 10)

Ash stack interior drainage is piped to a lift station on the south side of the impoundment.

The storm water pond overflow structure consists of a reinforced concrete lift station shown in Figure 5.4.1-3.



Figure 5.4.1-3: Gypsum Disposal Facility Storm Water Pond Overflow Structure.

5.4.2 Outlet Conduit

Ash Pond C

The Ash Pond C Settling Pond spillway outlet conduits consist of six 36-inch diameter reinforced concrete pipes from the risers to a reducer

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located at the discharge structure headwall. At the headwall the outlet conduits are reduced to 24-inch diameter ductile iron pipes. The invert elevation of the conduits is below the normal pool elevation of the receiving plant water intake channel. Figure 5.4.2-1 shows the outlet conduits entering the plant intake water channel.



Figure 5.4.2-1: Ash Pond C Settling Pond Overflow Conduits to Plant Water Intake Channel

Gypsum Disposal Facility

The original outlet conduits for the ash stack drop overflow structure consist of three 48-inch diameter, reinforced concrete pipes (See Appendix A – Doc 02). The new outlet structure shown on the redesign drawings consist of three 91-inch diameter conduits (See Appendix A – Doc 10).

The ash stack interior drainage lift station outlet is a 4-inch diameter PVC pipe fed by a 200 gallon per minute submersible pump (See Appendix A – Doc 02). The lift station pumps water to the Gypsum Disposal Facility storm water pond.

The outlet from the storm water pond is pumped from a lift station at the south end of the pond to the plant intake water channel by three submersible turbine pumps, each linked to a 24-inch diameter discharge pipe.

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5.4.3 Emergency Spillway

Ash Pond C

The Ash Pond C embankment does not have an emergency spillway.

Gypsum Disposal Facility

The Gypsum Disposal Facility storm water pond has a riprap lined emergency spillway in the central length of the south embankment. The spillway has a width of 15 feet, side slopes of 8(H):1(V) and a depth of 2-feet (See Appendix A – Doc 02). Figure 5.4.3-1 shows the emergency spillway.



Figure 5.4.3-1: Gypsum Disposal Facility Riprap Lined Emergency Spillway

5.4.4 Low Level Outlet

Ash Pond C

Ash Pond C has a newly constructed low level outlet consisting of a four pipe siphon system. Each siphon pipe is a 24-inch diameter HDPE installed inside a 30-inch diameter casing. The siphon system discharges to the plant's water intake channel for recycling. Figure 5.4.4-1 shows the siphon inlet and Figure 5.4.4-2 shows the outlet.

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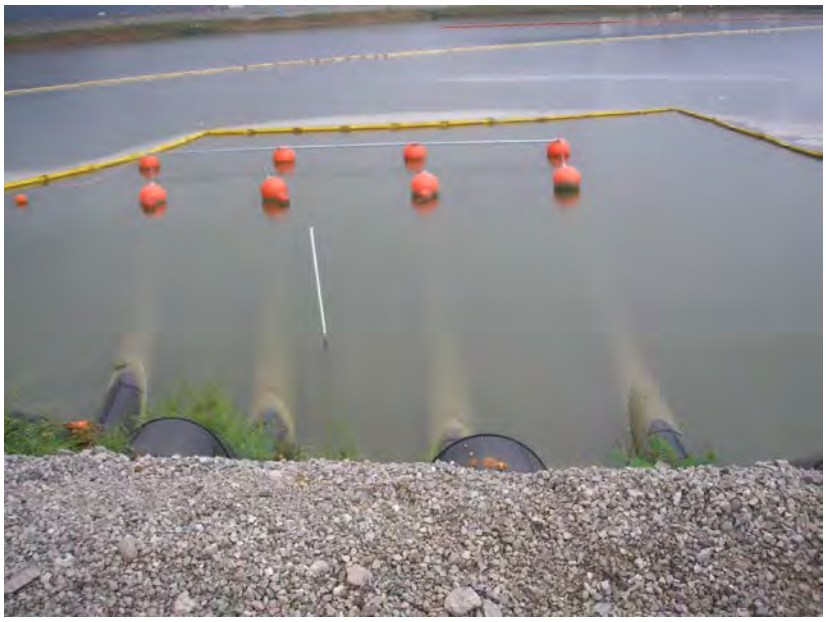


Figure 5.4.4-1: Ash Pond C Siphon System Inlet



Figure 5.4.4-2: Ash Pond C Siphon System 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

Although, as a federal entity, TVA is waved from compliance with Tennessee dam safety regulations, those regulations include criteria for evaluating the hydrologic and hydraulic safety of existing dams based on size and hazard rating. Dewberry classified both Ash Pond C and the Gypsum Disposal Facility as Significant hazards. The Tennessee hydrologic/hydraulic evaluation criteria are shown in Table 6.1.

Table 6.1: Tennessee Dam Safety Hydrologic/Hydraulic Criteria for Significant Hazard Dams Based on Size

Small	1/3 PMP
Intermediate	1/2 PMP
Large	PMP

Ash Pond C

TVA retained Stantec to conduct a hydrologic and hydraulic analysis of Ash Pond C. The analysis evaluated 24-hour peak storm events with recurrence intervals of 1, 10, 25, 50 and 100 years, and the 6-hour Probable Maximum Precipitation (PMP) (See Appendix A – Doc 11). The input parameters and results for the 24-hour, one percent probability (100-year storm) and the 6-hour PMP are shown in Table 6.2.

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Table 6.2: Hydrologic and Hydraulic Analysis Summary Ash Pond C				
Section	Embankment Crest Elevation	Normal Pool Elevation	Estimated Peak Pool Elevation	
			100 Year 24-Hour Storm	6-Hour PMP
Ash Pond	765.0	760.0	761.4	**765.4**
Stilling Pond	765.0	754.4	755.6	761.0

The results indicate that Ash Pond C is capable of containing the 24-hour, one percent probability storm event without overtopping. However, the Ash Pond dike is overtopped by the 6-hour PMP.

Stantec conducted a supplemental hydrologic and hydraulic analysis using the Tennessee dam safety criteria of one-half the 6-hour PMP applicable to Intermediate, Significant Hazard dams. The results indicate that the estimated peak pool elevation in the Ash Pond is 764.4, thus the embankment is not overtopped (See Appendix A – Doc 12).

Gypsum Impoundment

TVA retained Geosyntec Consultants to conduct a hydrologic/hydraulic analysis of the Gypsum Disposal Facility. (See Appendix A – Doc 13). The analysis was conducted using the Tennessee dam safety criteria of one-third the 6-hour PMP event applicable to Small, Significant Hazard dams. The input parameters and results for the one-third 6-hour PMP are shown in Table 6.3.

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Table 6.3: Hydrologic and Hydraulic Analysis Summary Gypsum Disposal Facility			
Area Section	**Embankment Crest Elevation**	**Normal Pool Elevation**	**Estimated Peak Pool Elevation**
Storm Water Pond	769	759.5	761
Gyp Pond Phase 1 A	780	756	759
Gyp Pond Phase 1B	770*	N/A	763

* Elevation of interior dike between Phase 1A and 1B

The results indicate the embankments will not be overtopped due to a one third 6-hour PMP event, nor would the pool elevation reach the invert of the stormwater pond emergency spillway.

6.1.3 Spillway Rating

Ash Pond C

The Stantec hydrologic and hydraulic analysis report provides ratings for two spillways (See Appendix A – Doc 11). The spillways are:

- Overflow spillway from Ash Pond C to Settling Pond rating of 600 cubic feet per second (cfs)
- Overflow spillway from Settling Pond to plant water intake channel rating of 800 cfs

Gypsum Disposal Area

Spillway rating data for the Gypsum Disposal Facility impoundments were not provided to Dewberry for review.

6.1.4 Downstream Flood Analysis

As part of the dam safety hazard classification review, Stantec Consultants conducted a qualitative evaluation of the downstream impacts of a release from Ash Pond C due to a failure or misoperation of the embankments. The review evaluated a failure along the east section of Dike C adjacent to the Emory River and Watts Bar Reservoir. The minimum reservoir width at the location evaluated was about 800 feet. Prior to 2008 there were a

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number of residential structures along the river bank across from Ash Pond C and the adjacent dredge cell. In response to the 2008 failure of the dredge cell embankment, TVA purchased most of those homes. Stantec reports that the three remaining homes are located a minimum of about 17 feet above the normal pool of the lake and were not expected to be impacted by a release from Ash Pond C (See Appendix A – Doc 05).

6.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Supporting documentation reviewed by Dewberry is adequate.

6.3 ASSESSMENT OF HYDROLOGIC/HYDRAULIC SAFETY

Based on the technical data provided to Dewberry for review, the hydrologic and hydraulic safety of Ash Pond C and the Gypsum Disposal Facility impoundments are considered SATISFACTORY.

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7.0 STRUCTURAL STABILITY

7.1 SUPPORTING TECHNICAL DOCUMENTATION

7.1.1 Stability Analyses and Load Cases Analyzed

Ash Pond C

In 2009 TVA retained Stantec Consulting Services to perform a geotechnical exploration and slope stability analysis for Dike C (See Appendix A – Doc 01). Stantec performed field sampling, laboratory testing and static slope analyses for several sections along Dike C.

The stability of the slopes were evaluated under fully drained conditions (static, long-term, steady state seepage) using SLOPE/W software.

In the 2009 Stantec report, dike Factors of Safety for long term stability were less than the minimum required 1.5 (See Appendix A – Doc 1). In 2011 Stantec completed a buttress design report for structural modifications to the Ash Pond C dike. The 2011 report provided design drawings of the recommended modifications and resulting Factors of Safety (See Appendix A - Doc 16).

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 14). The seismic analysis included a single cross-section. Ground motion used in the analysis was based on a peak ground acceleration with 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 20).

Gypsum Disposal Facility

TVA retained Geosyntec Consultants to conduct a seismic analysis of the Gypsum Disposal Facility (See Appendix A – Doc 15). Ground motion used in the analysis was based on a peak ground acceleration with an exceedance probability of 2 percent in 50 years.

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During initial site development, evidence of karst features was observed. Due to the potential for sinkhole development in karst terrain, TVA revised the proposed operation of the gypsum stack to include dry disposal of waste materials, except for a portion of Phase I that will receive wet gypsum during the first year of operation. TVA retained Geosyntec to conduct additional slope stability analyses in conjunction with the revised design and operational plan (See Appendix A – Doc 21).

7.1.2 Design Parameters and Dam Materials

Ash Pond C

Documentation provided to Dewberry for review indicated the stability analyses for Dike C included 11 strata. The material properties used in the analyses are shown in Table 7.1. The long term, static analyses used drained soil shear strength parameters. The seismic loading analysis used undrained soil shear strength parameters.

Table 7.1: Summary of Soil Properties Used in Stability Analyses, Ash Pond C

Soil Strata	Static Analyses			Seismic Analysis		
	Unit Weight γ (pcf)	Cohesion c' (psf)	Friction Angle Ø'	Unit Weight γ (pcf)	Cohesion c (psf)	Friction Angle Ø
Riprap Buttress	115	0	38	110	0	38
Starter Clay Dike	129	0	30	129	300	26
Raised Clay Dike	125	0	30	125	65	23
Constructed Ash	93	0	30	93	0	30
Hydraulically Placed Ash	96	0	25	96	0	10
Gravel to Clayed Gravel	120	0	32			
Sensitive Silt/Clay	127	0	28	127	c/p = 32	N/A
Lean Clay Foundation Soil	129	0	30			
Sandy Silt to Silty Sand	105 to 113	0	27 to 29	105	0	27
Fine Grained Sand to Sand with Silt	118 to 128	0	31 to 36	122	0	31
Rock				110	0	38

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Gypsum Disposal Facility

Documentation provided to Dewberry for review indicated the static stability analyses included six strata (See Appendix A - Doc 21). The material properties used in the analyses are shown in Table 7.2

Table 7.2: Summary of Soil Properties Used in Stability Analyses, Gypsum Disposal Facility

Material	Unit Weight (pcf)	Effective Strength	
		Cohesion (psf)	Friction Angle (degrees)
Gypsum (Phase I)	100	0	30
Dry Fly Ash (Phase II)	100	0	30
Soil fill	117	0	30
Native Soil (N>4)[1]	120	0	34
Native Soil (N<4)[2]	105	0	25
Bedrock	155	10,000	30

1. N>4 refers to native soils exhibiting a Standard Penetration Resistance greater than 4 blow per foot.
2. N<4 refers to native exhibiting a Standard Penetration Resistance less than 4 blow per foot.

Documentation provided to Dewberry for review indicated the seismic stability analysis for the Gypsum Disposal Facility included six strata (See Appendix A – Doc 21). The material properties used in the analyses are shown in Table 7.3.

Table 7.3: Summary of Soil Properties Used in Seismic Analysis – Gypsum Disposal Facility

Strata	Unit Weight (pcf)	Shear Wave Velocity (ft/sec)
Landfill Cover	120	771
Fly Ash	100	600
Landfill Liner	125	984
Medium Stiff to Stiff Silty Clay	120	712
Soft Silty Clay	105	394
Crystalline Bedrock	155	2,500

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7.1.3 Uplift and/or Phreatic Surface Assumptions

Ash Pond C

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

Embankment pore pressures for the long-term, static loading condition were obtained from the seepage analysis conducted as part of the geotechnical explorations (See Appendix A – Doc 01). Phreatic surface assumptions were made using the information developed by Stantec using their SEEP/W analysis program and also water levels obtained from piezometer readings. The SEEP/W program is discussed in detail in the Stantec geotechnical report.

Gypsum Disposal Facility

The stability documentation provided to Dewberry did not specifically identify uplift forces acting on the base of the embankment. Based on the identification of potential for sink hole development, the redesign of the gypsum impoundment included an extensive interior subgrade drainage system.

7.1.4 Factors of Safety and Base Stresses

Ash Pond C

The minimum safety factors computed for Ash Pond C, Dike C are summarized in Table 7.4.

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Table 7.4: Summary of Slope Stability Factors of Safety Ash Pond C, Dike C

Location (Station No.)	Slope	Long Term Static Load		Rapid Draw Down Conditions	Seismic
		Minimum Required Safety Factor			
		1.5		1.3	1.0
		Initial Design	With Buttress	With Buttress	With Buttress
108+93	Starter Dike	1.42	1.80	1.93	
	Raised Dike	1.39	2.05		
	Deep Failure	1.66	1.93		
119+69	Starter Dike	1.18	1.50	1.72	
	Raised Dike	1.39	2.51		
	Deep Failure	1.47	1.68		
132+37	Starter Dike	1.24	1.76	1.75	1.0
	Raised Dike	1.22	2.25		
	Deep Failure	1.47	1.88		
138+27	Starter Dike	1.13	1.53	1.36	
	Raised Dike	1.38	2.29		
	Deep Failure	1.52	1.76		
149+14	Starter Dike	1.15		1.92	
	Raised Dike	1.35			
	Deep Failure	1.49	1.93		

The stability analyses for long term static conditions for both the initial design and with buttress slope configurations were reported to have been conducted using Geo-Slopes Slope/W software. The calculated Factors of Safety with buttressed slope are equal to or exceed the minimum required value of 1.5.

The rapid drawdown conditions were conducted using the UTEXAS4 software. The calculated Factors of Safety are equal to or exceed the minimum required value of 1.3.

The 2011 Stantec report (Appendix A, Document 14) also analyzed slope stability for undrained soil strength conditions. The analyses, conducted using UTEXAS4 software indicated Factors of Safety less than the required minimum value of 1.3 at Station 119+69 and 138+27. The report indicates the Factors of Safety at those locations were greater than 1.3 when the analyses were repeated using the Slope/W software. The report opines that the differences were due to "the inherent conservatisms in the UTEXAS4 model are influencing the model output". As construction of the buttresses was substantially complete at the time of Dewberry site

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visit, and no indication of slope stability issues were observed, the issue of short-term stability during construction is not considered a concern.

The seismic analysis indicated a minimum calculated safety factor of 1.0 at Station 132+37 along Dike C. The computed safety factor is equal to the minimum recommended value of 1.0 for seismic loading conditions.

Gypsum Disposal Facility

The minimum safety factors computed for the Gypsum Disposal Facility are summarized in Table 7.5.

Table 7.5: Summary of Static Slope Stability Factors of Safety, Gypsum Disposal Facility

Location	Cross-Section	Minimum Required Factor of Safety	Minimum Calculated Factor of Safety	
			Drained Shear Strength	Undrained Shear Strength
Phase 1	1-1	1.5	1.75	3.39
Phase 1	2-2	1.5	1.73	4.08
Phase 2	3-3	1.5	1.78	3.01
Phase 2	4-4	1.5	1.74	3.30
Phase 2	5-5	1.5	1.73	3.19

The seismic analysis indicated a minimum calculated safety factor of 1.02 for the Gypsum Disposal Facility gypsum stack cell. The computed safety factor meets the minimum recommended value of 1.0 for seismic loading conditions.

7.1.5 Liquefaction Potential

No assessment of liquefaction potential was performed for the Ash Pond C, Dike C. A qualitative assessment by Dewberry of materials within and under Dike C indicates there is a potential for liquefaction of CCR and soils under seismic conditions (See Appendix A - Doc 22). We recommend that TVA proceed with conducting a quantitative liquefaction analysis of the impoundment to better determine the risk of a major release caused by a seismic event.

No assessment of liquefaction potential was performed for the Gypsum Disposal Facility. A qualitative assessment by Dewberry of material with

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and under the impoundment dikes did not identify a potential for liquefaction (See Appendix A - Doc 22).

7.1.6 Critical Geological Conditions

The Kingston Fossil Plant is located in the Valley and Ridge Physiographic province near the base of the Columbia Plateau. Numerous northeast to southwest trending faults have been mapped in the Valley and Ridge province. The Kingston Fossil Plant is mapped as being located between the Chattanooga Fault to the north and the Kingston Fault to the south.

The majority of the plant is underlain by the Conassauga Shale Formation consisting of clayey to silty shale with zones of shaley limestone.

A narrow band of Maynardville Limestone separates the main plant area from the Gypsum Disposal Facility peninsula. USGS maps show enclosed drainage basins which can be an indication of karst activity (i.e., sinkhole development).

The USGS National Seismic Risk Map for the Central and Eastern United States estimates the peak ground acceleration having a 2-percent probability of exceedance in 50 years of 0.2g.

7.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Structural stability documentation of Ash Pond C and the Gypsum Disposal Facility is adequate. Liquefaction documentation is inadequate.

7.3 ASSESSMENT OF STRUCTURAL STABILITY

Overall the structural stability of the Ash Pond C appears to be **FAIR** based on the following observations:

- Implementation of recommendations made by Stantec to increase the slope stability of Ash Pond C, Dike C by construction of a riprap buttress for each area has been substantially completed.
- Liquefaction analyses of soils and materials with Ash Pond C and the Gypsum Disposal Facility have not been performed. A qualitative analysis indicated that Ash Pond C may be susceptible to liquefaction. The need for a quantitative liquefaction analysis of Ash Pond C is critical.

The Gypsum Disposal Facility is **SATISFACTORY**.

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8.0 ADEQUACY OF MAINTENANCE AND METHODS OF OPERATION

8.1 OPERATING PROCEDURES

The operations and maintenance procedures for the Kingston CCR impoundments are defined in the *Tennessee Valley Authority (TVA) Routine Handling Operations and Maintenance (RHO&M) Operations Support Document, Kingston Fossil Plant*, July, 2011 (See Appendix A – Doc 04).

Ash Pond C

Ash Pond C is divided by an interior dike into two sections: the ash pond on the north side of the impoundment, and the stilling pond to the south. Ash from the plant is sluiced to and deposited into the northwest corner of the ash pond. Sluice water flows in a bottom ash drainage channel to the eastern portion where an overflow spillway connects to the stilling pond.

Ash from the drainage ditch is excavated and stacked in the ash pond.

The stilling pond discharges to the plant water intake channel through a newly installed siphon system, and an overflow spillway structure.

Gypsum Disposal Facility

The original design for the Gypsum Disposal Facility was to sluice gypsum to a cell in the ash stack impoundment. The receiving cell would include a settling pond. Water from the settling pond was to be pumped to the stormwater pond and from there pumped to the plant's intake channel. A cell adjacent to the receiving cell would be used to store gypsum excavated from the receiving cell.

Due to sinkhole development encountered during final construction of Phase 1, the facility is currently out of service pending completion of the redesigned facility.

8.2 MAINTENANCE OF THE DAM AND PROJECT FACILITIES

TVA has implemented a formal operations and maintenance program for Kingston Fossil Plant Ash Pond C and the Gypsum Disposal Facility when it returns to service. (See Appendix A – Doc 04). Specifically, a detailed program regarding coal combustion residue streams, coal combustion residue handling and disposal, compliance with government regulations, and roles and responsibilities is in place.

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The 2010 annual inspection report provided to Dewberry for review identified potential embankment seepage, and tree growth on some embankments as the most significant maintenance issue (See Appendix A – Doc 07).

In response to the recommendations, TVA retained Stantec to prepare a maintenance plan addressing the seepage (See Appendix A – Doc 17)

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 Kingston Fossil Plant CCR management include written reports of daily, weekly, monthly, quarterly and annual inspections. Special inspections are required after significant storms; i.e. rain events exceeding the 10-year recurrence intensity, and after a significant earthquake (See Appendix A – Doc 04).

The 2010 annual inspection report was provided to Dewberry for review (See Appendix A – Doc.07).

9.2 INSTRUMENTATION MONITORING

Ash Pond C

Dike C impounding Ash Pond C is instrumented with piezometers and slope inclinometers. The instrumentation points are monitored weekly. Dewberry was provided copies of the weekly reports of recent piezometer and slope inclinometer readings (See Appendix A – Docs 18 and 19, respectively). No significant changes in ground water elevations or indications of slope movement were reported.

Gypsum Disposal Facility

Piezometers are located along the toe of the south embankment of the ash stack impoundment. As the ash stack portion of the facility is out of service, the piezometers were not being monitored at the time of Dewberry's site visit.

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 program is adequate.

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APPENDIX A

Document 1

Report of Geotechnical Exploration and Slope Stability for Dike C, Kingston Fossil Plant, Harriman, Tennessee, Stantec Consulting Services, August 3, 2009

Report of Geotechnical Exploration and Slope Stability for Dike C

Kingston Fossil Plant
Harriman, Tennessee

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Tennessee Valley Authority
Chattanooga, Tennessee

August 3, 2009

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August 3, 2009

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Mr. Barry Snider
Tennessee Valley Authority
1101 Market Street
LP 2N
Chattanooga, Tennessee 37402

Re: Report of Geotechnical Exploration and Slope Stability for Dike C
Kingston Fossil Plant
Harriman, Tennessee

Dear Mr. Snider:

As requested, Stantec Consulting Services Inc. (Stantec) has completed our Report of Geotechnical Exploration and Slope Stability Evaluation for Dike C at the Kingston Fossil Plant. The report documents the subsurface conditions, results of laboratory testing, findings from the historical document reviews, results of our analyses and evaluation, and recommendations for the facility. These services were performed under Engineering Service Request ESR/TAO 650 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.

Adam A. Crace, PE
Senior Project Engineer

Don W. Fuller, PE
Principal

/cmw



Report of Geotechnical Exploration and Slope Stability for Dike C

Kingston Fossil Plant
Harriman, Tennessee

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

August 3, 2009

Report of Geotechnical Exploration and Slope Stability for Dike C
Kingston Fossil Plant
Harriman, Tennessee

Table of Contents

Section		Page No.
Executive Summary		**iv**
1.	**Introduction**	**1**
2.	**General Site Description and Geology**	**1**
	2.1. Location and Description	1
	2.2. Geology	3
3.	**Review of Available Information**	**4**
	3.1. General	4
	3.2. Site History	4
4.	**Scope of Exploration**	**5**
5.	**Results of Geotechnical Exploration**	**7**
	5.1. Summary of Borings	7
	5.2. Subsurface Soil Conditions	11
	5.3. Instrumentation	12
	5.3.1. Piezometers	12
	5.3.2. Slope Inclinometers	14
6.	**Laboratory Testing**	**14**
	6.1. General	14
	6.2. Testing of Cohesive Soils/Undisturbed (Shelby) Tube Testing	14
	6.2.1. Unconfined Compressive (UC) Strength Testing	15
	6.2.2. Consolidated-Undrained (CU) Triaxial Testing	15
	6.2.3. Direct Simple Shear (DSS) Testing	16
	6.2.4. Unconsolidated – Undrained (UU) Triaxial Testing	16
	6.2.5. One-Dimensional Consolidation Testing	17
	6.3. Testing of Standard Penetration Test (SPT) Samples	18
7.	**Engineering Analyses**	**18**
	7.1. General	18
	7.2. Soil Horizons	19
	7.3. Seepage Analysis	20
	7.3.1. SEEP/W Model	20
	7.3.2. Boundary Conditions	20
	7.3.3. Seepage Properties	21
	7.3.4. Comparison to Field Observations	22
	7.3.5. Results from Seepage Analyses	24

Table of Contents
(Continued)

Section **Page No.**

7.3.6. Critical Exit Gradients 25
7.4. Strength Parameter Selection 25
7.4.1. Drained Soil Parameters 26
7.4.2. Soil Parameters for Dike C 27
7.5. Slope Stability Analysis 30
7.5.1. Limit Equilibrium Methods in SLOPE/W 30
7.5.2. Slope Stability of Dike C 30

8. Results 31
8.1. Seepage Gradients 31
8.2. Slope Stability Results 32
8.2.1. Conceptual Mitigation Evaluation 34

9. Limitations of Study 34

10. Conclusions 35

11. Recommendations 36

12. Closure 37

13. References 38

List of Tables

Table **Page No.**

Table 1. Summary of Borings 8
Table 2. Summary of Piezometers 13
Table 3. Summary of Unconfined Compressive Strength Tests 15
Table 4. Summary of Consolidated – Undrained Triaxial Testing 15
Table 5. Summary of Direct Simple Shear Testing 16
Table 6. Summary of Unconsolidated – Undrained Triaxial Testing 17
Table 7. Summary of One Dimensional Consolidation Tests 18
Table 8. Material Properties for SEEP/W Analysis 21
Table 9. Selected Strength Parameters for Stability Analysis 29
Table 10. Summary of Computed Exit Gradients and Factors of Safety against Piping 32
Table 11. Summary of Computed Factors of Safety for Slope Stability 33

Table of Contents
(Continued)
List of Figures

Figure		Page No.
Figure 1.	Kingston Fossil Plant Overview	2
Figure 2.	Comparison between the Field Piezometer Readings and Pore Water Pressures predicted in the SEEP/W Model	23
Figure 3.	Comparison between the borehole water levels and the phreatic surface predicted in the SEEP/W Model	24
Figure 4.	Charts Used to Correlate $(N_1)_{60}$ to ϕ'	29

List of Appendixes

Appendix	
Appendix A	Typed Boring Logs
Appendix B	CPT Boring Plots
Appendix C	Instrumentation Installation Details
Appendix D	Instrumentation Results
Appendix E	Laboratory Testing
Appendix F	Boring Layout, Stability Sections, Profile Sheets and Logs of Borings
Appendix G	Strength Parameter Selection Charts
Appendix H	Pore Water Pressure Contours

Executive Summary

The purpose of this study was to evaluate the current stability of Dike C, the perimeter containment dike around the ash pond and stilling pond at Tennessee Valley Authority's Kingston Fossil Plant. Dike C, which includes the Road Dike, is located adjacent to the failed ash dredge cell and is underlain by similar geologic deposits. TVA is in the process of converting the Kingston plant systems to dry handling of fly ash. This will significantly reduce the fly ash combustion product storage role for the ash pond. It is anticipated that the Dike C structure and pond basin configuration will be modified in association with the conversion and reduced storage needs. The assessment of Dike C and the associated recommendations are based on this understanding of the plant setting.

The existing condition of Dike C was investigated in a subsurface exploration and laboratory testing program, and then assessed for geotechnical stability under static, long-term, steady-state conditions. Seepage and slope stability were evaluated using engineering analyses to quantify factors of safety.

The investigation began with a review of available geologic and historical project information provided by TVA. The site is underlain by alluvial and lacustrine deposits of sands, silts, and clays, with a weathered shale bedrock beneath. Dike C consists of two primary stages, with the lower, initial starter dike constructed in the 1950s to a crest elevation of about 748 ft. The dike crest was raised to an elevation of 765 in the 1970s, with the construction of the upper (clay) dike built over existing ash deposits using the upstream method of construction.

To collect additional subsurface data needed to support updated analyses, a field investigation program was undertaken in the spring of 2009. In total, 54 soil borings and 21 cone penetration tests were advanced at locations along the length of Dike C. Most of these test penetrations were advanced from the crest of the dike levee, or from a bench on the lower downstream face (crest of the original starter dike). Nine borings were completed through interior divider dikes within the ash pond. Instrumentation, consisting of slope inclinometers and piezometers, were installed in several of these borings to monitor conditions within the dike. Samples obtained from the site boring program were tested in the laboratory to establish key index properties, as well as permeability and strength parameters.

The data collected for the Dike C investigation was supplemented with subsurface information obtained by AECOM for the Root Cause Analysis of the dredge cell failure. Logs from AECOM borings (23 locations) in the ash pond, plus laboratory test data, provided additional information on the ash and soil materials within Dike C. AECOM's conclusions regarding the mechanisms that contributed to the dredge cell failure were also considered in the investigation and analysis of Dike C.

The engineering analyses focused on five cross sections through Dike C. Spaced at about 600 to 1200 ft apart, these cross sections were selected to represent the typical conditions in reaches of the perimeter dike. 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 information on the site development. The 2008 failure of the adjacent dredge cell deposited coal ash against the outslope of Dike C, mostly along the eastern reach. This accumulation of ash, which is primarily fluid in nature and highly erodible and will be removed during the cleanup, was ignored in the engineering analyses.

Eight soil horizons within and beneath Dike C were identified and described. Key properties, including unit weight, saturated hydraulic conductivity, horizontal to vertical permeability ratio, and drained shear strength parameters, were estimated for each soil horizon. These estimates were developed from the available laboratory data. Where needed, some soil parameters were estimated or adjusted based on typical values for similar soils. The soil parameter values selected for use in the stability analyses are tabulated and described in the report.

To understand the seepage conditions within the dike, a finite element seepage model was developed for each of the five cross sections through Dike C. The seepage model was based on the previously defined cross sectional geometry and the estimated hydraulic properties of the principal soil horizons. The applied boundary conditions represented the water levels in the ash pond or stilling pond, depending on the location, and the normal winter pool elevation of Watts Bar Lake. Steady-state seepage conditions were assumed, and a solution was obtained for the total hydraulic head at nodal points throughout the section. The seepage model was iteratively "calibrated" to match the existing field conditions. This was accomplished by varying the estimated hydraulic soil properties until the head at corresponding locations were in reasonable agreement with water levels measured in piezometers installed in the dike. Graphical results from the seepage analyses are presented in Appendix H of the report. The hydraulic pressures predicted with the seepage model were then mapped across the modeled sections to give the pore water pressures needed for the subsequent slope stability analyses.

The results from the seepage analyses were also 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) within the starter dike; these results are generally confirmed by the observation of shallow water in areas where trees and stumps have been removed from the starter dike. On four of the five cross sections, the model indicated seepage flows to the sloping downstream face of the starter dike. This condition creates the potential for the initiation of soil piping, as seepage water will tend to erode material from the dike face at these locations.

Upward, vertical exit gradients in the area of the dike toe were also evaluated. Factors of safety against piping, computed for the surficial 3 to 5 feet of soil in these areas, ranged from 1.3 to 2.7. Based on USACE design criteria for dams (EM 110-2-1901), target minimum factor of safety against piping is three. The results from the seepage model indicate that Dike C does not meet current criteria for soil piping due to seepage.

The stability of the Dike C slopes was evaluated using conventional, two-dimensional, limit equilibrium methods. Factors of safety for slope stability were computed using Spencer's method of analysis, circular and noncircular slip surfaces, and search routines that helped to identify critical (low safety factor) sliding mechanisms. Using pore water pressures predicted with the seepage model, the resistance to sliding was quantified using effective stresses and shear strength parameters determined on the basis of laboratory testing.

This analysis is limited to static, long-term, fully drained conditions within the existing dike. Dike C has existed in its current cross sectional geometry (slopes and crest elevation) for at least 30 years. Excess pore water pressures generated in the underlying soil during construction have had sufficient time to dissipate, and steady state seepage conditions have developed within the dike. Hence, for the current static conditions, the soils can be treated as

fully drained and the stability can be assessed using effective stress analyses. If stabilizing berms or other modifications to the dike cross section are built, then undrained, total stress stability analyses will be needed to assess stability during construction.

The slope stability calculations produced factors of safety against sliding along various potential failure mechanisms. In general, each cross section was evaluated for potential deep-seated slides that would impact the dike crest, plus critical slip surfaces that corresponded to minimum factors of safety. The potential for upstream sliding, into the ash pond or stilling pond, was also evaluated. The results of the stability analyses indicate global dike factors of safety ranging from 1.47 to 1.66. Current USACE criteria for the long-term stability of Dike C require a factor of safety for slope stability of at least 1.5. Considering only potential deep-seated or global failure mechanisms that would immediately impact the crest, the slope stability results show that Dike C meets this criteria. The difference between the minimum computed value of 1.47 and the required value of 1.5 is negligible, considering the inherent uncertainty and localized variability in the characterization of the soil strength. These results suggest that Dike C is stable with respect to the potential formation of large, deep-seated failures that would immediately compromise the retention of the ash pond.

The analysis also indicates that the controlling factors of safety are associated with smaller maintenance type sliding masses about 10 ft in thickness on the downstream face of the starter dike or the upper raised dike section depending on specific geometry present at that section. The projected potential for these shallow slides or slip surfaces occurs on the downstream face along the full length of Dike C (plus the upstream face in the vicinity of Sta. 132+37). The potential shallow slip surfaces occur in the slopes of both the lower starter dike and the upper raised dike (see the cross sections in Appendix F). The lowest factors of safety are currently found along the eastern branch of Dike C, where factors of safety less than 1.2 were computed. As directed by TVA, remedial design work plans are now underway to increase the long-term factors of safety against shallow sliding to achieve the target minimum value. The mitigation plan will incorporate specific interim risk reduction strategies and include both enhanced geotechnical instrumentation and construction of a dike embankment buttress and seepage control system.

Past shallow sliding of the downstream face of Dike C was reported in the project records, in an area along the perimeter of the dredge cell. This surficial slide was observed and repaired by TVA as part of their routine maintenance program. If additional shallow sliding were to develop again in Dike C, it is anticipated the slip surface would be initially confined to the sloping face of the dike. If not repaired and given enough time, these shallow slides could progress up the slope and endanger the ash pond. Because this progressive failure mechanism would be expected to take months or longer to impact the crest, a robust monitoring program can reduce the risk of dike breeching due to shallow sliding. Hence, in the interim while the permanent mitigation design and plant dry ash conversion process is developed, analyzed, and constructed, TVA should continue to monitor Dike C with routine inspections and instrument readings.

In conclusion, the current configuration of Dike C around the Kingston ponds 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.

Report of Geotechnical Exploration and Slope Stability for Dike C
Kingston Fossil Plant
Harriman, Tennessee

1. Introduction

In December, 2008, the Tennessee Valley Authority (TVA) requested that Stantec Consulting Services Inc. (Stantec) develop a geotechnical exploration plan to perform initial characterization and evaluation of Dike C. Dike C (also includes the Road Dike) is approximately 5,600 feet in length and provides perimeter containment for the ash pond and stilling pond. It is Stantec's understanding that TVA is in the process of converting Kingston Plant Systems to the dry handling of fly ash. This will significantly reduce fly ash combustion storage role for the ash pond. It is anticipated that the Dike C structure and pond basin will be modified in association with the conversion process. The assessment of Dike C and the recommendations reflect these planned operations for the pond. Stantec's scope of work associated with this assignment is defined in the document titled "*Revised Addendum to Work Plan for ESR/TAO Request 650*", dated June 17, 2009, submitted to TVA.

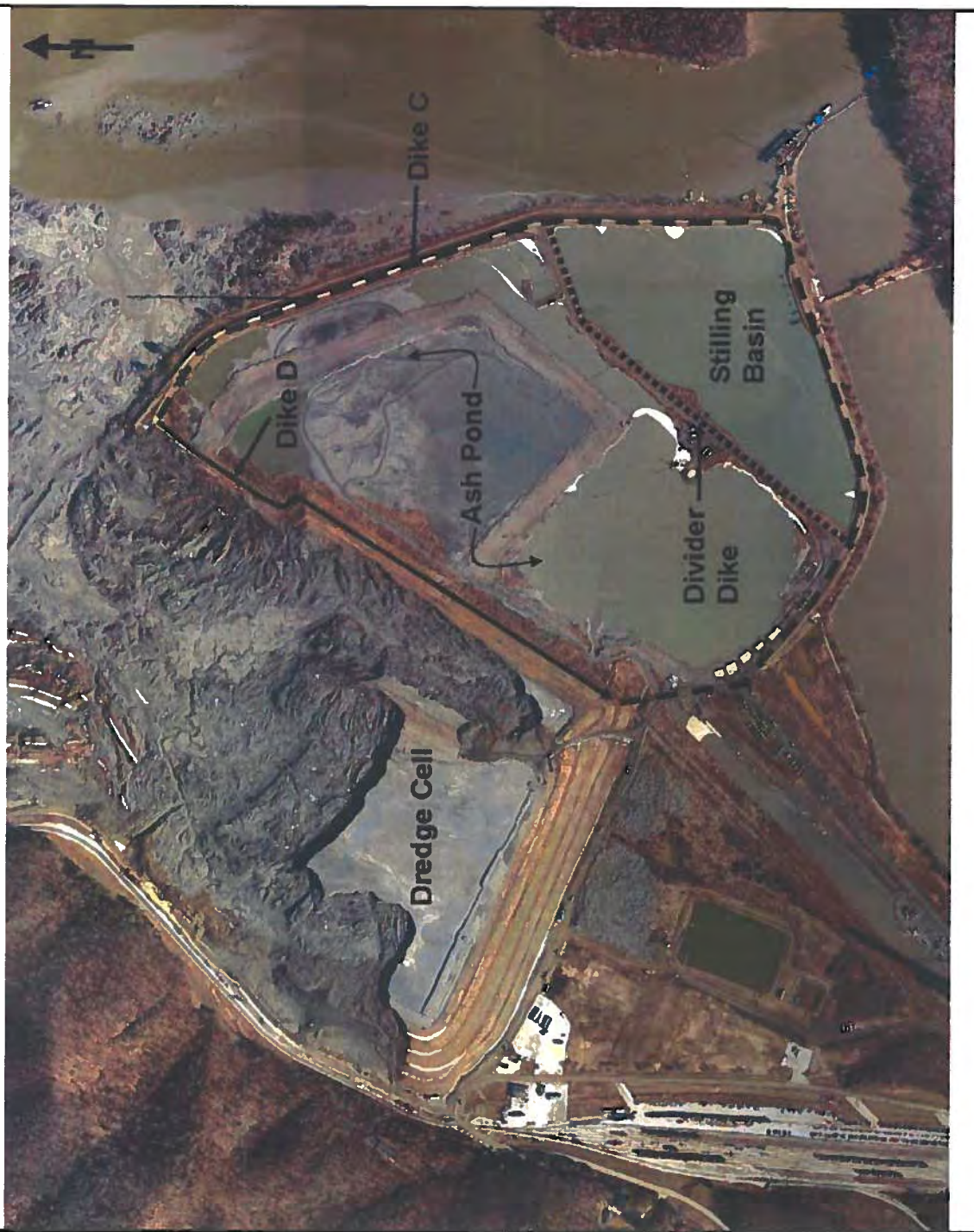
The results of initial geotechnical characterization are intended to supplement the historical data reviewed as a part of TVA's fossil group Facility Assessment Phase 1 program. This report presents the conclusions and recommendations of Stantec's evaluation along with general site geology and results of geotechnical exploration, laboratory testing and engineering analysis.

2. General Site Description and Geology

2.1. Location and Description

The Kingston Fossil Plant is located on the Watts Bar Reservoir, at the confluence of the Emory and Clinch Rivers, near Kingston, Tennessee approximately 35 miles southwest of Knoxville, Tennessee. The ash disposal area is centered approximately 4000 feet northeast of the plant's powerhouse. The disposal area (Figure 1) is bordered by the Emory River on the east and southeast with the excavated channel carrying intake condenser water present immediately south of the disposal area.

The ash pond and settling pond have a total surface area of approximately 120 acres and are enclosed to the south, east and north by Dike C, which is approximately 5,600 feet in length. The ash pond is enclosed to the west by Dike D. The top of the dike supports a gravel access road that is at approximate elevation 765 feet, which is approximately 28 feet above the winter pool elevation (737 feet) of Watts Bar Lake. The elevation of the crest from the starter dike crest is approximately 750 feet. Dike C, including the starter dike and the raised dike, have an overall constructed height that varies from 20 feet to 30 feet. The slope



Kingston Fossil Plant
Harriman, Tennessee

Stantec

Figure 1. Kingston Fossil Plant Overview

of the raised dike interior slope varies from 1.7H:1V to 2.8H:1V and the exterior slope varies from 1.5H:1V to 3H:1V. Based on historical documentation and some limited field observations it is estimated that the starter dike was constructed with 3H:1V interior slopes and 6H:1V exterior slopes. The slopes are vegetated with grass turf, with areas of mature trees at various locations around the perimeter near the pool level for Watts Bar Lake. Recent facility maintenance activities include removal of trees and replacement of rip rap armor within the vicinity of the Emory River/Watts Barr Lake water line between approximate Station 128+00 and Station 136+00.

2.2. Geology

The plant is situated on the western edge of the Valley and Ridge Physiographic province near the base of the Cumberland Plateau. Numerous faults, trending northeast to southwest as characteristic of the Valley and Ridge Province, are depicted on the geologic mapping. The plant is positioned between the Chattanooga Fault to the north, and the Kingston Fault to the south.

According to the USGS Geologic Map of the Harriman Quadrangle (1993), the plant is underlain by Lower Ordovician and Cambrian age limestone and shale bedrock formations. Although not depicted on the geologic mapping, previous drilling programs at the plant indicate that alluvial deposits consisting of fine sands, silty sands, and sandy silts are present at the site, as are commonly found adjacent to rivers.

The majority of the plant, including the majority of the ash disposal area, is underlain by the Conassauga Shale Formation, which consists of an argillaceous to silty shale with zones of shaley limestone scattered throughout. The geologic mapping indicates this formation is contorted, brecciated, and sheared throughout. The geologic mapping indicates Pine Ridge, just north of the plant, and the northern corner of the ash disposal area are underlain by the Rome Formation, consisting of arenaceous shale interbedded with very thin siltstone layers and thin beds of sandstone.

A narrow band of the Maynardville Limestone separates the mainland plant from the peninsula and does not underlie Dike C. The Maynardville Limestone Formation consists of fine grained, laminated to thinly bedded dolomite in the upper part and fine grained, thin to very thick bedded limestone with ribboned shale seams and dolomite laminate in the lower part. The peninsula is underlain by the Knox Group, consisting of siliceous dolomite with a few limestone beds in the upper part and scattered thin quartz sandstone beds and lenses. The Knox Group generally weathers to a thick residuum of red-orange soil with chert fragments and is known for karst activity. The USGS topographic map depicts a few enclosed drainage basins indicative of karst activity within the peninsula area.

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 Dike C, Ash Pond, and the Stilling Pond. The main objective of the document review was to develop a historical knowledge base prior to beginning the field geotechnical exploration. The documents reviewed included record drawings, cross sections of dikes, aerial photographs, old contour maps, and annual dike stability reports. A complete listing of the reviewed documents is included in the Phase 1 report.

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

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

- "Kingston Fossil Plant Annual Ash Pond Dike Stability Inspection" TVA Engineering Design Services, 1967 to 2009

These studies included boring plans, driller's logs, and results from laboratory tests. The information gained from these reports was evaluated and used to supplement the information gathered from Stantec's geotechnical exploration. Logs from AECOM borings (23 locations) in the ash pond, plus laboratory test data, provided additional information on the ash and soil materials within Dike C. AECOM's conclusions regarding the failure mechanisms that contributed to the dredge cell failure were also considered in the investigation and analyses of Dike C.

3.2. Site History

Kingston Fossil Plant began construction in 1951 and was on-line during February 1954. At that time, ash slurry was discharged directly to slack water area created by Watts Bar Reservoir (Reservoir), which was filled in 1941. The August 1951 TVA design drawings of the ash pond show a gap between the East and North Dikes that formed the initial ash pond storage area and allowed ash to mix with waters of the Reservoir. TVA reports that by 1958 the northern 275-acre ash pond containment dike was completed with the construction of Dike C, which was constructed of residual clay and bottom ash to an elevation of 748 feet. The ash pond containment dike was comprised of a southern leg, which is referred to as the Road Dike, and a north to northeastern leg which is comprised of a portion of Dike C. This initial structure is referenced herein as the "starter dike".

After the initial ash disposal cell bounded by the North and East Dikes was filled in 1965 ash was directed into the main ash disposal cell where it deposited and progressively filled from south to north. From 1958 to 1977, water in the main ash disposal cell exited through a duel Dike C riser pipe system at the north end of the ash pond where it then reentered the Reservoir. In 1974, Dike C and the Road Dike underwent an interim raise of approximately 1.5 feet to provide additional freeboard for the ash pond and a deflector dike was created in the southwestern portion of the ash pond (near the current location of Dredge Cell 1). Between 1976 and 1978, major construction and operational changes took place with a major increase in the height of Dike C and the Road Dike and the creation of the stilling

pond. Dike C and the Road Dike were both raised, using upstream construction methodology, to an elevation of 765 feet to provide the additional freeboard when the stilling pond was created to allow the finer particles to settle out before the water reentered Watts Bar Lake. This upstream construction component of the dike is referenced herein as the "raised dike". The stilling pond was created in the southeastern most portion of the ash pond when an all ash divider dike was constructed in a northeastern to southwestern direction between Dike C and the Road Dike. This divider dike would allow the stilling pond to operate at a pool elevation of 754 to 755 feet while the main ash collection pond would be operating at elevation 760 to 761 feet.

As plant operations continued the ash pond transitioned into smaller subdivided cells formed with interior dikes. In 1983 an interior ash dike was constructed in a southwest to northeast direction that roughly reduced the size of the ash pond by half and is most likely the first effort in the planned transition to Dredge Cell operations. In 1984 TVA reported a failure of the interior ash dike associated with dredge maintenance undermining; this event did not result in any offsite releases, but it did establish the approximate location of future Dike D. The 1985 annual inspection report recognizes the limits of a contained "dredge area" along the western side of the pond designated as Dredge Cell 1. Between 1987 and 1988 Dredge Cells 2 and 3 were created in the northwestern portion of the ash pond. As the Dredge Cells began to fill an Intermediate Dredge Cell was created to the east of Dredge Cell Nos. 1, 2, and 3 and the eastern dike became what we now know as Dike D.

Prior to the dredge cell failure in 2008, the Kingston Fossil Plant operated by sluicing fly ash and bottom ash into a channel that drains into the ash pond. The bottom ash was excavated out of the sluice channels and the ash pond and then transported to the dredge cells. Sluice water from the ash pond flows into five 48-inch reinforced concrete pipe (RCP) riser pipe/weirs that discharge into the stilling basin. Once the particulates have settled out, water from the stilling basin flows into six 48-inch RCP pipe/weirs that discharge into the Emory River intake channel. Approximately 390,000 dry tons of fly ash and 95,000 dry tons of bottom ash is sluiced to the ash pond annually.

4. Scope of Exploration

In response to TVA Engineering Service Request (ESR) 650, Stantec submitted a work plan on February 18, 2009. TVA ESR 650 includes the Dike D buttress construction and support (submitted under a separate cover) and the ash pond stability (addressed under this cover). As the project continued and the scopes were better defined, Stantec submitted an Addendum to the original work plan on April 21, 2009. Finally, as the field exploration was completed and Stantec began to work through some preliminary slope stability analyses, and a Revised Addendum was submitted on June 17, 2009. The Revised Addendum included a slightly reduced scope that was agreed to by TVA and Stantec personnel and will be discussed in the following paragraphs. The reduced scope was a result of the information gained during the field exploration and the preliminary slope stability analyses.

Stantec personnel advanced 54 conventional sample borings using a combination of all terrain vehicle and truck-mounted drill rigs on approximately 200-foot intervals along the centerline of the starter dike, raised dike and select other locations (designated herein as STN-1 through STN-71) from March 17, 2009 to May 27, 2009. Selected borings were also advanced approximately 20 feet into bedrock using NQ-size (approximately two-inch diameter) rock coring equipment. The borings were advanced into bedrock to determine

general engineering characteristics of the uppermost geology. In addition, Stantec assessed the hydraulic conductivity properties of the encountered bedrock utilizing water pressure testing. In response to information provided by AECOM relative to the Root Cause Analysis (RCA), Stantec conducted eight borings with continuous Shelby tube sampling at selected intervals near the original ground line at four discrete locations along the perimeter dike. In addition, Stantec advanced 21 cone penetration test (CPT) borings.

Stantec's original boring plan submitted to TVA on April 21, 2009, identified 71 conventional sample borings that were to be advanced. Based on the results from the initial field exploration, review of TVA's hydrogeographic data and the preliminary slope stability analyses, Stantec removed 17 borings from the original program. STN-1, STN-7, STN-13, STN-17, STN-25, STN-30, STN-35, STN-40, and STN-46 were removed from the program because the preliminary slope stability analyses concluded that the critical failure mechanism was not anticipated to change based on the results of advancing those borings within Watts Bar Lake. STN-33, STN-39, STN-57, STN-58, STN-67, STN-69, and STN-70 were removed from the program because fairly uniform subsurface conditions were encountered in the nearby borings. STN-44 was removed from the program because it was scheduled to be advanced near a previously completed AECOM boring. At the conclusion of the field exploration, TVA personnel surveyed as-drilled boring locations and transmitted the information to Stantec.

The subsurface exploration was performed using 3¼ and 4¼ inch (ID) hollow stem augers equipped with a carbide-tipped tooth bit. Standard Penetration Testing (SPT) or undisturbed (Shelby) tube samples were performed in all 54 of the conventional sample borings at continuous intervals. A standard penetration test consists of dropping a 140-pound hammer to drive a split-spoon sampler 18 inches. The consistency or relative density of soil is estimated by the number of blows it takes to drive the spoon the last 12 inches. This method is typically used to obtain soil samples, estimate the consistency or relative density of the soil, and also to estimate the vertical limits of the subsurface soil horizons. In addition, undisturbed samples (Shelby Tubes) were obtained with a fixed head piston sampler from selected depth intervals within the cohesive materials to provide samples for subsequent laboratory strength testing. After completion of the drilling and sampling procedures, the boreholes were checked for subsurface water and backfilled with bentonite grout.

CPT borings were conducted at offset locations to borings as shown in the list of borings in Section 5. Cone penetration testing was performed by advancing an integrated electronic seismic piezo cone within the soil-like overburden materials to measure tip resistance, sleeve friction and dynamic pore pressure at roughly one-inch intervals. In addition, pore pressure dissipation testing was performed at selected intervals.

Based on encountered site conditions, Stantec installed six slope inclinometers as a part of the overall stability evaluation. The slope inclinometers were constructed by advancing the boring approximately 10 feet into bedrock, installing 2.75 inch slope inclinometer casing, and backfilling the annulus with a bentonite-cement grout that is mixed to a similar consistency as the surrounding soils. Flush-mounted or riser type protective covers were set in concrete to protect the slope inclinometers. These instruments are currently scheduled to be monitored once every week until mid-November 2009.

Stantec installed 20 piezometers within the starter dike and the raised dike as a part of the overall stability evaluation, to provide data on piezometric levels within the existing dikes. Piezometer construction consisted of one-inch diameter Schedule 40 PVC well screen (five

feet) and riser pipe. The annular backfill consisted of a sand filter pack to some distance above the screened interval followed by a minimum two-foot bentonite seal. After allowing the bentonite to hydrate, the remaining annulus was backfilled with bentonite grout tremmied into place. Flush-mounted or riser type protective covers were set in concrete to protect the piezometers. These instruments are currently scheduled to be monitored once every two weeks until mid-November 2009.

An engineer/geologist was present with each drill crew throughout the drilling operations. The engineer/geologist directed the drill crews, logged the subsurface materials encountered during the exploration and collected soil samples. Particular attention was given to the soil's color, texture, moisture content and consistency or relative density. The soil samples extracted from the borings were transported to Stantec laboratories for testing.

In the laboratory, SPT samples were subjected to natural moisture content determination in accordance with American Society of Testing and Materials (ASTM) D 2216 and No. 200 wash gradation (ASTM D 1140). Selected SPT samples were also combined and subjected to soil classification tests that included Atterberg limits testing (ASTM D 4318), specific gravity tests (ASTM D 854) and sieve and hydrometer analyses (ASTM D 422). Undisturbed samples were extruded and subjected to unit weight determination, unconfined compression (ASTM D 2166), unconsolidated undrained triaxial compression (ASTM D 2850), consolidated undrained triaxial compression with pore pressure measurements (ASTM D 4767), one-dimensional consolidation (ASTM D 2435), direct simple shear (ASTM D 2435) and permeability (ASTM D 5084) testing.

The results of the field and laboratory testing services were used to develop critical stability sections perpendicular to Dike C. As a part of the original scope that was submitted to TVA on April 21, 2009, Stantec projected 10 stability sections that were located approximately 500 feet apart along the perimeter of Dike C. Based on the results of the field exploration, cross-section geometry and the preliminary slope stability analyses Stantec reduced the total number of stability sections to five under the revised addendum that was submitted on June 17, 2009. Stantec performed seepage and slope stability analyses on five cross sections using as detailed later in the report.

5. Results of Geotechnical Exploration

5.1. Summary of Borings

Stantec developed a boring plan for the field exploration of Dike C at the Kingston Fossil Plant after a review of historical information and existing site conditions. TVA survey personnel established the boring locations and surface elevations in the field in accordance with Stantec's boring plan. A summary of the boring information is presented in Table 1, where all measurements are expressed in feet. Typed boring logs with the results of the water pressure testing are presented in Appendix A and the CPT results are included in Appendix B.

Table 1. Summary of Borings

Boring No.	Northing	Easting	Surface Elevation	Top of Rock Elevation	Refusal/ Begin Core Elevation	Boring Termination Depth	Bottom of Hole Elevation
STN-1	Not Drilled						
STN-2	556804.57	2442329.32	751.2	713.7	710.2	41.0	710.2
STN-2A	556806.57	2442329.32	751.2	--	NR(721.7)	29.5	721.7
STN-2B*	556801.29	2442323.04	751.0	--	--		
STN-3	556756.78	2442263.59	763.7	714.7	713.4	50.3	713.4
STN-3A	556761.99	2442259.75	763.9	--	NR(733.9)	30.0	733.9
STN-3B	556765.05	2442256.36	763.8	--	NR(713.8)	50.0	713.8
STN-3C	556763.50	2442257.42	763.8	--	NR(728.8)	35.0	728.8
STN-3D	556762.50	2442258.42	763.8	--	NR(728.8)	35.0	728.8
STN-3E*	556764.10	2442559.45	763.8	--	--		
STN-4**	556625.56	2442047.01	763.3	702.3	700.3	83.0	680.3
STN-5	556611.15	2442365.31	764.9	712.4	710.9	54.0	710.9
STN-6	556416.17	2442407.10	763.4	699.4	698.9	64.5	698.9
STN-7	Not Drilled						
STN-8**	556248.52	2442540.30	752.2	698.7	696.4	76.7	675.5
STN-8A*	556254.96	2442535.17	751.7	--	--		
STN-9	556233.82	2442499.72	764.8	702.3	701.4	63.4	701.4
STN-9A*	556234.82	2442499.81	764.8	--	--		
STN-10**	556162.76	2442251.63	765.0	705.5	704.0	81.0	684.0
STN-11	556034.83	2442535.05	763.2	704.7	701.2	62.0	701.2
STN-12	555873.09	2442622.48	765.1	704.1	703.1	62.0	703.1
STN-13	Not Drilled						
STN-14	555685.58	2442733.21	753.1	706.1	702.1	51.0	702.1
STN-14A	555687.58	2442734.21	753.1	--	NR(734.6)	18.5	734.6
STN-14B*	555690.29	2442730.41	753.0	--	--		
STN-15	555662.69	2442667.52	763.7	706.2	704.8	58.9	704.8
STN-15A	555669.72	2442691.43	765.3	--	NR(735.3)	30.0	735.3
STN-15B	555672.20	2442690.32	765.3	--	NR(707.3)	58.0	707.3
STN-15C	555671.50	5442690.88	765.3	--	NR(730.3)	35.0	730.3
STN-15D	555665.72	2442690.78	765.3	--	NR(730.3)	35.0	730.3
STN-15E*	555694.24	2442681.34	765.0	--	--		
STN-16	555501.40	2442725.94	764.5	706.5	706.0	58.5	706.0
STN-17	Not Drilled						
STN-18**	555204.87	2442894.11	751.0	702.5	700.5	71.5	679.5
STN-18A*	555204.57	2442886.00	751.8	--	--		
STN-19	555204.68	2442842.94	765.6	701.1	700.6	65.0	700.6
STN-19A*	555204.68	2442843.94	765.6	--	--		
STN-20	555168.84	2442666.59	762.9	703.9	701.9	61.0	701.9
STN-21**	555076.61	2442149.85	765.0	709.0	707.5	77.5	687.5
STN-22**	554990.27	2441723.40	765.0	707.0	706.0	79.0	686.0

Table 1. Summary of Borings

Boring No.	Northing	Easting	Surface Elevation	Top of Rock Elevation	Refusal/ Begin Core Elevation	Boring Termination Depth	Bottom of Hole Elevation
STN-23	555020.22	2442857.46	764.7	703.7	702.7	62.0	702.7
STN-24	554803.45	2442843.16	765.1	701.6	700.9	64.2	700.9
SNT-25	Not Drilled						
STN-26	554624.86	2442889.00	750.0	696.0	692.7	57.3	692.7
STN-26A	554604.66	2442904.17	750.0	--	NR(730.0)	20.0	730.0
STN-26B*	554604.57	2442896.12	751.0	--	--		
STN-27	554601.77	2442850.67	765.1	700.1	697.6	67.5	697.6
STN-27A	554600.64	2442840.21	765.0	--	NR(705.0)	60.0	705.0
STN-27B	554606.18	2442840.52	765.0	--	NR(729.0)	36.0	729.0
STN-27C*	554607.18	2442840.92	765.0	--	--		
STN-28	554406.25	2442841.10	764.8	700.5	700.2	64.6	700.2
STN-29	554155.15	2442854.72	764.7	697.2	697.0	67.7	697.0
STN-30	Not Drilled						
STN-31	553954.94	2442758.22	749.5	696.5	695.5	54.0	695.5
STN-31A*	553960.30	2442764.66	749.7	--	--		
STN-32	553994.90	2442746.44	764.8	696.3	695.8	69.0	695.8
STN-32A*	553996.72	2442757.13	764.8	--	--		
STN-33	Not Drilled						
STN-34	553853.66	2442184.35	764.7	699.7	688.7	76.0	688.7
STN-35	Not Drilled						
STN-36	553776.74	2442198.78	751.9	712.4	707.9	44.0	707.9
STN-36A	553766.08	2442199.97	751.9	--	NR(736.9)	15.0	736.9
STN-36B*	553753.47	2442155.66	751.5	--	--		
STN-37	553799.90	2442184.40	763.8	712.8	709.6	54.2	709.6
STN-37A	553799.38	2442182.40	763.8	--	NR(734.8)	29.0	734.8
STN-37B	553800.49	2442186.20	763.7	--	NR(711.7)	52.0	711.7
STN-37C	553798.81	2442171.01	763.6	--	NR(730.1)	33.5	730.1
STN-37D	553798.26	2442168.95	763.6	--	NR(730.6)	33.0	730.6
STN-37E*	553799.26	2442183.40	763.8	--	--		
STN-38	553730.83	2441988.70	764.1	715.1	712.8	51.3	712.8
STN-39	Not Drilled						
STN-40	Not Drilled						
STN-41**	553583.10	2441510.71	752.7	714.7	709.7	63.0	689.7
STN-41A*	553584.59	2441512.86	751.8	--	--		
STN-42	553623.48	2441513.69	764.7	713.7	713.2	51.5	713.2
STN-42A*	553624.48	2441512.69	764.7	--	--		
STN-43	554004.68	2441548.50	765.9	718.9	711.9	54.0	711.9
STN-43A*	554005.71	2441549.61	765.9	--	--		
STN-44	Not Drilled						
STN-45	553740.50	2441308.40	763.9	716.4	175.4	48.5	715.4
SNT-46	Not Drilled						
STN-47	553747.39	2441146.83	753.4	717.4	713.9	39.5	713.9

Table 1. Summary of Borings

Boring No.	Northing	Easting	Surface Elevation	Top of Rock Elevation	Refusal/ Begin Core Elevation	Boring Termination Depth	Bottom of Hole Elevation
STN-47A	553733.63	2441141.18	753.1	--	NR(745.6)	7.5	745.6
STN-47B*	553742.43	2441143.28	753.8	--	--		
STN-48	553773.29	2441154.53	765.3	716.3	711.3	54.0	711.3
STN-48A	553771.75	2441159.01	765.3	--	NR(742.8)	22.5	742.8
STN-48B	553769.40	2441163.30	765.3	--	NR(717.1)	48.2	717.1
STN-48C*	553770.51	2441164.28	765.3	--	--		
STN-49	553921.85	2441024.39	763.1	718.1	715.6	47.5	715.6
STN-50**	553631.96	2440496.85	741.6	705.9	704.6	57.0	684.6
STN-50A*	553632.98	2440497.85	741.6	--	--		
STN-51	553696.02	2440548.46	750.4	701.5	700.8	49.6	700.8
STN-52	553992.98	2440817.40	753.2	714.2	713.7	39.5	713.7
STN-53	554011.12	2440902.46	763.9	713.9	712.4	51.5	712.4
STN-53A*	554012.14	2440904.05	763.9	--	--		
STN-54	555263.94	2441476.12	765.0	707.0	706.0	59.0	706.0
STN-55	554943.73	2442287.49	764.1	703.1	699.9	64.2	699.9
STN-56**	554555.61	2441998.50	765.8	704.8	703.7	82.0	683.8
STN-57	Not Drilled						
STN-58	Not Drilled						
STN-59	556075.53	2442603.06	752.2	701.2	700.7	51.5	700.7
STN-60	555886.92	2442663.31	752.5	701.0	699.0	53.5	699.0
STN-61	555513.59	2442792.80	752.5	706.5	701.0	51.5	701.0
STN-62	555020.69	2442907.23	749.8	702.8	700.8	49.0	700.8
STN-63	554822.75	2442910.57	750.0	702.5	701.0	49.0	701.0
STN-64	554411.29	2442911.08	749.4	702.9	693.9	55.5	693.9
STN-65	554147.51	2442915.09	748.6	700.6	698.6	50.0	698.6
STN-66	553888.83	2442564.24	750.9	695.9	693.9	57.0	693.9
STN-67	Not Drilled						
STN-68	Not Drilled						
STN-69	553607.58	2441718.01	752.3	707.3	692.8	59.5	692.8
STN-70	Not Drilled						
STN-71	553840.20	2440981.04	752.0	717.0	706.5	45.5	706.5

* Denotes Cone Penetration Test (CPT) boring.

** Boring was advanced approximately 20 feet into bedrock utilizing rock coring techniques
NR indicates no refusal

5.2. Subsurface Soil Conditions

Based on a review of the information obtained from the borings performed, placed embankment fill forming Dike C can be separated into four major horizons. Additionally, three major horizons have been identified as the foundation soils for Dike C.

Beginning just above the foundation soil materials, a discontinuous layer of "Gravel to Clayey Gravel", approximately three-foot thick, was encountered. This layer is most likely associated with placement of subgrade stabilization materials for starter clay dike construction. It should be noted that starter clay dike construction appears to have occurred at times within the impounded slack water area of Swan Pond formed by Watts Bar Reservoir.

The "Starter Clay Dike" horizon extends from the original groundline up to elevation 748 feet. The material was used to construct the initial perimeter containment dike prior to 1954. The starter clay dike soils primarily have a Unified Soil Classification System (USCS) classification of CL with lesser occurrences of CH, SM, and SC. Textural classifications consisted mostly of sandy lean clay and clayey sand with gravel. The soil was described as damp to wet in moisture content and brown to gray in color. Based on SPT N-values and laboratory strength testing, the starter clay dike has strength consistencies ranging from soft to medium stiff.

Above the starter clay dike, the "Constructed Ash" horizon, extending from approximate elevation 748 feet to elevation 750 feet was encountered primarily on the eastern leg of Dike C. The constructed ash was placed as an interim raise in 1974, to provide additional freeboard for the ash pond. It should be noted that the constructed ash was denoted as a thicker layer with a maximum thickness of 13.5 feet in some areas on the boring logs. This greater thickness is most likely attributed to some interim use of available ash materials to form the starter dike. The material generally consists of bottom ash. The bottom ash was described as damp to wet in moisture content and gray to dark gray and black in color. Based on SPT N-values the constructed ash has strength consistencies ranging mostly from loose to dense.

From 1954 to 1976, ash sluiced into the pond formed the "Hydraulically Placed Ash" horizon that now lies below the third raising of Dike C. The hydraulically placed ash extends from the original ground surface to approximate elevation 745 feet. The material generally consists of bottom ash and fly ash in varying mixtures. Classification tests performed on a selected hydraulically placed ash samples resulted in a USCS classification of SM and a textural classification of silty sand with gravel. The hydraulically placed ash was described as damp to wet in moisture content and gray to dark gray and black in color. Based on SPT N-values the hydraulically placed ash has strength consistencies ranging from very loose to dense.

The "Raised Clay Dike," constructed between 1976 and 1978, extends from approximate elevation 745 feet to elevation 765 feet. The raised clay dike soils classified primarily as CL with areas of CH. The raised clay dike had textural descriptions of lean clay with sand, sandy lean clay, fat clay with sand, and sandy fat clay. The soil was described as damp to wet in moisture content and mostly brown in color. Based on SPT N-values and laboratory strength testing, the raised clay dike has strength consistencies ranging from very soft to very stiff.

The uppermost native foundation material encountered at the boring locations consists of a silt and clay material, which appears to be original ground lake sediment deposits associated with the Swan Pond embankment. The material designated herein as the "Sensitive Silt/Clay" horizon, approximately a half to one-foot in thickness, lies near the original ground surface. The sensitive silt/clay soils classified primarily as CL-ML with occurrences of CL and ML. The sensitive silt/clay horizon was usually described as having a silt or silty clay texture, light brown in color, and saturated in moisture content. Based on SPT N-values the sensitive silt/clay horizon has strength consistencies ranging from very soft to soft.

No material matching the "slime layer" as described in the Dredge Cell Root Cause Analysis report was noted to be encountered during this exploration. This is likely attributed to the subject study area being the most active portion of the historical ash pond operations. If the material had formed in this area, probably removed in association with periodic dredging operations.

There is an occasional "Lean Clay Foundation Soil" layer found below the sensitive silt/clay horizon ranging from about three to twelve feet in thickness. The lean clay foundation soil had a USCS classification of CL with textural descriptions of sandy lean clay and lean clay with sand. The soil was described as predominately saturated in moisture content and light brown in color. Based on SPT N-values the lean clay foundation soil has strength consistencies of very soft to medium stiff.

Below the sensitive silt/clay horizon, "Sandy Silt to Silty Sand" was encountered down to elevations ranging between 717 feet and 703 feet. The sandy silt to silty sand had USCS classifications of predominately SM with occasional occurrences of CL, ML, and CL-ML. Textural descriptions were mostly of silty sand and sandy silt with isolated pockets of clayey soils typical of alluvial deposits. The soil was described as brown to gray in color and moist to saturated in moisture content. Based on SPT N-values and laboratory strength testing, the sandy silt to silty sand has strength consistencies ranging mostly from very loose or very soft to medium dense or medium stiff.

Below the sandy silt to silty sand horizon, "Fine Grained Sand" was encountered down to bedrock having an elevation ranging from about elevation 717 feet to 700 feet. The fine grained sand horizon had a USCS classification of SM with a textural classification of sand. The soil was described as wet or saturated in moisture content and brown to gray in color. Based on SPT N-values the fine grained sand horizon has strength consistencies of loose to very dense.

5.3. Instrumentation

5.3.1. Piezometers

At select boring locations, piezometers were installed to measure pore water pressures within Dike C. In general, piezometer readings were obtained on four separate occasions spaced approximately two weeks apart, beginning with the initial reading on May 28, 2009. Based on Stantec's current scope, it is anticipated that these readings will continue every two weeks until the middle of November. This report includes the results up to the last reading date on July 9, 2009; future readings will be issued under a separate cover. Refer to appendixes C and D for piezometer installation details and the latest piezometer readings, respectively. Piezometer locations and tip elevations are summarized in Table 2 below.

Table 2. Summary of Piezometers

Boring No.	ID	Concrete Pad Elevation (Feet)	Piezometer Tip Elevation (Feet)
STN-2A	PZ-1	751.2	724.7
STN-3A	PZ-2 (U)	763.9	751.9
	PZ-3 (L)		737.9
STN-3B	PZ-4	763.8	718.8
STN-14A	PZ-5	753.1	737.6
STN-15A	PZ-6 (U)	765.3	753.3
	PZ-7 (L)		738.3
STN-15B	PZ-8	765.3	710.3
STN-26A	PZ-9	750.0	733.0
STN-27A	PZ-10 (U)	765.0	752.2
	PZ-11 (L)		735.2
STN-27B	PZ-12	765.0	708.2
STN-36A	PZ-13	751.9	738.4
STN-37A	PZ-14 (U)	763.8	752.9
	PZ-15 (L)		738.1
STN-37B	PZ-16	763.7	715.0
STN-47A	PZ-17	753.1	746.9
STN-48A	PZ-18 (U)	765.3	756.7
	PZ-19 (L)		746.1
STN-48B	PZ-20	765.3	720.1

U – Denotes upper piezometer within the same boring

L – Denotes lower piezometer within the same boring

In general, the water levels were fairly consistent, with slight increases in the water elevation reported for the crest borings and mostly decreases in the water level observed within the toe borings. These changes in the water level observed over the two months of readings are likely attributed to equalization of the water level within the piezometers over time. However, it should be noted that water levels can also fluctuate due to the seasons, precipitation events, variations in pool levels, and other factors.

5.3.2. Slope Inclinometers

Slope inclinometers were installed in borings STN-8, STN-18, STN-34, STN-64, STN-69, and STN-71 to monitor any future movement within Dike C. In general, slope inclinometer readings were obtained soon after installation of the slope inclinometer casing so that a baseline reading could be established. The readings were then taken on a weekly basis with other instruments that were installed as a part of separate projects. Based on Stantec's current scope, it is anticipated that these readings will continue every two weeks for the next six months. This report includes the results up to the last reading date on July 9, 2009; future readings will be issued under a separate cover. Based on the readings taken to date, there has been no significant movement in any of the slope inclinometers installed within Dike C. Refer to Appendixes C and D for slope inclinometer installation details and the latest slope inclinometer readings, respectively.

6. Laboratory Testing

6.1. General

Stantec performed laboratory testing in accordance with applicable ASTM soil testing specifications. Results from the laboratory tests are included in Appendix E. In particular, natural moisture content test results are shown on the attached boring logs in Appendix A and are also shown on the drafted sheets in Appendix F. No further discussion relative to the results of moisture content and classification testing are provided in this section. The discussion that follows is limited to the laboratory testing associated with evaluation of the laboratory strength test characteristics.

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

The borings drilled for the subject dike included 3-inch diameter undisturbed (Shelby) tube sampling within predominantly cohesive soil horizons. Stantec's soils laboratory extruded the tubes and trimmed six-inch long specimens. Lab personnel determined visual inspections, unit weights (wet and dry), and natural moisture for each six-inch specimen prior to submitting a summary of the extruded specimens to a geotechnical engineer for assignment of lab testing. Select 6-inch specimens extruded from Shelby tubes were subjected to unconfined compressive (UC) strength testing, consolidated-undrained (CU) triaxial testing, unconsolidated-undrained (UU) triaxial testing, direct simple shear (DSS), testing one-dimensional consolidation, and testing permeability testing. Selected tube specimens associated with continuous sampling at the pond base/original ground elevation were also subjected to longitudinal cutting, visual inspection and laboratory microscopic viewing to identify any suspect "slime" type materials. The results of these tests are included in Appendix E and discussed below.

6.2.1. Unconfined Compressive (UC) Strength Testing

Unconfined compressive strength testing was performed to provide information from which soil strength parameters could be estimated. The results of the unconfined compressive strength testing are presented next to the sample borings on the geotechnical drawings in Appendix F and are summarized in Table 3.

Table 3. Summary of Unconfined Compressive Strength Tests

Boring No.	Sample Interval (feet)	Dry (pcf)	Wet (pcf)	Moisture Content (%)	Unconfined Compressive Strength (psf)
STN-20	35.0 - 35.5	103.6	127.0	23.7	760
STN-55	42.6 - 43.1	106.1	128.0	20.4	2,020

6.2.2. Consolidated-Undrained (CU) Triaxial Testing

Stantec performed CU triaxial testing with pore pressure measurements on selected 6-inch long specimens extruded from 3-inch diameter Shelby tubes obtained during drilling. CU testing provides indicators of effective-stress shear-strength parameters. The results of the CU triaxial tests are presented on the stability section in Appendix F, and are summarized in Table 4. The stress path envelopes derived from CU triaxial testing are also presented in Appendix E.

Table 4. Summary of Consolidated – Undrained Triaxial Testing

Boring No.	Sample Interval (feet)	USCS Classification	CU Triaxial Strength	
			c' (psf)	ϕ' (degrees)
STN-28	38.3 - 38.9		103	32.3
	38.9 - 39.5	CL		
	46.3 - 46.9			
STN-29	60.2 - 60.8	CL	0	34.0
	60.8 - 61.4			
	61.4 - 62.0			
STN-26	20.0 - 20.5		125	37.8
STN-64	20.6 - 21.1	CL		
STN-27	30.3 - 30.8	CL-ML		
STN-31	33.6 - 34.1		345	32.6
STN-66	33.6 - 34.1			
STN-32	50.0 - 50.5			
STN-3D	31.0 - 31.6	CL-ML	113	35.6
	31.6 - 32.2			
STN-37D	31.6 - 32.8			
STN-37	13.5 - 14.0		40	41.1
	14.3 - 14.8	CH		
	20.0 - 20.3			

Table 4. Summary of Consolidated – Undrained Triaxial Testing

Boring No.	Sample Interval (feet)	USCS Classification	CU Triaxial Strength	
			c' (psf)	ϕ' (degrees)
STN-14	23.6 - 24.1		240	33.2
	24.2 - 24.7	CL-ML		
STN-61	23.0 - 23.5			
STN-2	9.0 - 9.5		400	34
	14.0 - 14.5	CL		
	14.6 - 15.1			
STN-50	20.0 - 20.5		80	36.6
	20.5 - 21.0	CL		
STN-51	23.5 - 24.0			
STN-3	10.5 - 11.0		600	21.1
	12.5 - 13.0	CH		
STN-5	11.3 - 11.8			

6.2.3. Direct Simple Shear (DSS) Testing

Stantec selected samples extruded from Shelby tubes for DSS testing to provide additional indicators of effective-stress shear-strength parameters. The results of the DSS tests are presented in the stability section in Appendix F, and are summarized in Table 5. The stress path envelopes are also presented in Appendix E.

Table 5. Summary of Direct Simple Shear Testing

Boring No.	Sample Interval (feet)	USCS Classification	DSS Strength	
			c' (psf)	ϕ' (degrees)
STN-15D	34.00 - 34.25	CL	170	21
STN-15D	34.25 - 34.50		210	24
STN-27B	33.20 - 33.45	CL	60	22
STN-27B	35.20 - 35.45	CL-ML	260	25
STN-27B	35.45 - 35.80		100	27

6.2.4. Unconsolidated – Undrained (UU) Triaxial Testing

Stantec performed UU triaxial testing on selected soil specimens to provide information for estimated total stress strength parameters for cohesive soil horizons. Table 6 summarizes the data obtained from this test. The Subsurface Data Sheets provided in Appendix F also depict the results of the UU testing adjacent to the appropriate graphical log. The laboratory testing results are also included in Appendix E.

Table 6. Summary of Unconsolidated – Undrained Triaxial Testing

Boring No.	Sample Interval (feet)	Confining Stress (psi)	UU Triaxial Strength	
			(psf)	(tsf)
STN-2	20.5 - 21.0	14	1,224	0.61
STN-2	30.6 - 31.1	18	1,670	0.84
STN-5	38.5 - 39.0	24	660	0.33
STN-8	15.5 - 16.0	13	660	0.33
STN-18	25.5 - 26.0	15	1,670	0.84
STN-19	5.0 - 5.5	4	4,310	2.15
STN-19	39.7 - 40.2	25	880	0.44
STN-22	49.5 - 50.0	20	3,460	1.73
STN-27	12.6 - 13.1	10	2,580	1.29
STN-27	44.7 - 45.2	27	1,440	0.72
STN-31	20.5 - 21.0	10	600	0.30
STN-31	38.0 - 38.5	19	450	0.22
STN-32	5.3 - 5.8	3	2,510	1.25
STN-36	29.0 - 29.5	13	300	0.15
STN-41	15.1 - 15.6	12	1,340	0.67
STN-43	29.1 - 29.6	16	1,390	0.70
STN-48	33.5 - 34.0	21	3,840	1.92
STN-48	4.0 - 4.5	3	1,930	0.97
STN-50	9.0 - 9.5	7	490	0.24
STN-63	9.3 - 9.8	5	900	0.45
STN-64	20.0 - 20.5	11	490	0.25
STN-64	30.0 - 30.5	15	1,930	0.97
STN-65	13.5 - 14.0	10	2,220	1.11

6.2.5. One-Dimensional Consolidation Testing

Stantec selected samples extruded from Shelby tubes for one-dimensional consolidation testing to provide initial void ratio and consolidation parameter indicators for estimates of use in settlement analyses. The results of the consolidation tests are summarized in Table 7 and are presented in Appendix E.

Table 7. Summary of One Dimensional Consolidation Tests

Boring No.	Test Interval (feet)	Initial Void Ratio (e_o)	Compression Index (C_c)	Recompression Index (C_r)	Pre-Consolidation Pressure (Pc) (psf)
STN-2	31.4 - 31.9	0.589	0.182	0.025	5,000
STN-9	11.7 - 12.2	0.844	0.216	0.040	12,000
STN-14	24.8 - 25.3	0.534	0.145	0.025	4,000
STN-19	14.1 - 14.6	0.681	0.125	0.032	6,200
STN-32	14.0 - 14.5	0.700	0.175	0.035	6,000
STN-52	25.2 - 25.7	0.573	0.150	0.025	3,800
STN-63	15.2 - 15.7	0.785	0.110	0.020	1,600
STN-64	21.2 - 21.7	0.752	0.320	0.030	1,200
STN-65	33.8 - 34.3	0.815	0.225	0.033	3,400

6.3. Testing of Standard Penetration Test (SPT) Samples

In general, recovered soil specimens from SPT samples were subjected to natural moisture content determinations and select samples were combined for engineering classification testing. The engineering classification testing consisted of Atterberg limits, specific gravity, and sieve and hydrometer analyses. The results of the classification testing were used in conjunction with the N-values from SPT's to estimate soil strength based on published correlations of such data. The results of the moisture content tests are included on the boring logs and drawings in Appendixes A and F, respectively. The results of the engineering classifications are included on the drawings in Appendix F.

7. Engineering Analyses

7.1. General

Geotechnical engineering analyses included evaluations of strength and permeability parameters, seepage analyses, and slope stability analyses. Prior to beginning the analyses, the geotechnical data and cross sections were combined and the geometry of the existing dikes and soil horizons were approximated using current and historical information. Once the geometry of the sections was determined, each section was reviewed and evaluated to determine the critical cross section for analyses. The criteria for selecting the critical sections was based on the steepness of slopes, the geometry of the sections, and the phreatic surface, and soil conditions. Based on this evaluation, representative cross sections were selected for analyses. Results of the analyses and evaluations are summarized in the following paragraphs. Results of the seepage and slope stability analyses, along with the plan locations of the cross sections, are shown on the drawings included in Appendix F.

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

were needed to construct the seepage and stability models. The 2008 failure of the adjacent dredge cell deposited coal ash against the slope of Dike C, mostly along the eastern reach. This accumulation of ash, which is highly erodible and will be removed during the cleanup, was ignored during the analyses.

7.2. Soil Horizons

Based on the results of the drilling, laboratory testing, historical documentation, and drawings, the materials on site were divided into different soil layers. Refer to the stability sections in Appendix F for locations of the soil horizons. For these analyses, the soil layers are as follows:

- *Starter Clay Dike*: This represents the material used for construction of the original perimeter dike. Based on historical information the interior slopes were constructed at a 3:1 (H:V) and the exterior slopes were constructed at a 6:1 (H:V).

- *Raised Clay Dike*: This represents the material that was used for the upstream raising of the original dike.

- *Constructed Ash*: This represents material encountered during the field exploration above the original starter dike between Station 127+00 and 160+00 on the eastern portion of the dike. This material appears to have been used for an interim raising of the starter dike or used for a repair. The material consists of a mixture of fly ash and bottom ash.

- *Hydraulically Placed Ash*: This represents the material that is contained by Dike C. It was encountered upstream of the starter dike and below and upstream of the raised dike.

- *Gravel to Clayey Gravels:* This represents a gravelly layer that is in place below the original dike construction. This material appears to have been used to stabilize the original dike construction.

- *Sensitive Silt/Clay:* This represents the uppermost and relatively recent slack water silt and clay sediment deposits associated with the Swan Pond embankment at the time of the initial ash pond construction. The results of the exploration indicate this material is generally very soft and exhibits relatively low shear strength.

- *Lean Clay Foundation Soil*: This represents a lean clay foundation layer that was encountered in several of the borings.

- *Sandy Silt to Silty Sand*: This represents a foundation layer that was encountered at varying thickness within all the borings.

- *Fine Grained Sand to a Sand with Silt*: This represents a foundation layer that was encountered in several of the borings with varying thickness as the borings were advanced near the top of bedrock.

7.3. Seepage Analysis

7.3.1. SEEP/W Model

An analysis of steady state seepage through the dike was needed to estimate the magnitude of seepage gradients (for the evaluation of potential piping) and pore water pressures within the soils (for the evaluation of slope stability). The numerical seepage model for Kingston Fossil Plant Dike C 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).

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

Five cross sections through Dike C were modeled with SEEP/W, then subsequently evaluated for slope stability (Section 7.5). For the numerical analysis, each cross section was subdivided into a mesh of elements, consisting of first-order quadrilateral and triangular finite elements. For seepage problems, where the primary unknown (hydraulic head) is a scalar quantity, first-order elements provide for efficient, effective modeling. Given appropriate hydraulic conductivity properties and applied boundary conditions, the finite element method (as implemented in the SEEP/W code) was then used to simulate steady seepage across the mesh. The total hydraulic head is computed at each nodal location, from which pore water pressures and seepage gradients can be determined.

7.3.2. Boundary Conditions

Steady-state seepage was assumed for the analysis, with static water levels on the upstream and downstream sides of Dike C. For the cross sections at Station 108+93, 138+27, and 149+14, the ash pond was assumed to be at elevation 760 feet. The cross sections at Station 119+69 and 132+37 pass through the stilling basin, where the water levels were assumed to be at elevation 755 feet. On the downstream side, Watts Bar Lake was assumed to be at an elevation of 737 feet, corresponding to a normal winter pool. This lower pool elevation was selected to represent the more severe conditions for stability, with increased seepage gradients and lower external stabilizing pressures.

Boundary conditions for the SEEP/W analysis were assumed as follows. Along the vertical, upstream and downstream edges of the model, the hydraulic head at each node was constant with depth and equal to the pool elevation on that side of the dike. A total head equal to the pool level was also applied to all submerged nodes along the ground surface of the model. Other nodes along the ground surface were treated as potential seepage exits. At steps in the analysis, if water was flowing out of the mesh at these nodes, SEEP/W assigned a head equal to the elevation of the node. This routine effectively models the seepage of water out to the ground surface. The horizontal boundary at the base of the model (located within the shale bedrock) was modeled as a seepage barrier, with no vertical flow across the boundary nodes.

7.3.3. Seepage Properties

For each modeled cross section of the dike, 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, if no data was available, based on typical values for similar soils. Material properties used in the seepage analysis are summarized in Table 8.

Table 8. Material Properties for SEEP/W Analysis

Soil Horizon	Saturated k_v (cm/s)	Ratio k_h / k_v	Specific Gravity G_s	Void Ratio e	Volumetric Water Content		Basis
					Saturated (%)	Residual (%)	
Hydraulically Placed Ash	3.0e-5	50	2.31	0.85	46	0.04	Available Laboratory Data (Test Trench #3)
Constructed Ash	3.0e-5	25	2.31	0.85	46	0.04	Available Laboratory Data (Test Trench #3)
Clay Raised Dike	1.2e-7	10	2.74	0.83	45	0.03	Available Laboratory Data (STN-27 & STN-48)
Clay Starter Dike	3.1e-8	10	2.69	0.61	38	0.02	Available Laboratory Data (STN-8)
Sensitive Silt/Clay	3.0e-5	50	2.31	0.85	46	0.04	Available Laboratory Data (Test Trench #3)
Lean Clay Foundation Soil	1.9e-8	20	2.70	0.61	38	0.02	Available Laboratory Data (STN-36)
Sandy Silt to Silty Sand	1.0e-5	50	2.70	0.65	39	0.01	Based on Typical Values
Gravel to Clayey Gravel	1.0e-3	50	2.70	0.65	39	0.01	Based on Typical Values
Fine Grained Sand to Sand with Soil	1.0e-5	50	2.70	0.65	39	0.01	Based on Typical Values s
Shale	1.0e-7	10	2.60	0.25	20	0.01	Based on Typical Values

Note: SEEP/W requires input parameters k_h and ratio of k_v/k_h

Significant engineering judgment is needed to select appropriate hydraulic properties for earth materials. Unlike other key properties, hydraulic conductivity can vary over several orders of magnitude for a range of soils, often with substantial anisotropy for seepage in horizontal versus vertical directions. Laboratory test samples often do not represent important variations within a larger soil deposit. For Dike C, an iterative process of parametric

calibration (Section 7.3.4) was used to arrive at final estimates of the seepage properties. Results from trial simulations were compared to field data (measured piezometric levels and the depth to groundwater in borings). The material parameters were then varied until the solutions reasonably matched the field data on all five cross sections. The final set of parameters (Table 8) resulted in the comparisons presented in Section 7.3.4.

Hydraulic conductivities for vertical seepage through saturated materials (k_v) were estimated using available laboratory data. As indicated in Table 8, typical values for similar soils (obtained from various published sources) were used where laboratory data were not available. The value of k_v selected for the alluvial sandy silt to silty sand foundation deposit is one example where engineering judgment was critical to the selection of appropriate material properties. Laboratory permeability tests were conducted on Shelby tube samples of cohesive soils within this deposit; however, the global conductivity of this layer will be closer to that of the more predominant sandy materials.

The ratio of horizontal hydraulic conductivity (k_h) to vertical hydraulic conductivity (k_v) was estimated based on placement of the material. An isotropic material would have $k_h/k_v = 1$, while deposits of horizontally layered soils might have values as high as $k_h/k_v = 100$. For Dike C, relatively high ratios ($k_h/k_v = 50$) were assumed for the hydraulically placed ash and the silty sand to sandy silt, in both cases reflective of periodic deposition of materials with different gradations. Such deposits typically exhibit much greater permeability in the horizontal direction than in the vertical direction. More modest values ($k_h/k_v = 10$) were assumed for the dike materials, which would have been constructed in horizontal lifts.

The governing equations in SEEP/W are formulated to consider seepage through unsaturated soils. In the simulations for Dike C, this formulation is used to locate the phreatic surface for unconfined seepage through the dike cross sections. To represent the change in hydraulic conductivity due to de-saturation of each soil, SEEP/W implements a model based on two curves, a hydraulic conductivity function and a volumetric water content function. Three parameters are needed to define this behavior: the saturated hydraulic conductivity, saturated water content, and residual water content (water content of air dried soil). Of these, only the residual water contents were not previously estimated for each soil. Values were estimated (Table 8) based on typical values for similar soils. Parametric variations, wherein the hydraulic conductivity of the surficial soils were held constant for all saturation levels, indicated that the simulation results were not sensitive to these model selections.

7.3.4. Comparison to Field Observations

After the initial seepage parameters were estimated, results from the SEEP/W model were compared to pore water pressures measured in piezometers installed in Dike D. Data from 12 piezometers, at locations on three of the five modeled cross sections, were used in this evaluation. Nodes were placed in the model at the same location as the piezometer tip was installed in the field, then the total head predicted at the node was compared to the corresponding piezometer reading.

Considering the results for all 12 piezometers, the material properties in each modeled cross section were varied until a reasonable match was obtained between the predictions and field data. Specifically, the saturated hydraulic conductivity of the sandy silt to silty sand and the gravel to clayey gravel were varied, as was the k_h/k_v ratio for all materials. After several iterations, the final soil parameters were within expected ranges, based on soil type and laboratory data, and calibrated to give model predictions consistent with field measurements.

The comparison between the field measurements and SEEP/W predictions at each piezometer are plotted in Figure 2. The maximum difference between the predictions and measurements is 7.3 feet (a PZ9), while most differ by less than 3 feet. These differences are acceptable in a seepage model, given the typical differences between the modeled cross section and unknown details within the subsurface soil conditions.

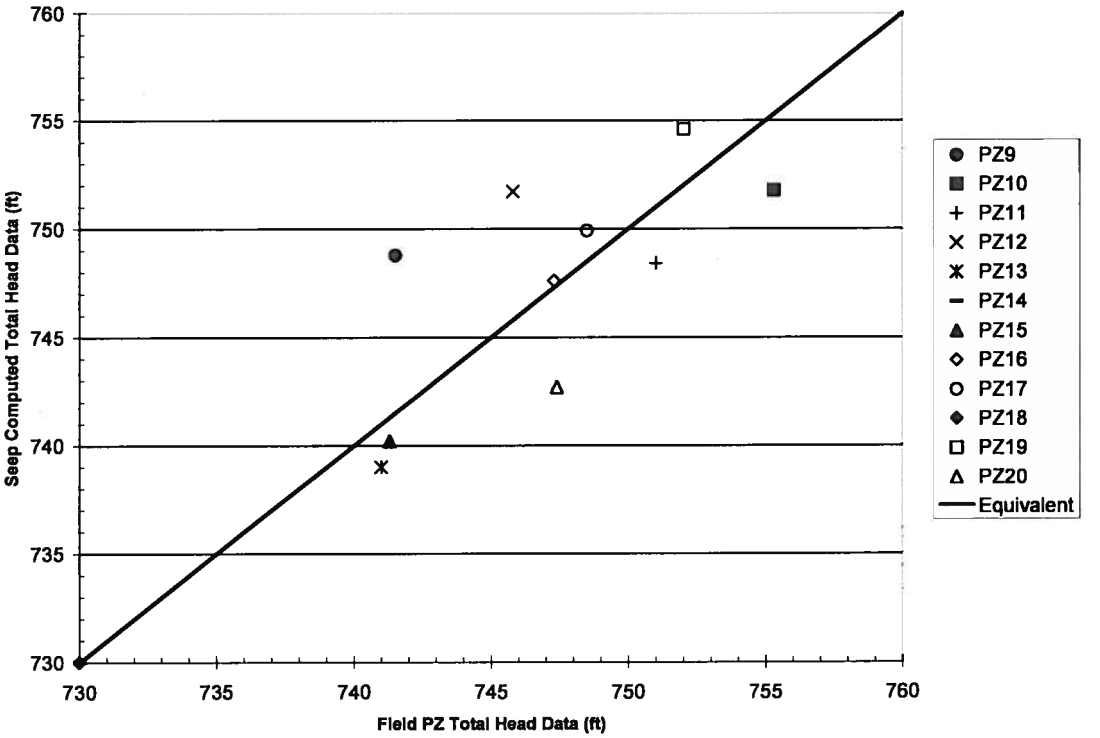


Figure 2. Comparison between the Field Piezometer Readings and Pore Water Pressures predicted in the SEEP/W Model

The results from the seepage model can also be compared to observations of water on the ground surface and within boreholes in the field. Following the recent removal of trees along the starter dike, standing water has been observed in areas where the tree stumps have been removed. The SEEP/W results (see plots in Appendix H) indicate seepage near the ground surface in these areas, consistent with these field observations. In Figure 3, the elevation of water in the 2009 boreholes are compared to the elevation of the phreatic surface (ground water table) predicted with the seepage model at the same location. With a couple of exceptions, the observed water levels are below the predicted phreatic surface. This may result from having insufficient time for the borehole water levels to reach equilibrium, as well as intercepting subsurface strata with varying piezometric levels.

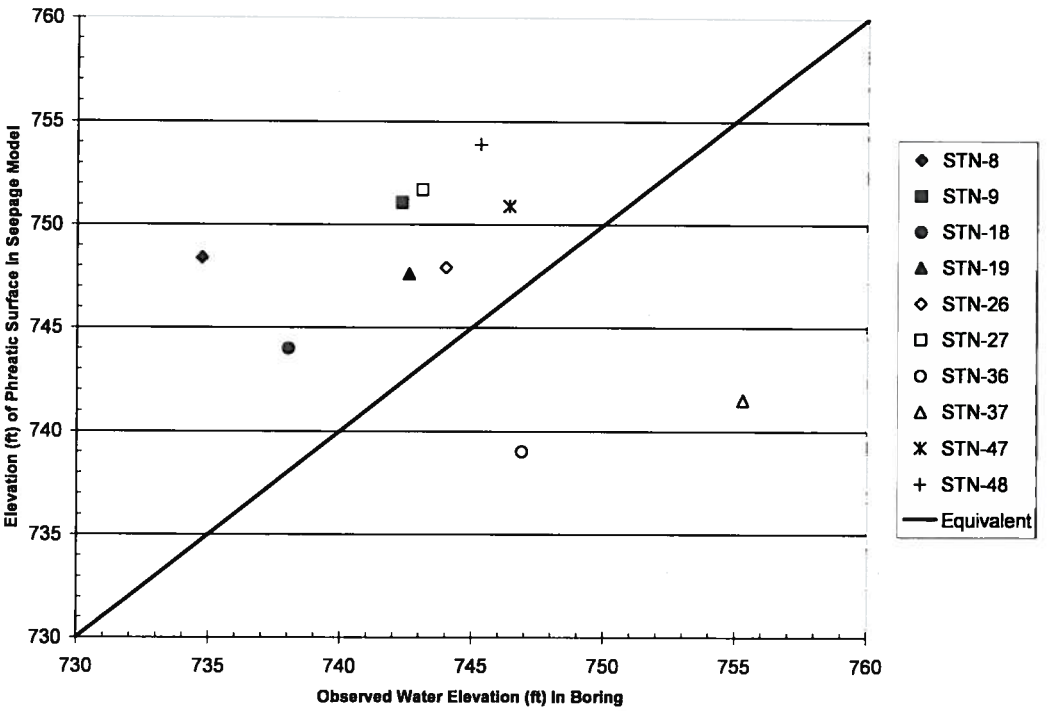


Figure 3. Comparison between the borehole water levels and the phreatic surface predicted in the SEEP/W Model

7.3.5. Results from Seepage Analyses

Plots from the SEEP/W analyses of the five cross sections through Dike C are presented in Appendix H. The plots show the finite element mesh, material zones, and boundary conditions used in each analysis. The results are depicted in contour plots of total head, pore water pressure, and seepage gradients. For the slope stability analyses (Section 7.5), the pore water pressures along the considered slip surfaces were determined by interpolation between the nodal pore pressures predicted with he SEEP/W model. The seepage gradients were assessed for maximum exit gradients and the potential for soil piping (Section 8.1).

The phreatic surface (groundwater table or line of zero pore water pressure) is shown on the plots in Appendix H. In several cases, the shape of the phreatic line may not meet one's initial expectations, particularly where more pervious materials are found below the dike zones near the ground surface. In SEEP/W, the location of the phreatic surface is found by interpolation between positive pore water pressures in the upper areas of saturated soil and negative or suction pore pressures in the unsaturated soil zone above. In the SEEP/W formulation, seepage flows are tracked in both the saturated and unsaturated zones. Hence, the top flow line in the SEEP/W results will be above the phreatic line. In more traditional seepage analyses, where unsaturated flows are ignored, the top flow line and the phreatic

surface coincide. Hence, while the more complete unsaturated flow formulation in SEEP/W gives a reasonable prediction of the phreatic surface location and shape, the results are often different than would be obtained with a solution that considers only saturated flow. Furthermore, the pore water pressures in the stability analysis are determined from the full finite element solution, and not just from the depth below phreatic surface.

7.3.6. Critical Exit Gradients

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

Where upward seepage through a uniform soil exits to the ground surface, the factor of safety with respect to soil piping (FS_{piping}) is defined as:

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

where i is the vertical gradient in the soil at the exit point. The critical gradient (i_{crit}) is related to the submerged unit weight of the soil, and can be computed as:

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

where γ_{sub} is the submerged unit weight of the soil, γ_w is the unit weight of water, G_s is the specific gravity of the soil particles, and e is the void ratio. For nearly all soils, the critical gradient is between about 0.6 and 1.4, with a typical value near 1.0.

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

7.4. Strength Parameter Selection

The static stability of Dike C around the ash pond and stilling pond at the Kingston Fossil Plant were evaluated using the limit equilibrium slope stability methods. The soil parameters used in these stability analyses were established as follows.

Dike C was originally constructed in the mid 1950s and was raised in the mid 1970s. The dike has existed in its current cross sectional geometry (slopes and crest elevation) for at least 30 years. Hence, excess pore pressures generated in the underlying soil during construction have had sufficient time to dissipate and steady state seepage conditions have developed within the dike. In addition, the current analyses will focus only on static conditions (no earthquake or other dynamic loads). For these conditions, only soil unit weights and drained strength parameters (c' and ϕ') are needed. If stabilizing berms or other modifications to the dike cross section are built, then undrained, total stress stability analyses will be needed to assess stability during construction.

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

To select the representative strengths for each horizon, the methodology outlined in the US Army Corps of Engineers Engineer Manual EM 1110-2-1902 was used as a guide. Failure stresses measured in the laboratory tests were expressed in terms of "p'-q" values $[p' = 0.5(\sigma_1' + \sigma_3'), q = 0.5(\sigma_1' - \sigma_3')]$, then envelopes were conservatively fit through the data. In general, the selected strength parameters represent a failure envelope where about two-thirds of the test data falls above the envelope.

7.4.1. Drained Soil Parameters

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

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

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

The Mohr-Coulomb failure envelope for normally consolidated, saturated clays exhibits $c' = 0$. At effective stresses below the preconsolidation pressure, overconsolidated clays have a curved failure envelope that can be represented with a straight line having $c' > 0$. However, overconsolidated clays in the field are often fissured and the in situ c' is significantly smaller than values determined from testing of small samples in the laboratory. To avoid progressive failures in overconsolidated, stiff fissured clays, remolded soil samples are recommended for testing; this generally results in "fully softened" strengths with $c' = 0$.

Thus, in the absence of particle cementation/bonding, long term (drained) shearing resistance related to $c' > 0$ is considered unreliable. In routine geotechnical design practice, values of $c' = 0$ are usually assumed for both normally and overconsolidated saturated clays, and for uncemented granular soils. Detailed testing and characterization of a particular soil, coupled with careful application of the fitted strength envelopes, are necessary where values of c' are used in a stability evaluation.

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

7.4.2. Soil Parameters for Dike C

The shear strength parameters were conservatively estimated based on site-specific geotechnical data, published data/information and our experience with these materials in similar applications. Discussions regarding selection of the shear strength parameters are provided in the following paragraphs. Refer to Table 9 below for a summary of derived soil parameters.

The starter clay dike, raised clay dike, and lean clay foundation primarily consist of lean clay materials with occasional occurrences of fat clays. The cohesive soils sampled during the field exploration were subjected to CU triaxial tests. The results of triaxial testing were evaluated and effective stress p' versus q scatter plots were prepared using all of the data points. Failure was assumed to occur at the point of the maximum effective principal stress ratio (σ_1'/σ_3') Once the p' versus q plots were prepared, a failure envelope was then selected such that about two-thirds of the plotted values were above the envelope. The p' versus q plots and selection of the failure envelope are shown for each soil horizon on the graphs presented in Appendix G. The resulting strength parameters were rounded down to the nearest degree for ϕ' and to the nearest 50 pounds per square foot for the cohesion intercept. Consistent with the discussions in Section 7.4.1, the measured cohesion intercept was neglected (assumed c'=0) in evaluating the dike stability. The unit wet weight was determined by taking the average unit weight of the samples that are included within a soil horizon.

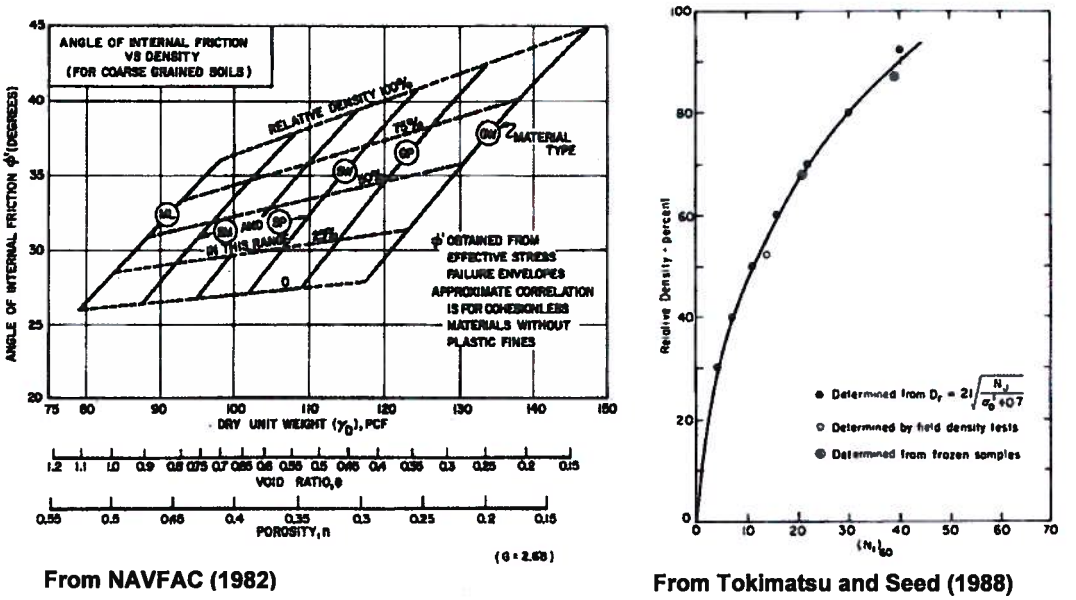
The constructed ash materials were primarily encountered in the upper portion of the eastern most starter dike. The material consists of a mix of fly ash and bottom ash. As a part of the Kingston Dredge Cell Closure Project, Stantec performed CU triaxial tests and the results were plotted on a scatter (Appendix G) plot as described above. The results compacted on samples of this material indicate that the ϕ' for the constructed ash is on the order of 38°. However, those tests were completed assuming the ash will be constructed using typical construction techniques with controlled compaction requirements associated with an engineered fill. The constructed ash material encountered in Dike C around the Ash Pond were placed several years ago and there is a lack of documentation on how the material was placed. Assuming this ash was not adequately compacted when placed in Dike C, it is prudent to assign a lower strength of ϕ' = 30°, which would be typical for an ML type material at medium density. The unit wet weight was determined by taking the results of the standard Proctor compaction tests conducted by Stantec and AECOM as a part of the projects associated with the dredge cell. It was then estimated that the material exists at a density corresponding to 90-percent of the standard Proctor maximum (75 pcf) and was near the optimum moisture content (24%).

The hydraulically placed ash materials were primarily encountered upstream of the starter dike and below the raised dike. As a part of the RCA, AECOM performed 25 triaxial compression tests with various consolidation techniques. In addition, six additional triaxial tests were completed by Law Engineering, Inc. in 1995, as a part of a testing program on ponded ash materials in Dredge Cell I and III. There results were plotted on a scatter plot (Appendix G), materials which indicate that ϕ' for the hydraulically placed ash is on the order of 25°. The unit wet weight was estimated using a dry unit weight at 90-percent of the standard Proctor maximum (75 pcf) and adjusting it based on the average in place moisture content (28%) of the hydraulically placed ash.

The gravel to clayey gravel layer was modeled just below the original dike. The layer was encountered during the field exploration and may have been placed below water levels as a working platform layer during initial construction of the starter dike. Strength and unit weight properties for this soil were estimated using correlations with measured SPT blowcounts. For a value of $(N_1)_{60} = 8$ in this soil horizon, the correlations in Figure 4 suggest $\phi' = 32°$.

The sensitive silt/clay was modeled near the top of original ground. The layer was identified as a result of the RCA performed by AECOM and the results of Stantec's field exploration program. Under the direction of AECOM, the University of Massachusetts (UMass) Amherst performed 12 direct simple shear (DSS) tests on thin-walled tube specimens obtained within the Dredge Cell. In addition, Geotesting Express (under the direction of Stantec) performed five DSS tests on thin-walled tube specimens obtained within the Dredge Cell. There results were plotted on a scatter plot as described above and a value of $\phi' = 28°$ was determined. The unit wet weight was determined by taking the average unit weight of the DSS test specimens obtained by Stantec.

The sandy silt to silty sand and fine grained sands were modeled at varying thicknesses within the foundation alluvium. Based on the results of the drilling program there appears to be variability in the results of the SPT N-values for these non-cohesive soil horizons. Therefore, the soil parameters were derived utilizing the SPT N-values encountered within the borings advanced for individual stability sections. The strength and unit weight parameters for these non-cohesive soil horizons were determined from published correlations between corrected blowcounts $(N_1)_{60}$, relative density and ϕ'. The corrected N-values were utilized to obtain relative densities based on relationships developed by Tokimatsu and Seed (1988) as shown in Figure 4 below. NAVFAC (1982) presents a relationship using relative density and of specific soil types to correlate angle of internal friction, unit weight, and void ratio as shown in Figure 4 below. The sandy silt to silty sand horizon contained SPT blowcounts in the range of weight of the sampling rods to 16, the unit weight of this soil horizon was estimated to vary between 105 to 113 pcf with a drained friction angle of 27° to 29°. The fine grained sand horizon contained SPT blowcounts in range of 4 to 60, the unit weight of this soil horizon was estimated to vary between 118 to 128 pcf with a drained friction angle of 31° to 36°.



From NAVFAC (1982)

From Tokimatsu and Seed (1988)

Figure 4. Charts Used to Correlate $(N_1)_{60}$ to ϕ'

Soil classifications for the correlations are based on laboratory testing results and field classifications performed during the drilling process. Once the relationships for the angle of internal friction, unit weight, and void ratio were established, the in situ unit weight was calculated based upon the natural moisture content. Refer to Table 9 below for a summary of derived soil parameters. Since a range is included for the sandy silt to silty sand and the fine grained sand layer refer to stability sections in Appendix F for specific alluvial deposit parameters.

Table 9. Selected Strength Parameters for Stability Analysis

Soil Horizon	Unit Weight (pcf)	Effective Stress Strength Parameters	
		c' (psf)	φ' (degrees)
Starter Clay Dike	129	0	30
Raised Clay Dike	125	0	30
Constructed Ash	93	0	30
Hydraulically Placed Ash	96	0	25
Gravel to Clayey Gravel	120	0	32
Sensitive Silt/Clay	127	0	28
Lean Clay Foundation Soil	129	0	30
Sandy Silt to Silty Sand	105 to 113	0	27 to 29
Fine Grained Sand to Sand with Silt	118 to 128	0	31 to 36

7.5. Slope Stability Analysis

The stability of the perimeter Dike C was evaluated using limit equilibrium methods as implemented in the SLOPE/W software, which is available from GEO-SLOPE International, Ltd., of Calgary, Alberta, Canada (www.geo-slope.com). Analyses were completed for static, long-term conditions with steady-state. SLOPE/W is a special-purpose computer code designed to analyze the stability of earth slopes using two-dimensional, limit equilibrium methods. With SLOPE/W, the distribution of pore water pressures within the earth mass can be mapped directly from a SEEP/W solution. In this study, steady-state pore pressures were obtained from the SEEP/W model described in Section 7.3. The unit weight and shear strength properties used in the stability analyses are detailed in Table 9 above.

7.5.1. Limit Equilibrium Methods in SLOPE/W

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

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

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

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

7.5.2. Slope Stability of Dike C

The slope stability analyses were carried out using SLOPE/W 2007 on the upstream and downstream face of Dike C, as applicable. SLOPE/W incorporates various search routines to locate the critical slip surface; for the analyses presented here, the "Grid and Radius" method and the "Entrance and Exit" method were employed. Center points for the trial circles were confined to a specified range above the slope surface, while the trial radii were varied based on tangent horizontal lines within the soil. The minimum and maximum range for the center points and tangent lines were parametrically varied over a wide range to determine the likely solution region for the critical circle. In subsequent runs, the search was refined by narrowing the range and spacing for the candidate center points.

Where the surface slope is composed of cohesionless (c' = 0) materials, an infinite slope failure (shallow sliding parallel to the surface) will be critical. While solutions were obtained for this case, as reported below, there is less concern for this potential failure mechanism. Suction pressures in unsaturated surface soils will often create enough apparent cohesion to

prevent this type of failure. If shallow sliding does occur, the resulting deformations are unlikely to threaten the integrity of the dike and can be repaired. To force the search routine to evaluate deeper failure mechanisms, the "Entrance and Exit" method was employed to force the slip surface to enter at a point that was approximately 15 feet from the pond.

The upstream surface of Dike C was evaluated at Station 119+69 and Station 132+37, where the dike slope was immediately adjacent to the stilling pond. For these analyses, the "Grid and Radius" method was used to determine the minimum factor of safety with a minimum depth of slice depth equal to 10 feet. At the other sections along the dike, the ash in the pond has accumulated to a height approaching the crest of Dike C. At those locations, the upstream dike face is supported by the ash and there is negligible risk for an upstream failure. Therefore, the upstream stability of Dike C along the ash pond was not analyzed further.

8. Results

8.1. Seepage Gradients

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

Considering the SEEP/W results in Appendix H, the predicted phreatic surface is observed to intersect the sloping ground surface above the lake level near the toe of the starter dike. With the exception of Station 119+69, this condition is predicted for all cross sections analyzed. Ground water seeping through the saturated dike materials may be flowing out to the ground surface, even though direct observations might be obscured by vegetation, evaporation, or the submerged ground surface. In these locations, the seepage forces associated with the hydraulic exit gradients are acting in the same direction as gravity. Because of the high potential for initiating the movement of soil particles and piping, a condition of groundwater seeping to the sloping surface of the downstream face is usually considered unacceptable in the evaluation of earth dams.

The potential for piping due to vertical seepage to the ground surface was also evaluated using the factor of safety defined in Section 7.3.6. First, contour plots of vertical gradient (Appendix H) were examined to determine the general location of the maximum vertical exit gradient. In all five cross sections, the maximum upward gradient occurs near the toe of the starter dike.

For the factor of safety calculations, average vertical gradients were determined over a depth of 3 to 5 feet just below the ground surface. On the sections at Station 108+93, 119+69, and 149+14, these gradients were computed across the upper few feet of the foundation soil deposit. In the other two sections that were modeled, at Station 132+37 and 138+27, the maximum computed gradients occur at the very toe of the clay starter dike. The model

geometry converges to a sharp point at this location, such that the computed gradients in this small area are not reflective of the actual conditions in the field. However, to evaluate the potential for heaving of the dike toe in this area, gradients were taken across a thickness of no less than 3 feet.

The factors of safety against piping (Section 7.3.6), computed based on the exit gradients computed in the SEEP/W and the critical gradients determined from the soil properties, are summarized in Table 10. The lowest computed factor of safety is 1.3 on the section at Station 132+37. TVA does not currently have guidelines for FS piping. United States Army Corps of Engineers (USACE) design criteria (EM 1110-2-1901) indicates factors of safety against piping should be at least three. Hence, at some location on all five modeled cross sections, Dike C does not meet the design criteria for piping at the seepage exits.

Table 10. Summary of Computed Exit Gradients and Factors of Safety against Piping

Cross Section*	Vertical Gradient (i_v) at Critical Exit Point	Location of Critical Exit Point	Material	Critical Gradient (i_{crit})	FS_{piping}
108+93	0.15 – 0.38	Toe	Silty Sand to Sandy Silt	1.03	2.7
119+69	0.70 – 0.74	Toe	Lean Clay	1.05	1.4
132+37	0.73 – 0.84	Toe	Clay Starter Dike	1.05	1.3
138+27	0.30 – 0.42	Toe	Constructed Ash	0.71	1.7
149+14	0.45 – 0.50	Toe	Silty Sand to Sandy Silt	1.03	2.1

*Refer to Appendix F for Plan View of site with project baseline

8.2. Slope Stability Results

Using the strength parameters selected (c' and ϕ') listed in Table 9, in conjunction with the results of the seepage analyses, the existing dike configuration was analyzed at each of the five cross sections. Geo-Slope's Slope/W computer program was used for the analyses with pore pressures imported from the seepage analyses. Long term (effective stress), steady state seepage conditions were analyzed using Spencer's method. For the Spencer's method analyses, circular failure surfaces with optimization were conducted. Minor details of the geometry, such as various rip rap zones, were not represented in the stability model.

The stability analyses focused on the potential for failure in the upstream slope and downstream dike face. SLOPE/W failure surfaces from these analyses are presented on the drafted sheets in Appendix F. The results are summarized in Table 10 below.

Table 11. Summary of Computed Factors of Safety for Slope Stability

Location*	Deep Seated Slide	Shallow Slide in Starter Dike	Shallow Slide in Raised Dike	Upstream Slide
108+93	1.66	1.42	1.39	N/A
119+69	1.48	1.25	1.39	1.78
132+37	1.47	1.24	1.22	1.33
138+27	1.52	1.13	1.38	N/A
149+14	1.49	1.15	1.35	N/A

*Refer to Appendix F for plan view of site with project baseline

The Tennessee Department of Environment and Conservation (TDEC) "Rules and Regulations Applied to the Safe Dams Act of 1973" provides guidance and standards with regards to existing dams. The standards do not specifically address target factors of safety for slope stability, but instead merely indicate that the dam shall be "stable". 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 USACE Manual EM 1110-2-1902 "Slope Stability".

Considering only potential deep-seated failure mechanisms that would immediately impact the crest, the slope stability results show that Dike C meets this criteria. The surfaces were generated using the "Entrance and Exit" method to force the slip surface to enter the dike approximately 15 feet from the ponds. The difference between the minimum computed value of 1.47 and the required value of 1.5 is negligible, considering the uncertainty in the characterization of the soil strength. These results suggest that Dike C is stable with respect to the potential formation of large, deep-seated failures that would immediately compromise the retention of the ash pond. The critical slip surfaces are depicted in Appendix F.

At the same time, the results clearly show factors of safety less than 1.5 for shallow sliding on the downstream face along the full length of Dike C (plus the upstream face in the vicinity of Station 132+37). The shallow potential failure surfaces, which are generally about 10 ft below the ground surface, occur in the slopes of both the lower starter dike and the upper dike raise (see the cross sections in Appendix F). The lowest factors of safety are currently found along the eastern branch of Dike C, where factors of safety less than 1.2 were computed. Based on established design criteria, remedial measures are needed to increase the long-term factors of safety against shallow sliding to values of at least 1.5. Mitigation is needed on the downstream face (both the lower starter dike and the upper dike raise) along nearly the full length of Dike C, and in more limited areas on the upstream (pond side) slope.

Past shallow sliding of the downstream face of Dike C was reported in the project records, in an area along the perimeter of the dredge cell. This surficial slide was observed and repaired by TVA as part of their routine maintenance program. If additional shallow sliding were to develop again in Dike C, the failure would be initially confined to the sloping face of the dike. If not repaired and given enough time, these shallow slides could progress up the slope and endanger the ash pond. Because this progressive failure mechanism would be expected to

take months or longer to impact the crest, a robust monitoring program can reduce the risk of failure due to shallow sliding. Hence, in the interim while a permanent mitigation plan is developed, analyzed, and constructed, TVA should continue to monitor Dike C with routine inspections and instrument readings.

There was no indication in the slope stability analyses that a noncircular failure surface would give a factor of safety lower than obtained for 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, curved slip surfaces. The results in Table 11 and Appendix F represent factors of safety computed from the optimized, circular slip surface routine.

A review of the stability analysis results indicates that while none of the projected failure surfaces represent true global failures of the dike system, it is likely that some of the modeled failures would subsequently progress toward the pond and could result in an eventual breach. Remedial engineering designs will need to be implemented to improve dike slope stability to meet USACE design criteria.

8.2.1. Conceptual Mitigation Evaluation

To aid in identifying potential mitigation options available for enhancing stability of the dike system, preliminary analyses of a slope buttress system were performed on each cross section. These preliminary analyses were performed to provide some indication of the level of mitigation needed to achieve factors of safety greater than or equal to 1.5 at each of the sections analyzed. The concept involved adding various thicknesses of rock to the outslopes on the starter dike and raised dike.

Based on the results of this conceptual mitigation analyses, achieving a factors of safety greater than or equal to 1.5 will ultimately require placement of a zone of a of rock blanket. The rock blanket starting from the crest of the raised could have a maximum regrade of 2½:1 (H:V) on the downstream face of the raised dike. It would then transition to a horizontal bench near the crest of the starter dike and extend to the river bank.

9. Limitations of Study

The scope of this evaluation was limited to consider only the potential risks at Dike C 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 Phase 2 facility assessment that is currently underway.

The stability of Dike C during a potential earthquake was not specifically analyzed. Data from the site explorations indicate low penetration resistance (low density) in saturated, sandy alluvium. In a strong earthquake, these foundation soils will be prone to liquefaction, which would undermine the stability of the dike. However, the seismic risk at this site (likelihood of experiencing a large magnitude earthquake) is quite low while the pond is in the conversion process to dry ash handling.

Stability analyses were not performed for rapid drawdown conditions:

- On the upstream side, a rapid drawdown condition would correspond to a failure of the ash pond, perhaps due to a breach in the dike or failure of the spillway. While the upstream dike slope may be vulnerable to sliding due to rapid drawdown, this mechanism would result from, and not cause, a pond failure. Hence, there is no need to evaluate the upstream slope for rapid drawdown.

- On the downstream side, rapid drawdown conditions would correspond to changes in the elevation of Watts Bar Lake. Preliminary information on the river conditions indicate that the lake might rise 11 ft, from a normal winter pool to 100-year flood stage, over a period of one to two days. Per the operational plan for Watts Bar Lake, the flood pool would then be drawn down over a period of seven days. Hence, high water conditions would not persist long enough to fully charge the dike slopes, and a fully undrained, rapid drawdown loading condition is unlikely to develop.

- Watts Bar Lake is also subject to annual drawdown from the normal summer pool (elevation 742 feet) to the normal winter pool (elevation 737 feet). Because the outslopes do not currently meet design criteria for long-term conditions, this rapid drawdown case was not analyzed in this study. Future engineering work to support the design of dike improvements should consider and evaluate the possibility of a rapid drawdown failure.

10. Conclusions

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

The results from the seepage analyses were examined to identify conditions were piping and erosion of soil might develop due to seepage forces. The model results indicated a shallow phreatic surface (ground water table) within the starter dike; these results are generally confirmed by the observation of shallow water in areas were trees and stumps have been removed from the starter dike. On four of the five cross sections, the model indicated seepage flows to the sloping downstream face of the starter dike. This condition creates the potential for the initiation of soil piping, as seepage water will tend to erode material from the dike face at these locations.

Upward, vertical exit gradients in the area of the dike toe were also found to be excessive. Factors of safety against piping, computed for the surficial 3 to 5 feet of soil in these areas, ranged from 1.3 to 2.7. 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 Dike C does not meet current criteria for preventing soil piping due to seepage.

Current criteria for the long-term stability of Dike C requires a factor of safety for slope stability of at least 1.5. Considering only potential deep-seated failure mechanisms that would immediately impact the crest, the slope stability results show that Dike C meets this criteria. The difference between the minimum computed value of 1.47 and the required value of 1.5 is negligible, considering the uncertainty in the characterization of the soil strength. These results suggest that Dike C is stable with respect to the potential formation of large, deep-seated failures that would immediately compromise the retention of the ash pond.

At the same time, the results clearly show factors of safety less than 1.5 for shallow sliding on the downstream face along the full length of Dike C (plus the upstream face in the vicinity of Station 132+37). The shallow potential failure surfaces, which are generally about 10 ft below the ground surface, occur in the slopes of both the lower starter dike and the upper dike raise (see the cross sections in Appendix F). The lowest factors of safety are currently found along the eastern branch of Dike C, where factors of safety less than 1.2 were computed. Based on established design criteria, remedial measures are needed to increase the long-term factors of safety against shallow sliding to values of at least 1.5. Mitigation is needed on the downstream face (both the lower starter dike and the upper dike raise) along nearly the full length of Dike C, and in more limited areas on the upstream (pond side) slope.

Past shallow sliding of the downstream face of Dike C was reported in the project records, in an area along the perimeter of the dredge cell. This surficial slide was observed and repaired by TVA as part of their routine maintenance program. If additional shallow sliding were to develop again in Dike C, the failure would be initially confined to the sloping face of the dike. If not repaired and given enough time, these shallow slides could progress up the slope and endanger the ash pond. Because this progressive failure mechanism would be expected to take months or longer to impact the crest, a robust monitoring program can reduce the risk of failure due to shallow sliding. Hence, in the interim while a permanent mitigation plan is developed, analyzed, and constructed, TVA should continue to monitor Dike C with routine inspections and instrument readings.

Based on the design criteria, remedial measures are needed to increase the factors of safety against shallow sliding on the downstream face along nearly the full length of Dike C. The lowest factors of safety are currently found along the eastern branch of Dike C, where factors of safety less than 1.2 were computed for sliding about 10 ft below the surface of the starter dike (see the cross sections in Appendix F). Along the southern reaches of the dike, marginal factors of safety against shallow sliding were computed on the downstream face of the dike raise.

11. Recommendations

The current configuration of the pond dikes do not exhibit acceptable factors of safety for long-term stability. While this does not imply that the dike is in immediate danger of failure, TVA should undertake specific efforts to improve the safety of this facility. The following specific actions are recommended:

11.1. To improve the long-term stability of Dike C, TVA should initiate a mitigation design and construction program as soon as possible. The mitigation project should involve the placement of stabilizing berms on the downstream face of the dike, along the full length, and continued monitoring to ensure the structure's safety while permanent improvements are being completed.

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

11.3. The berm should also be designed to provide protection against seepage and piping failures, and increase the factor of safety against piping to meet the design guideline value of 3. Where the berm is built over areas subject to exit seepage (crest and downstream face of the starter dike), the gradation of the berm should be selected to filter the underlying material.

11.4. The outer surface of the berm should be stone rip rap, which will provide protection against erosion from surface runoff and wave action along the water line of Watts Bar Lake. A zoned berm cross section, consisting of multiple layers of sand to stone, may be needed to meet filter criteria for protecting the slope toe against piping.

11.5. Between now and the completion of the stabilizing berms, TVA should implement engineering solutions to reduce the risk of failure in Dike C. This should include continuation of the daily inspection program and instrumentation monitoring program, with renewed focus on identifying the initiation of potential surface slides. Additional inclinometers should be installed in critical areas of Dike C, positioned on the basis of the slope stability analyses reported herein. The inclinometers should be read on a weekly basis. This monitoring program be continued until permanent improvements to the dike have been completed to address long term stability.

11.6. Lowering the water levels in the ash pond and stilling basin would lessen the potential for failure due to seepage and piping under the dike, and might improve slope stability. Operating the ponds at lower water levels should be considered as an option in the overall mitigation plan for Dike C.

11.7. The surface of the dike faces, especially along the mid-slope bench (crest of the initial starter dike) should be re-graded to improve drainage and prevent the accumulation of surface water on the dike.

12. Closure

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

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

12.3. It should be noted that construction records indicating the methods used to construct the dikes, as-built dike 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.

13. References

The following is a list of historical documents that were used to evaluate the stability of Dike C and the Road Dike:

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

Kingston Fossil Plant Annual Ash Pond Dike Stability Inspection, TVA Engineering Design Services, 1967 to 2009

Additional Documents:

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

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

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

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

Lambe, T.W and Whitman, R.V. Soil Mechanics. 3rd ed. New York: John Wiley and Sons. 1969

Rawls, W. J., Brakensiek, D. L., and Saxton, K. E. (1982). "Estimation of soil water properties," Transactions of the American Society of Agricultural Engineers. pp. 1316-1320.

Salgado, R. The Engineering of Foundations. Boston: McGraw Hill. 2008.

Appendix F

Boring Layout, Stability Sections, Profile Sheets and Logs of Borings

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NOTES

1. Topographic mapping was generated from a LIDAR survey performed on January 8, 2009 by Tuck Mapping Solutions, Inc. This plan was developed for discussion purposes only and is not to be used for construction.
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.

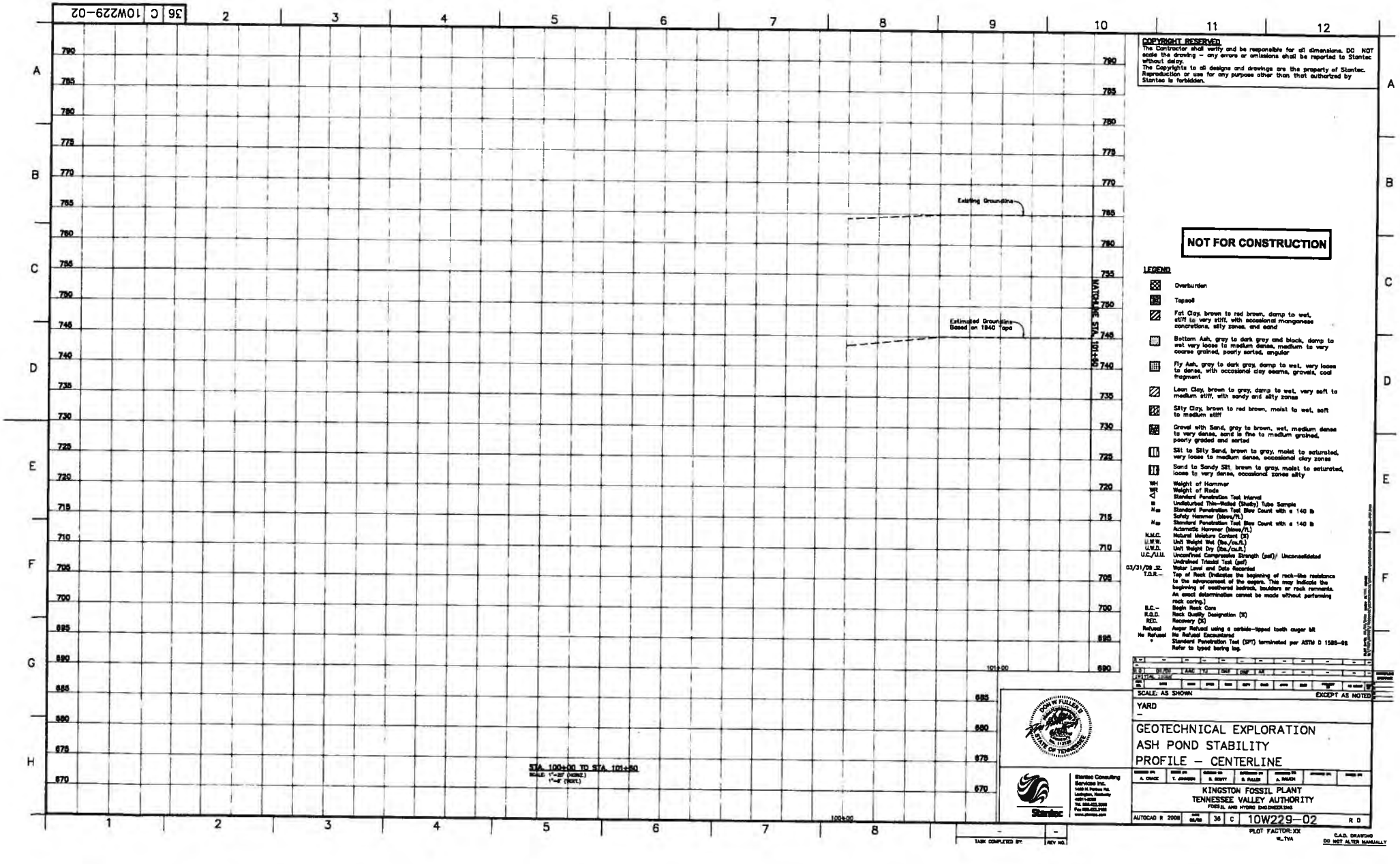
LEGEND

- Soil Boring with Undisturbed (Shelby) Tube Samples and/or Standard Penetration Tests
- Soil Boring with Undisturbed (Shelby) Tube Samples and/or Standard Penetration Tests and Rock Core
- Previously Drilled by AECOM

NOT FOR CONSTRUCTION

BORING LOCATION TABLE

BORING	NORTHING	EASTING	ELEV. (FT.)	BORING TYPE
STN-1	1,021,830.27	1,441,111.82	742.5	Sample
				P.S.T.
STN-2	1,021,830.27	1,441,111.82	742.5	Core Penetration Test
				Sample
				P.S.T. P.S.T.
STN-3	1,021,830.27	1,441,111.82	742.5	P.S.T.
				Sample
				Sample
STN-4	1,021,830.27	1,441,111.82	742.5	Core Penetration Test
				Sample and Core
				Sample
STN-5	1,021,830.27	1,441,111.82	742.5	Core Penetration Test
				Sample
STN-6	1,021,830.27	1,441,111.82	742.5	Sample and Core / Slope Inclinometer
				Core Penetration Test
				Sample
STN-7	1,021,830.27	1,441,111.82	742.5	Core Penetration Test
				Sample
STN-8	1,021,830.27	1,441,111.82	742.5	Sample and Core
				Sample
				P.S.T.
STN-9	1,021,830.27	1,441,111.82	742.5	Core Penetration Test
				Sample
STN-10	1,021,830.27	1,441,111.82	742.5	P.S.T. P.S.T.
				Sample
				Sample
STN-11	1,021,830.27	1,441,111.82	742.5	Sample and Core / Slope Inclinometer
				Core Penetration Test
				Sample
STN-12	1,021,830.27	1,441,111.82	742.5	Core Penetration Test
				Sample
STN-13	1,021,830.27	1,441,111.82	742.5	Sample and Core
				Sample and Core
				Sample
STN-14	1,021,830.27	1,441,111.82	742.5	Sample
				P.S.T.
STN-15	1,021,830.27	1,441,111.82	742.5	Core Penetration Test
				Sample
STN-16	1,021,830.27	1,441,111.82	742.5	Sample / P.S.T. P.S.T.
				Sample / P.S.T.
STN-17	1,021,830.27	1,441,111.82	742.5	Core Penetration Test
				Sample
STN-18	1,021,830.27	1,441,111.82	742.5	Sample
				Core Penetration Test
STN-19	1,021,830.27	1,441,111.82	742.5	Sample
				Core Penetration Test
STN-20	1,021,830.27	1,441,111.82	742.5	Sample
				Core Penetration Test
STN-21	1,021,830.27	1,441,111.82	742.5	Sample
				Core Penetration Test
STN-22	1,021,830.27	1,441,111.82	742.5	Sample
				P.S.T. P.S.T.
STN-23	1,021,830.27	1,441,111.82	742.5	Core Penetration Test
				Sample
STN-24	1,021,830.27	1,441,111.82	742.5	Sample and Core
				Core Penetration Test
STN-25	1,021,830.27	1,441,111.82	742.5	Sample
				Core Penetration Test
STN-26	1,021,830.27	1,441,111.82	742.5	Sample
				Core Penetration Test
STN-27	1,021,830.27	1,441,111.82	742.5	Sample
				Core Penetration Test



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NOT FOR CONSTRUCTION

LEGEND

- Overburden
- Topsoil
- Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
- Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
- Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
- Lean Clay, brown to gray, damp to wet, very soft to medium stiff, with sandy and silty zones
- Silty Clay, brown to red brown, moist to wet, soft to medium stiff
- Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
- Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH – Weight of Hammer
WR – Weight of Rods
SPT – Standard Penetration Test Interval
ST – Undisturbed Thin-Walled (Shelby) Tube Sample
N60 – Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N140 – Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
M.C. – Natural Moisture Content (%)
U.W.W. – Unit Weight Wet (lbs./cu.ft.)
U.W.D. – Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. – Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/08 ∑ – Water Level and Date Recorded
T.O.R. – Top of Rock (Indicates the beginning of rock-like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C. – Begin Rock Core
R.Q.D. – Rock Quality Designation (%)
REC. – Recovery (%)
Refusal – Auger Refusal using a carbide-tipped tooth auger bit
No Refusal – No Refusal Encountered
* – Standard Penetration Test (SPT) terminated per ASTM D 1586-98. Refer to typed boring log.

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TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

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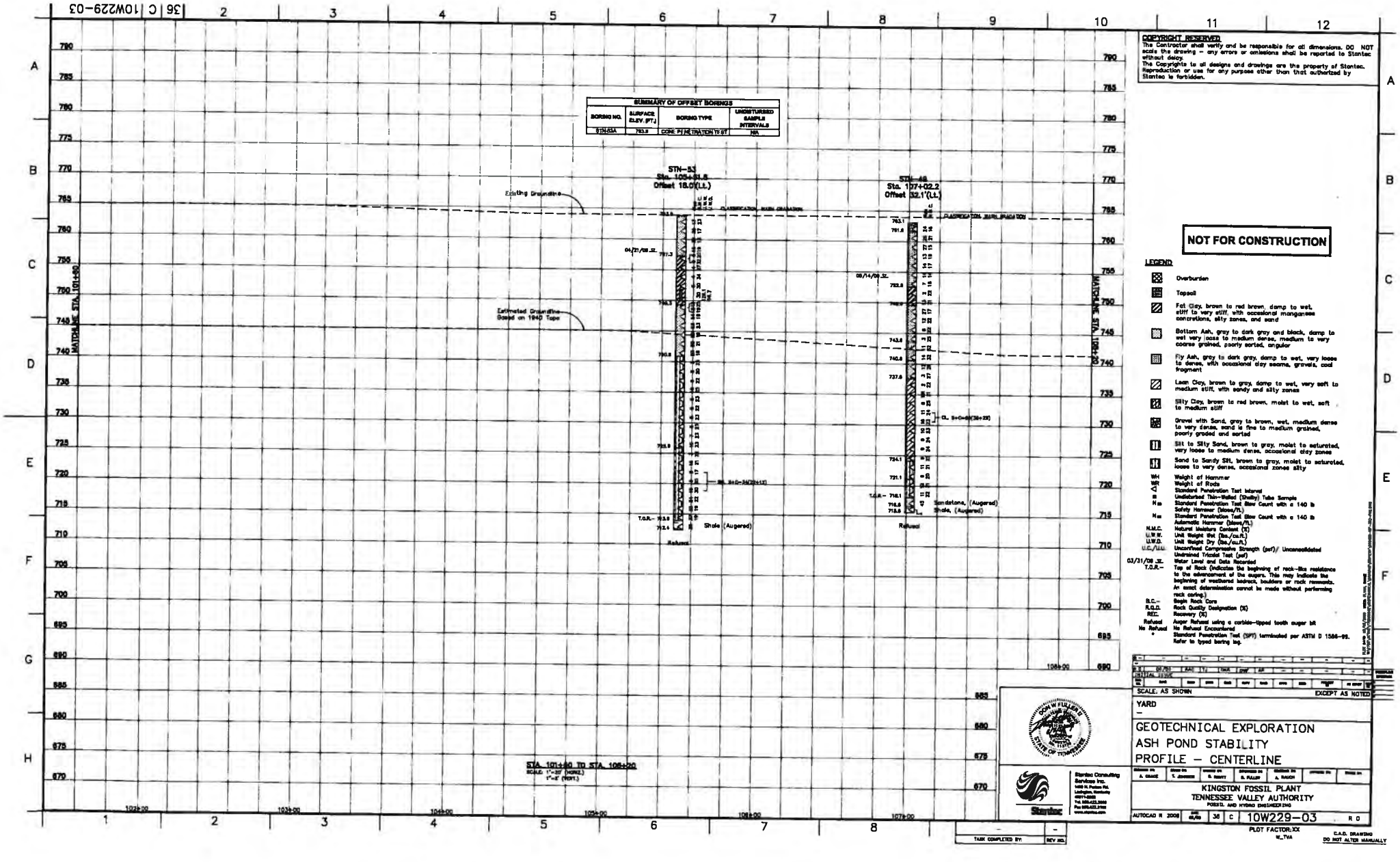
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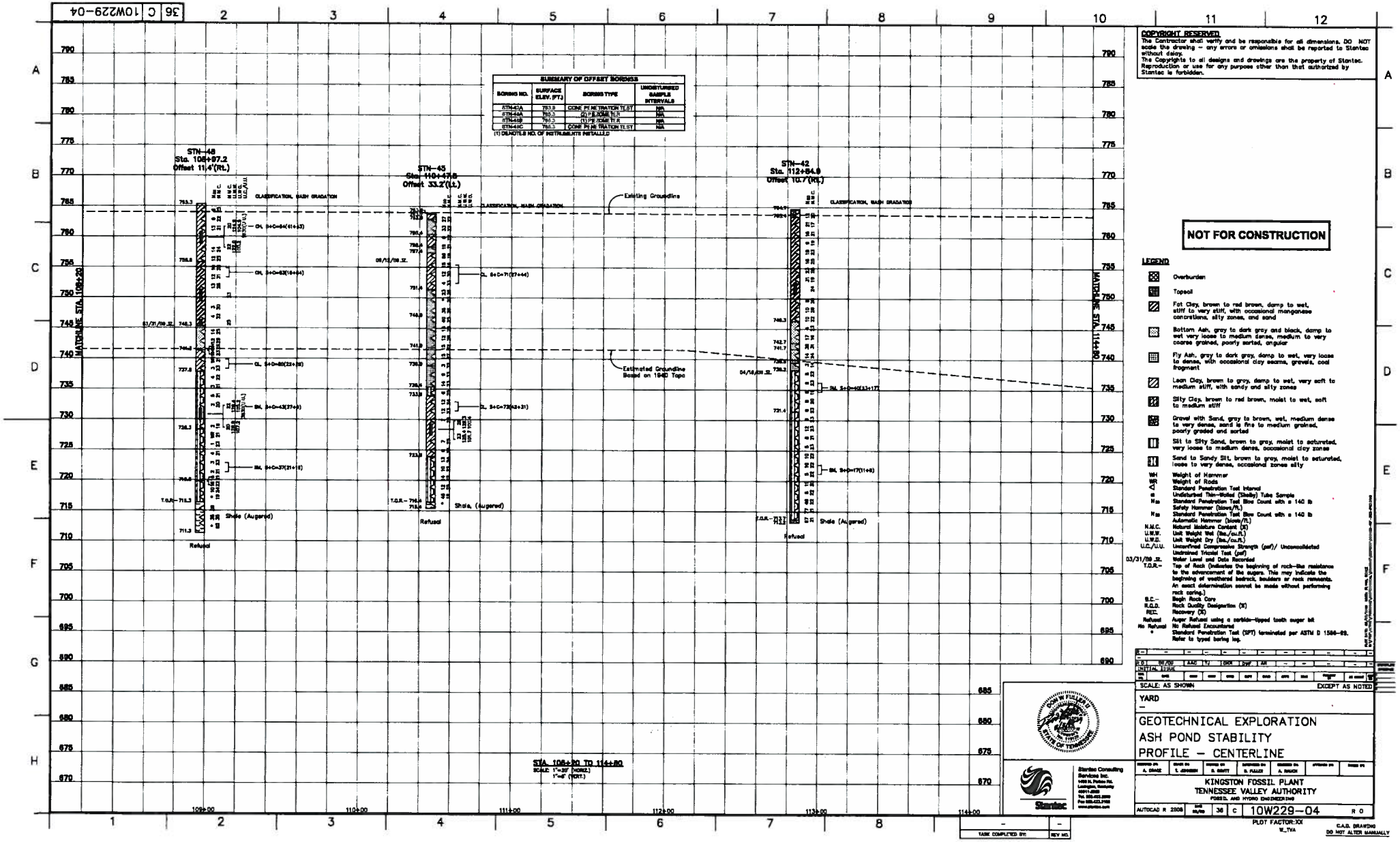
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37771-6200
Tel. 865-922-3000
Fax. 865-922-3990
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36 | C | 10W229-02


36 | C | 10W229-03
SUMMARY OF OFFSET BORINGS
BORING NO. | SURFACE ELEV. (FT.) | BORING TYPE | UNDISTURBED SAMPLE INTERVALS
B13-53A | 783.9 | CORE PENETRATION TEST | NA
STN-53
Sta. 105+91.5
Offset 18.0'(Lt.)
STN-48
Sta. 107+02.2
Offset 32.1'(Lt.)
Existing Groundline
Estimated Groundline Based on 1940 Tape
Shale (Augered)
Refusal
Sandstone, (Augered)
Shale, (Augered)
Refusal
STA. 101+80 TO STA. 108+00
NOT FOR CONSTRUCTION
LEGEND
Overburden
Topsoil
Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
Lean Clay, brown to gray, damp to wet, very soft to medium stiff, with sandy and silty zones
Silty Clay, brown to red brown, moist to wet, soft to medium stiff
Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty
WH Weight of Hammer
WR Weight of Rods
SPT Standard Penetration Test Interval
U Undisturbed Thin-Walled (Shelby) Tube Sample
N60 Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N60 Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. Natural Moisture Content (%)
U.W.W. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./UU Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/08 ▽ Water Level and Date Recorded
T.O.R.– Top of Rock (indicates the beginning of rock–like resistance to the advancement of the auger. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C.– Begin Rock Core
R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal Auger Refusal using a carbide–tipped tooth auger bit
No Refusal No Refusal Encountered
* Standard Penetration Test (SPT) terminated per ASTM D 1586–99. Refer to typed boring log.
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SUMMARY OF OFFSET BORINGS

BORING NO.	SURFACE ELEV. (FT.)	BORING TYPE	UNDISTURBED SAMPLE INTERVALS
STN-42A	783.9	CONE PENETRATION TEST	N/A
STN-44A	780.3	CONE PENETRATION TEST	N/A
STN-44B	780.3	(1) PENETROMETER	N/A
STN-46C	786.3	CONE PENETRATION TEST	N/A

(1) DENOTES NO. OF INSTRUMENTS INSTALLED

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NOT FOR CONSTRUCTION

LEGEND

- Overburden
- Topsoil
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- Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
- Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
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- Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
- Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH Weight of Hammer
WR Weight of Rods
Δ Standard Penetration Test Interval
ST Undisturbed Thin-Walled (Shelby) Tube Sample
Nss Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
Nas Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. Natural Moisture Content (%)
U.W.W. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 ∇ Water Level and Date Recorded
T.O.R.– Top of Rock (Indicates the beginning of rock–like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C.– Begin Rock Core
R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal Auger Refusal using a carbide-tipped tooth auger bit
No Refusal No Refusal Encountered
* Standard Penetration Test (SPT) terminated per ASTM D 1586-99. Refer to typed boring log.

SCALE: AS SHOWN EXCEPT AS NOTED

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TASK COMPLETED BY: – REV NO. –

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS			
Location	STN-37		
Depth	13.5'–14.0'	14.3'–14.8'	20.0'–20.5'
ϕ	41.1°		
c	40 p.s.f.		
Location	STN-3D	STN-3D	STN-37D
Depth	31.0'–31.6'	31.6'–32.2'	31.6'–32.2'
ϕ	35.6°		
c	113 p.s.f.		

SUMMARY OF OFFSET BORINGS			
BORING NO.	SURFACE ELEV. (FT.)	BORING TYPE	UNDISTURBED SAMPLE INTERVALS
STN-37A	763.9	(2) PIEZOMETER	NA
STN-37B	763.7	(1) PIEZOMETER	NA
STN-37C	763.6	SAMPLE	22.5-24.0 24.5-26.0 27.5-29.5 29.5-31.5 31.5-33.5
STN-37D	763.5	SAMPLE	22.7-24.7 24.7-26.9 28.0-29.4 31.6-32.0
STN-37E	763.8	CONE PENETRATION TEST	NA

(1) DENOTES NO. OF INSTRUMENTS INSTALLED

STN-38
Sta. 117+71.0
Offset 15.0'(RT.)

STN-37
Sta. 118+78.4
Offset 10.1'(RT.)

Existing Groundline

Estimated Groundline Based on 1940 Topo

MATCH LINE STA. 114+50

MATCH LINE STA. 121+00

Shale, (Augered)

Refusal

STA. 114+50 TO STA. 121+50
SCALE: 1"=20' (HORIZ.)
1"=5' (VERT.)

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- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH – Weight of Hammer
WR – Weight of Rods
SPT – Standard Penetration Test Interval
U – Undisturbed Thin-Walled (Shelby) Tube Sample
Nm – Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
Nc – Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. – Natural Moisture Content (%)
U.W.W. – Unit Weight Wet (lbs./cu.ft.)
U.W.D. – Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. – Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/08 ▽ – Water Level and Date Recorded
T.O.R. – Top of Rock (indicates the beginning of rock-like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C. – Begin Rock Core
R.Q.D. – Rock Quality Designation (%)
REC. – Recovery (%)
Refusal – Auger Refusal using a carbide-tipped tooth auger bit
No Refusal – No Refusal Encountered
* – Standard Penetration Test (SPT) terminated per ASTM D 1586-99. Refer to typed boring log.

SCALE: AS SHOWN
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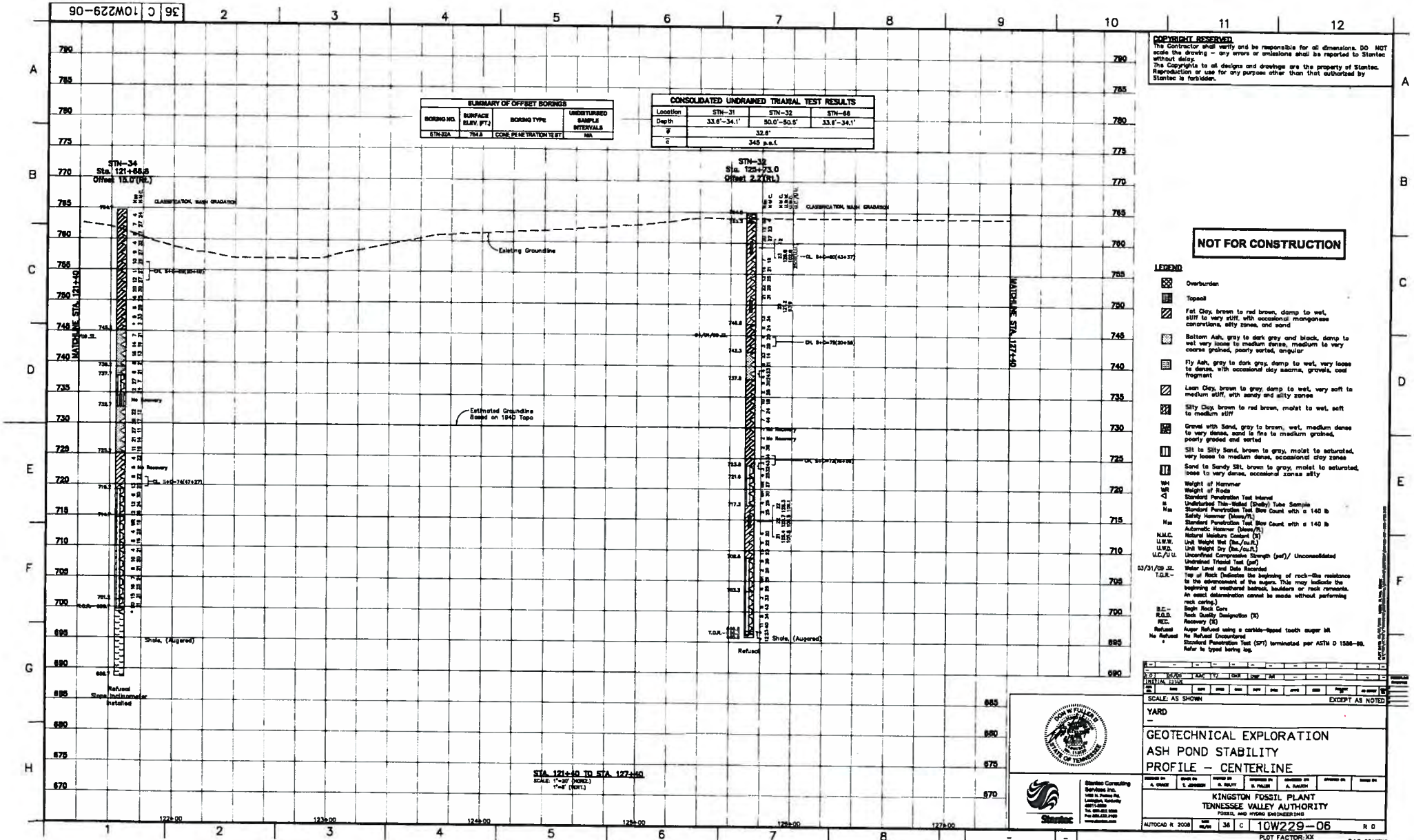
AUTOCAD ® 2006 | 36 | C | 10W229-05 | R 0

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1409 N. Forbes Rd.
Lexington, Kentucky
40511-2050
Tel. 859-422-3000
Fax 859-422-3100
www.stantec.com

TASK COMPLETED BY: – REV NO. –



SUMMARY OF OFFSET BORINGS			
BORING NO.	SURFACE ELEV. (FT.)	BORING TYPE	UNDISTURBED SAMPLE INTERVALS
STN-32A	764.8	CONE PENETRATION TEST	NA

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS			
Location	STN-31	STN-32	STN-68
Depth	33.6'-34.1'	50.0'-50.5'	33.6'-34.1'
ϕ	32.8°		
c	345 p.s.f.		

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NOT FOR CONSTRUCTION

LEGEND

- Overburden
- Topsoil
- Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
- Bottom Ash, gray to dark grey and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
- Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
- Lean Clay, brown to gray damp to wet, very soft to medium stiff, with sandy and silty zones
- Silty Clay, brown to red brown, moist to wet, soft to medium stiff
- Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
- Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH — Weight of Hammer
WR — Weight of Rods
SPT — Standard Penetration Test Interval
U — Undisturbed Thin-Walled (Shelby) Tube Sample
N60 — Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N60 — Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. — Natural Moisture Content (%)
U.W.W. — Unit Weight Wet (lbs./cu.ft.)
U.W.D. — Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. — Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 ∇ — Water Level and Date Recorded
T.O.R. — Top of Rock (Indicates the beginning of rock—the resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C. — Begin Rock Core
R.Q.D. — Rock Quality Designation (%)
REC. — Recovery (%)
Refusal — Auger Refused using a carbide-tipped tooth auger bit
No Refusal — No Refusal Encountered
* — Standard Penetration Test (SPT) terminated per ASTM D 1586-99. Refer to typed boring log.

SCALE: AS SHOWN
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GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – CENTERLINE

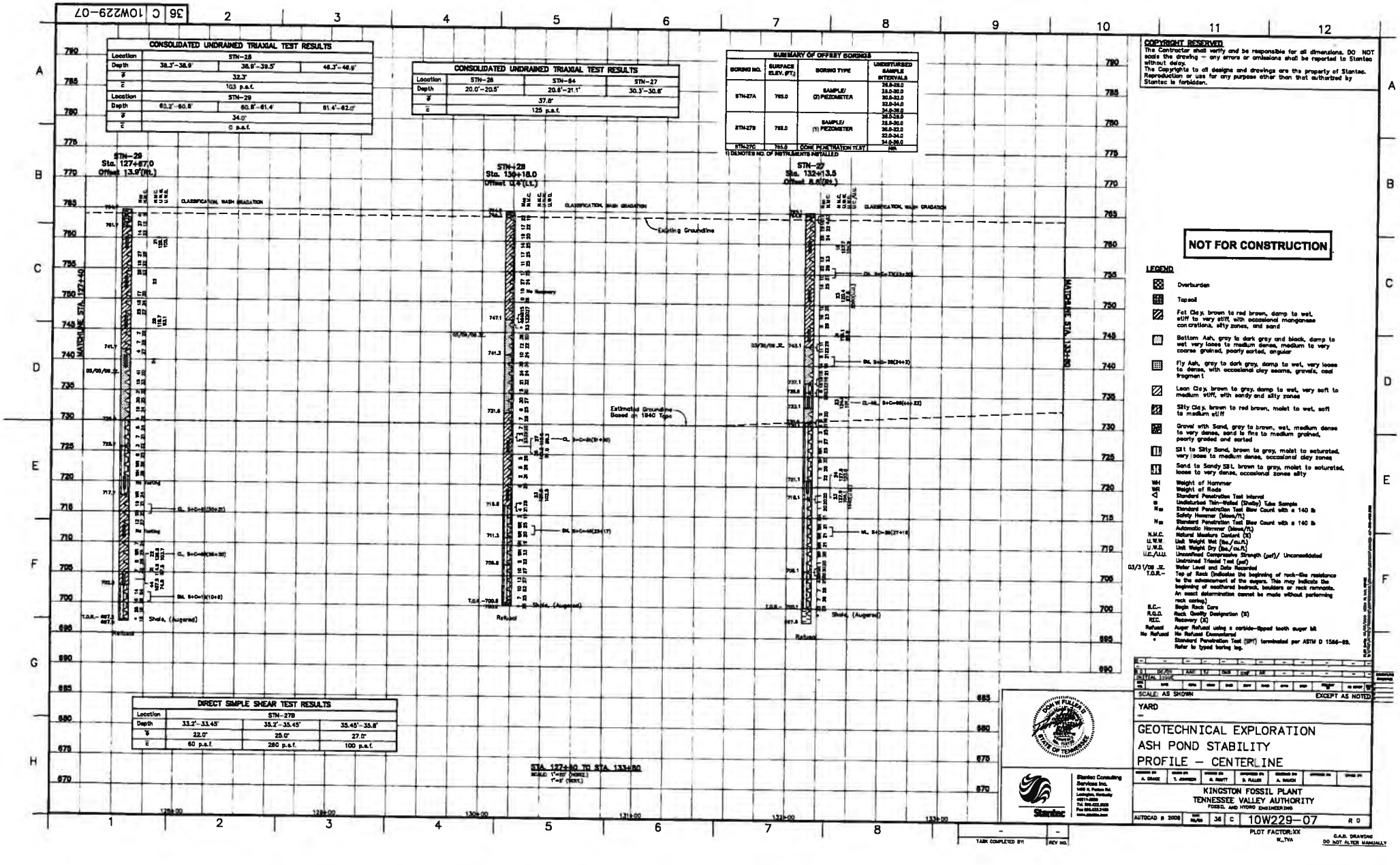
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

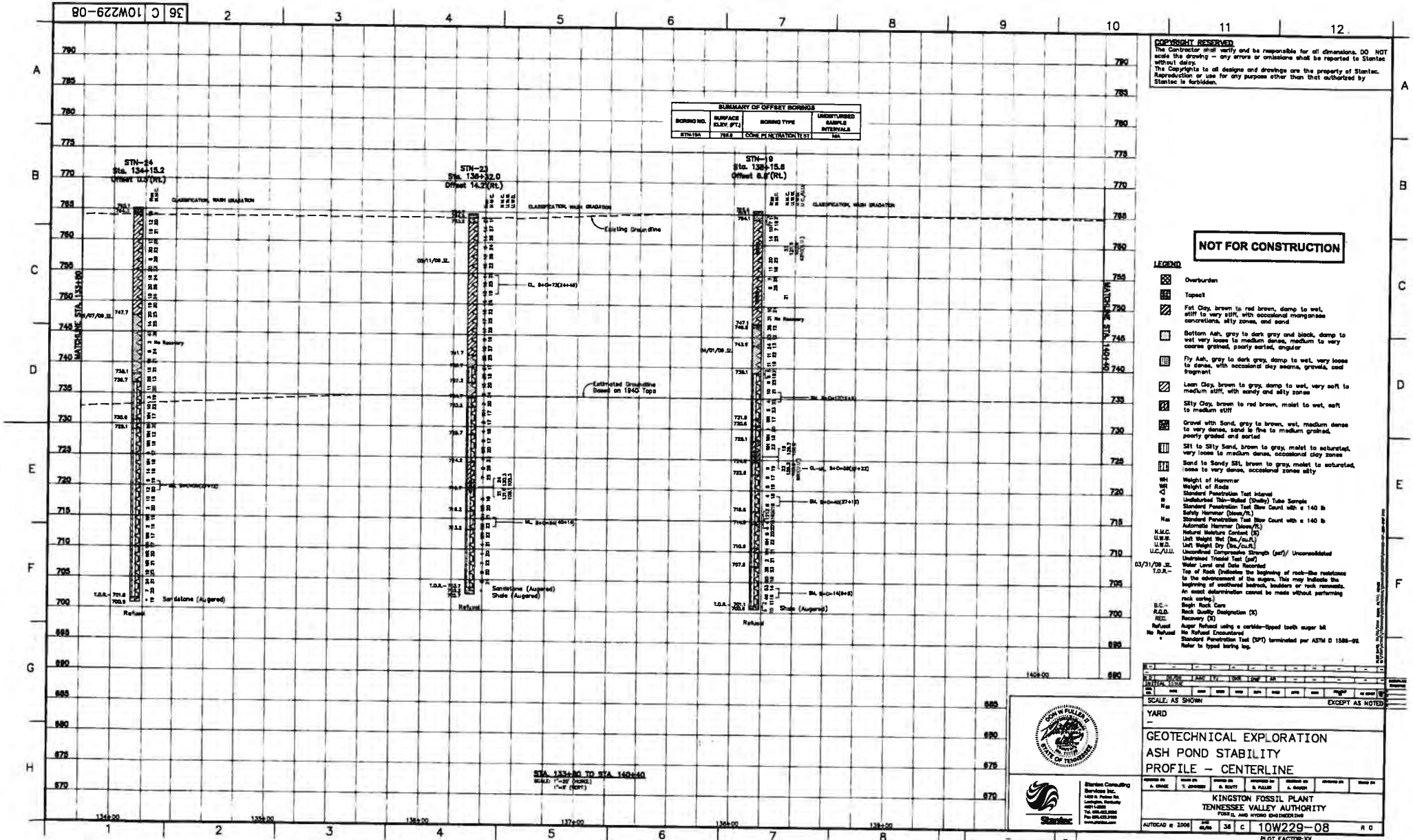
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36 C 10W229-07
CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS
Location STN-28
Depth 38.3'-38.9' 38.9'-39.5' 48.3'-48.9'
ϕ 32.3°
c 103 p.s.f.
Location STN-29
Depth 60.2'-60.8' 60.8'-61.4' 61.4'-62.0'
ϕ 34.0°
c 0 p.s.f.
CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS
Location STN-26 STN-84 STN-27
Depth 20.0'-20.5' 20.6'-21.1' 30.3'-30.8'
ϕ 37.8°
c 125 p.s.f.
SUMMARY OF OFFSET BORINGS
BORING NO. SURFACE ELEV. (FT.) BORING TYPE UNDISTURBED SAMPLE INTERVALS
STN-27A 765.0 SAMPLE/ (2) PIEZOMETER 25.0-27.0 28.0-30.0 30.0-32.0 32.0-34.0 34.0-36.0
STN-27B 752.0 SAMPLE/ (1) PIEZOMETER 26.0-28.0 28.0-30.0 30.0-32.0 32.0-34.0 34.0-36.0
STN-27C 765.0 CONE PENETRATION TEST NA
1) DENOTES NO. OF INSTRUMENTS INSTALLED
STN-29
Sta. 127+87.0
Offset 13.9'(Rt.)
STN-28
Sta. 130+18.0
Offset 0.4'(Lt.)
STN-27
Sta. 132+13.5
Offset 8.8'(Rt.)
CLASSIFICATION, WASH GRADATION
MATCHLINE STA. 127+40
MATCHLINE STA. 133+80
Existing Groundline
Estimated Groundline Based on 1940 Topo
Shale, (Augered)
Refusal
No Recovery
No Refusal
DIRECT SIMPLE SHEAR TEST RESULTS
Location STN-27B
Depth 33.2'-33.45' 35.2'-35.45' 35.45'-35.8'
ϕ 22.0° 25.0° 27.0°
c 60 p.s.f. 280 p.s.f. 100 p.s.f.
STA. 127+40 TO STA. 133+80
SCALE: 1"=50' (HORIZ.)
1"=5' (VERT.)
1290-00
1300-00
1310-00
1320-00
1330-00
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NOT FOR CONSTRUCTION
LEGEND
Overburden
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Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
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Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
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Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
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WH Weight of Hammer
WR Weight of Rods
Standard Penetration Test Interval
Undisturbed Thin-Walled (Shelby) Tube Sample
Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. Natural Moisture Content (%)
U.W.W. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
Water Level and Date Recorded
T.O.R.- Top of Rock (Indicates the beginning of rock-like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C.- Begin Rock Core
R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal Auger Refusal using a carbide-tipped tooth auger bit
No Refusal No Refusal Encountered
Standard Penetration Test (SPT) terminated per ASTM D 1586-99. Refer to typed boring log.
SCALE: AS SHOWN
EXCEPT AS NOTED
YARD
GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE - CENTERLINE
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TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING
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Knoxville, Tennessee
37871-4999
Tel. 865.922.9999
Fax 865.922.9100
www.stantec.com



SUMMARY OF OFFSET BORINGS			
BORING NO.	SURFACE ELEV. (FT.)	BORING TYPE	UNDISTURBED SAMPLE INTERVALS
STN-19A	788.9	CORE, PENETRATION TEST	NA

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NOT FOR CONSTRUCTION

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- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH	Weight of Hammer
WR	Weight of Rods
Δ	Standard Penetration Test Interval
■	Undisturbed Thin-Walled (Shelby) Tube Sample
Nss	Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
Nas	Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C.	Natural Moisture Content (%)
U.W.W.	Unit Weight Wet (lbs./cu.ft.)
U.W.D.	Unit Weight Dry (lbs./cu.ft.)
U.C./U.U.	Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 ▽	Water Level and Date Recorded
T.O.R.–	Top of Rock (Indicates the beginning of rock–like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
R.C.–	Begin Rock Core
R.Q.D.	Rock Quality Designation (%)
REC.	Recovery (%)
Refusal	Auger Refusal using a carbide–tipped tooth auger bit
No Refusal	No Refusal Encountered
*	Standard Penetration Test (SPT) terminated per ASTM D 1586–99. Refer to typed boring log.

SCALE: AS SHOWN
EXCEPT AS NOTED

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GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – CENTERLINE

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

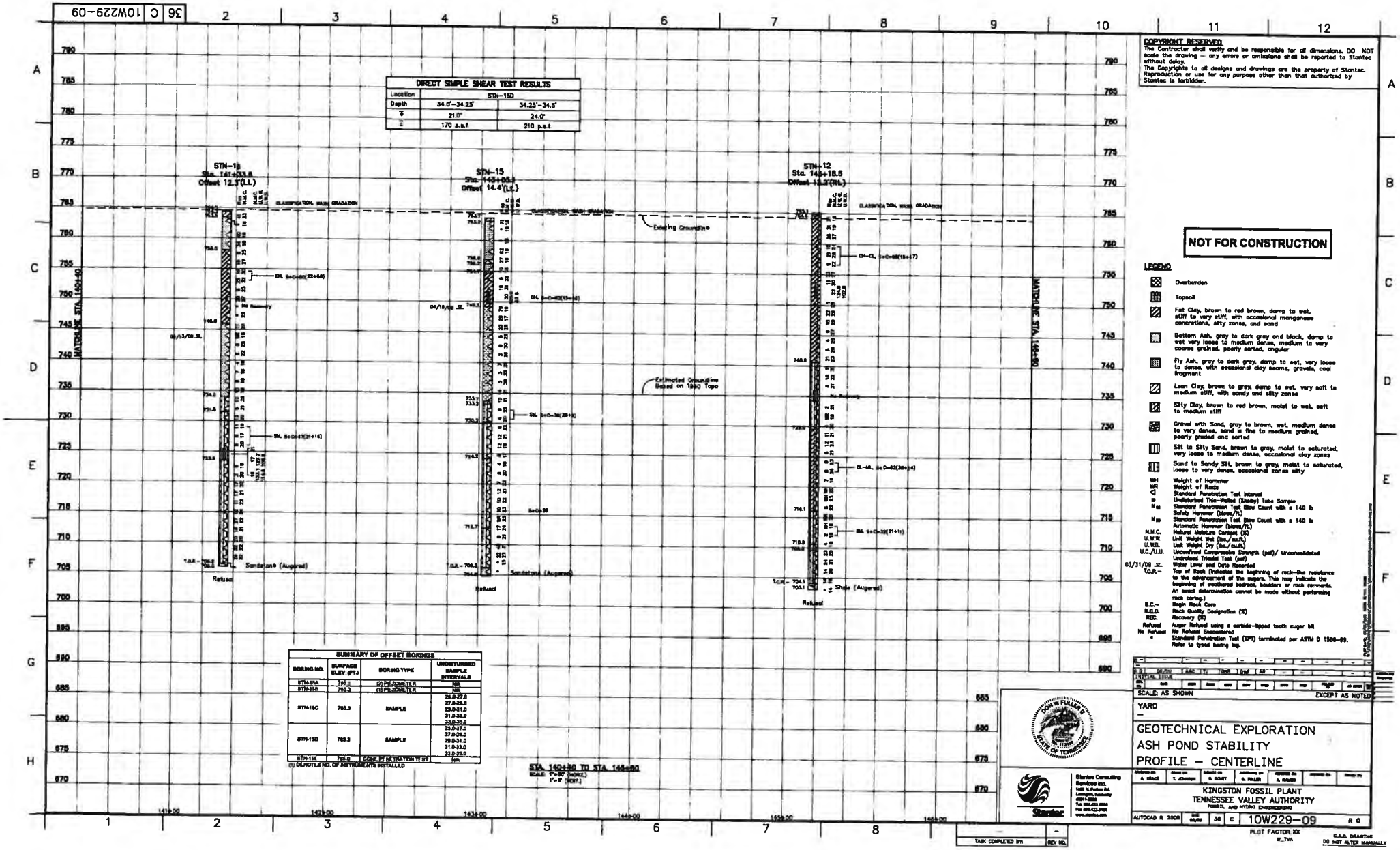
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TASK COMPLETED BY: – REV NO. –



DIRECT SIMPLE SHEAR TEST RESULTS		
Location	STN-15D	
Depth	34.0'-34.25'	34.25'-34.5'
ϕ	21.0°	24.0°
c	170 p.s.f.	210 p.s.f.

SUMMARY OF OFFSET BORINGS			
BORING NO.	SURFACE ELEV. (FT.)	BORING TYPE	UNDISTURBED SAMPLE INTERVALS
STN-15A	796.1	(2) PIEZOMETER	NA
STN-15B	760.3	(1) PIEZOMETER	NA
STN-15C	765.3	SAMPLE	25.0-27.0 27.0-29.0 29.0-31.0 31.0-33.0 33.0-35.0
STN-15D	769.3	SAMPLE	25.0-27.0 27.0-29.0 29.0-31.0 31.0-33.0 33.0-35.0
STN-15E	765.0	CONE PENETRATION TEST	NA

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NOT FOR CONSTRUCTION

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- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH – Weight of Hammer
WR – Weight of Rods
SPT – Standard Penetration Test Interval
ST – Undisturbed Thin-Walled (Shelby) Tube Sample
N_{60} – Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N_{140} – Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. – Natural Moisture Content (%)
U.W.W. – Unit Weight Wet (lbs./cu.ft.)
U.W.D. – Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. – Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 ⊻ – Water Level and Date Recorded
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REC. – Recovery (%)
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No Refusal – No Refusal Encountered
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SCALE: AS SHOWN
EXCEPT AS NOTED

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GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – CENTERLINE

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TENNESSEE VALLEY AUTHORITY
ROANE CO., AND HYDRO ENGINEERING

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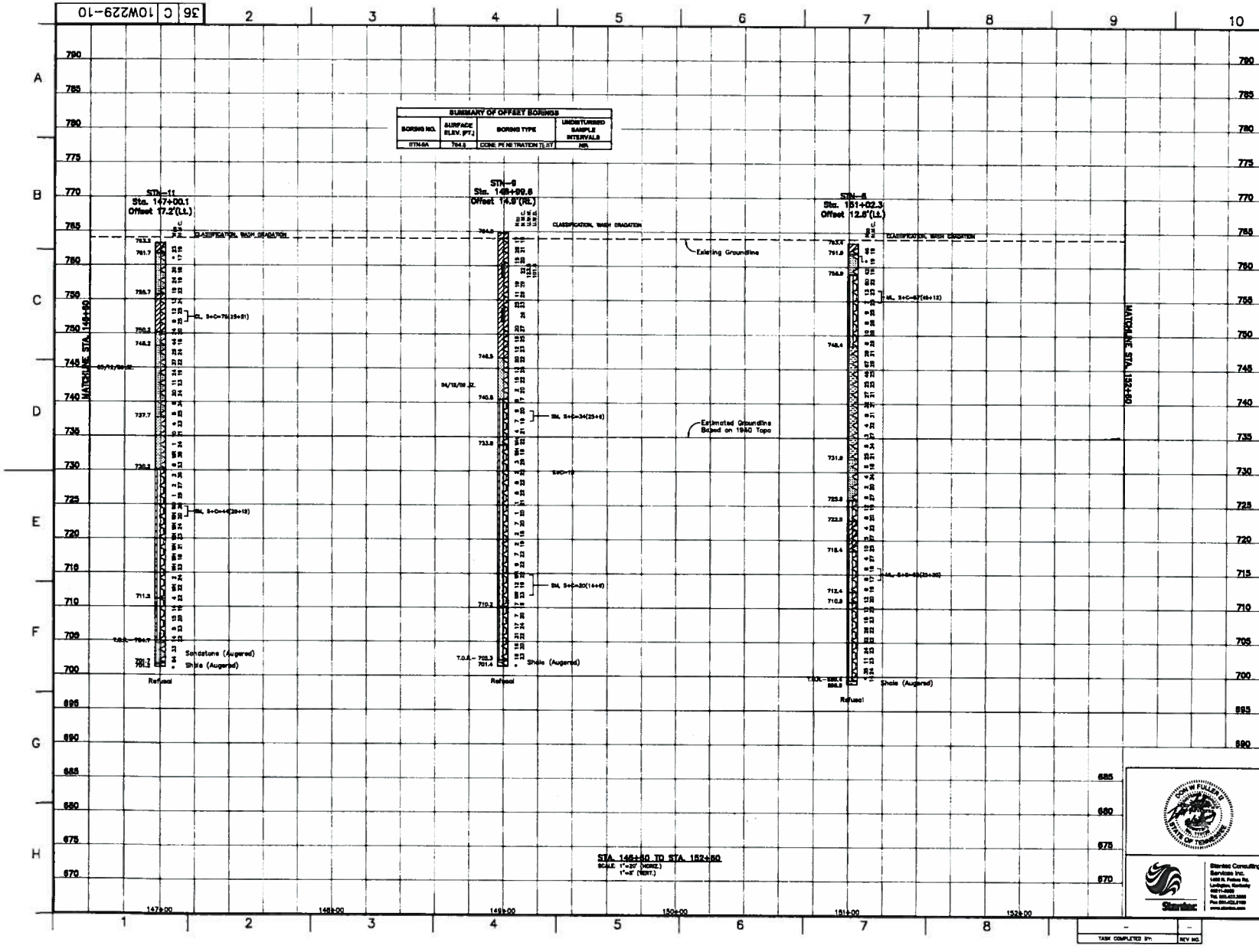
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STATE OF TENNESSEE

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- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH – Weight of Hammer
WR – Weight of Rods
S – Standard Penetration Test Interval
U – Undisturbed Thin-Walled (Shelby) Tube Sample
N60 – Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N60 – Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
M.C. – Natural Moisture Content (%)
U.W.W. – Unit Weight Wet (lbs./cu.ft.)
U.W.D. – Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. – Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/08 ▽ – Water Level and Date Recorded
T.O.R. – Top of Rock (indicates the beginning of rock-like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
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SCALE: AS SHOWN
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GEOTECHNICAL EXPLORATION
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PROFILE – CENTERLINE

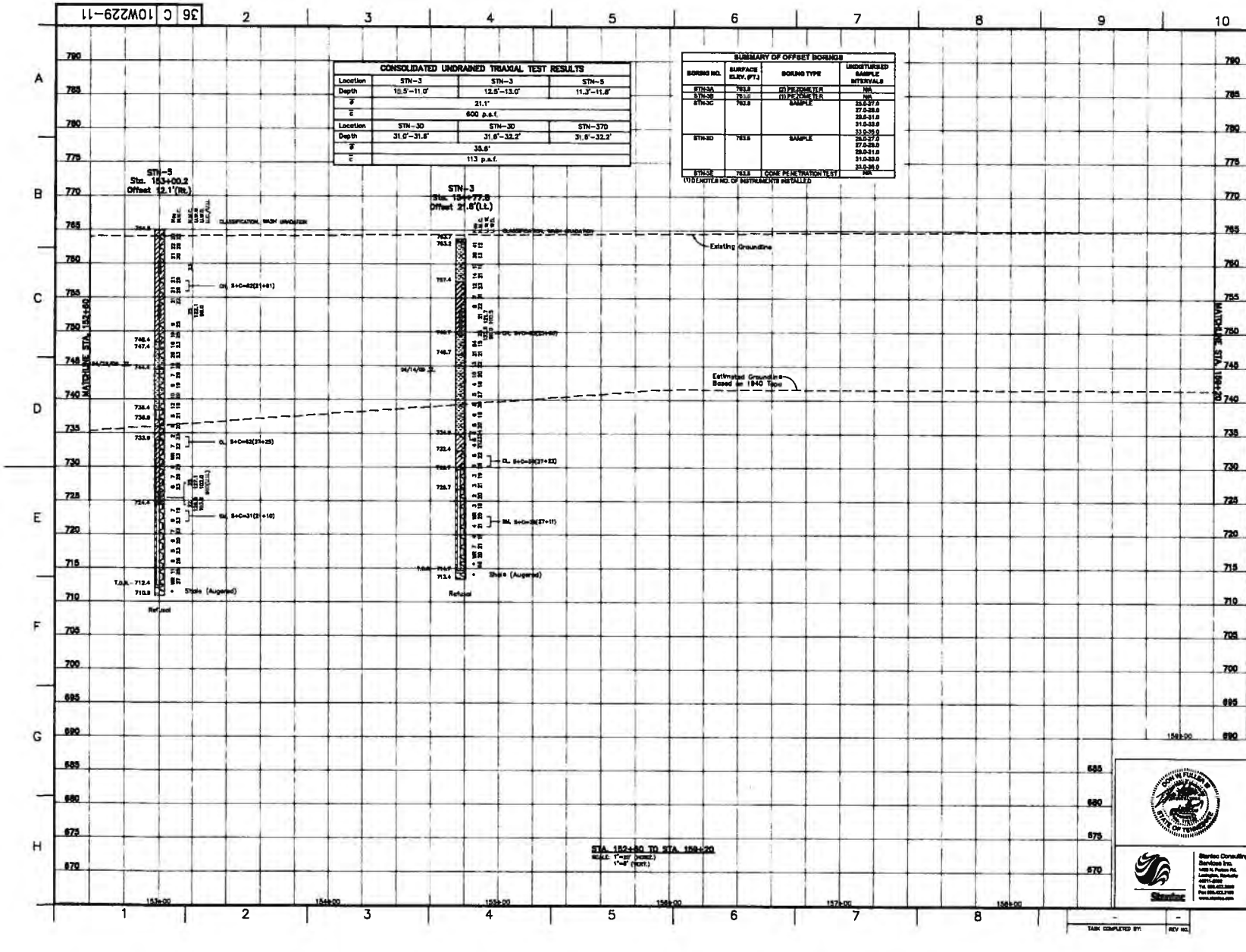
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CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS

Location	STN-3	STN-3	STN-5
Depth	10.5'-11.0'	12.5'-13.0'	11.3'-11.8'
ϕ	21.1°		
c	600 p.s.f.		
Location	STN-3D	STN-3D	STN-3TD
Depth	31.0'-31.8'	31.8'-32.2'	31.8'-32.2'
ϕ	35.8°		
c	113 p.s.f.		

SUMMARY OF OFFSET BORINGS

BORING NO.	SURFACE ELEV. (FT.)	BORING TYPE	UNDISTURBED SAMPLE INTERVALS
STN-3A	763.9	(2) PIEZOMETER	NA
STN-3B	763.6	(1) PIEZOMETER	NA
STN-3C	763.9	SAMPLE	23.0-27.0 27.0-29.0 29.0-31.0 31.0-33.0
STN-3D	763.6	SAMPLE	23.0-27.0 27.0-29.0 29.0-31.0 31.0-33.0
STN-3E	763.3	CONE PENETRATION TEST	23.0-36.0

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NOT FOR CONSTRUCTION

LEGEND

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WH Weight of Hammer
WR Weight of Rods
SS Standard Penetration Test Interval
U Undisturbed Thin-Walled (Shelby) Tube Sample
N60 Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N60 Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. Natural Moisture Content (%)
U.W.W. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/08 ▽ Water Level and Date Recorded
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R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal Auger Refusal using a carbide-tipped tooth auger bit
No Refusal No Refusal Encountered
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SCALE: AS SHOWN — EXCEPT AS NOTED

YARD
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GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – CENTERLINE

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

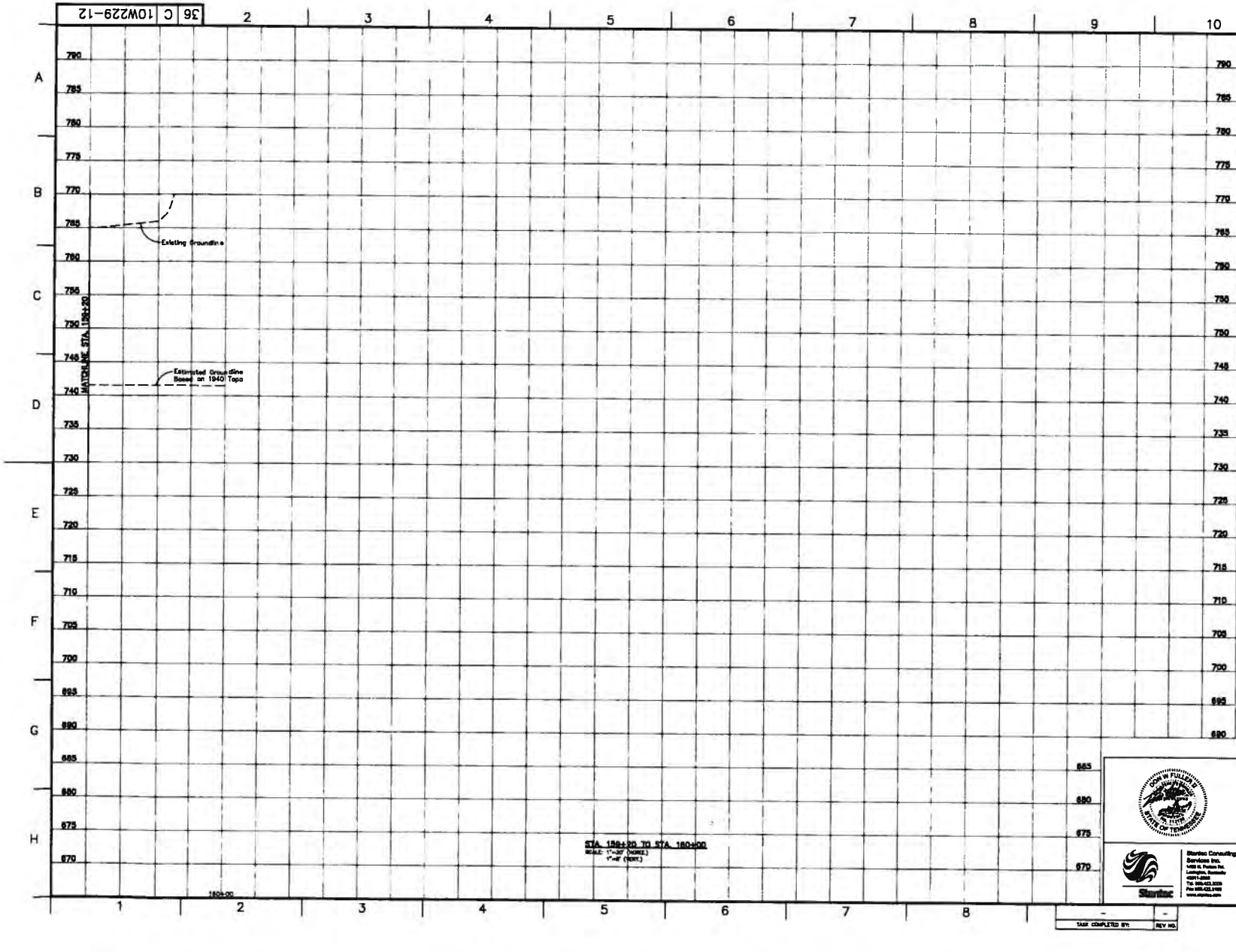
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C.A.D. DRAWING
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- Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
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- Lean Clay, brown to gray, damp to wet, very soft to medium stiff, with sandy and silty zones
- Silty Clay, brown to red brown, moist to wet, soft to medium stiff
- Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
- Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH – Weight of Hammer
WR – Weight of Rods
SPT – Standard Penetration Test Interval
ST – Undisturbed Thin-Walled (Shelby) Tube Sample
Nm – Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
Na – Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. – Natural Moisture Content (%)
U.W.W. – Unit Weight Wet (lbs./cu.ft.)
U.W.D. – Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. – Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 W.L. – Water Level and Date Recorded
T.O.R. – Top of Rock (Indicates the beginning of rock-like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C. – Begin Rock Core
R.Q.D. – Rock Quality Designation (%)
REC. – Recovery (%)
Refusal – Auger Refusal using a carbide-tipped tooth auger bit
No Refusal – No Refusal Encountered
* – Standard Penetration Test (SPT) terminated per ASTM D 1586-99. Refer to typed boring log.

SCALE: AS SHOWN
EXCEPT AS NOTED

YARD
–

GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – CENTERLINE

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2008 | 05/09 | 36 | C | 10W229-12 | R 0

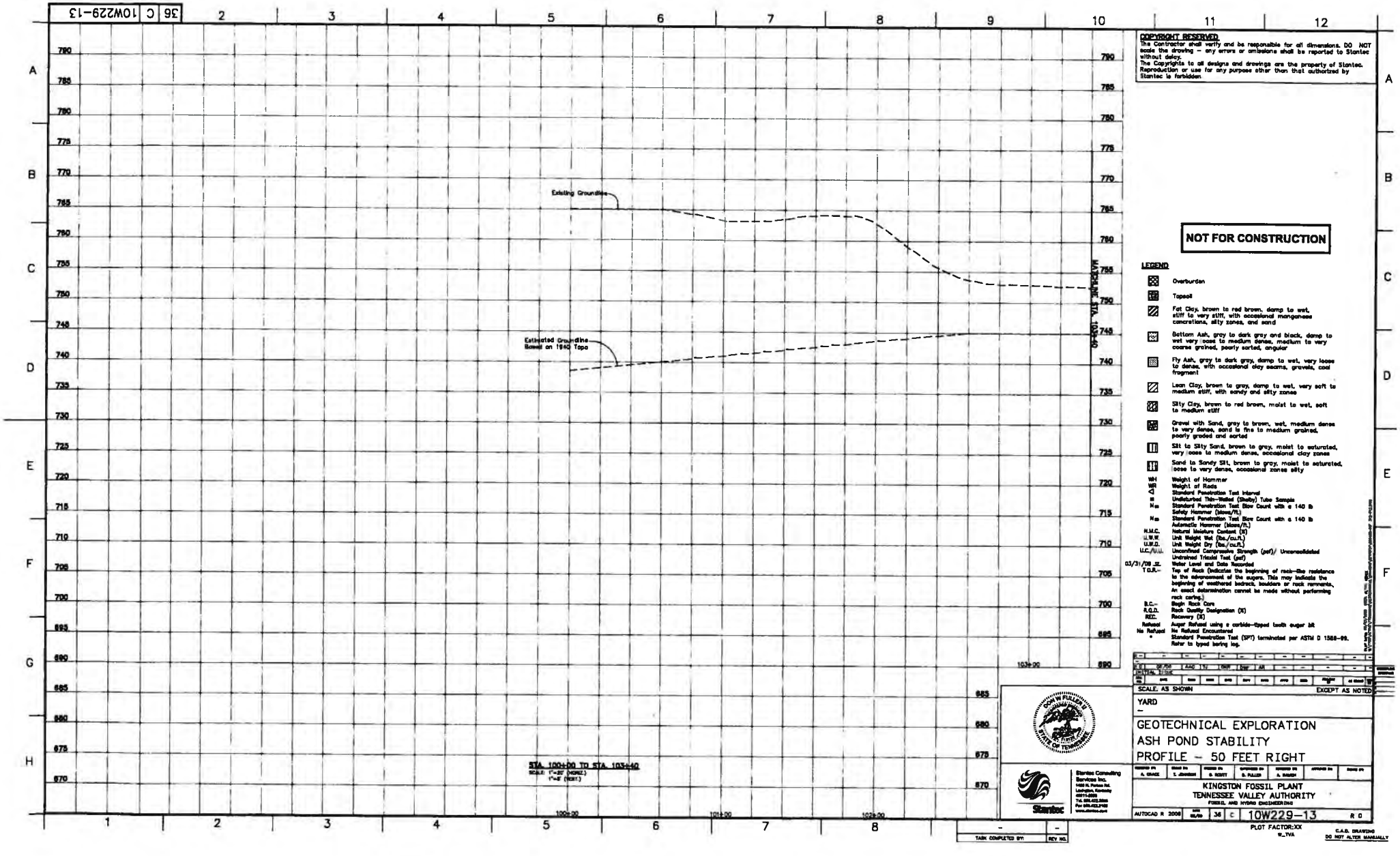
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NOT FOR CONSTRUCTION

LEGEND

- Overburden
- Topsoil
- Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
- Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
- Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
- Lean Clay, brown to gray, damp to wet, very soft to medium stiff, with sandy and silty zones
- Silty Clay, brown to red brown, moist to wet, soft to medium stiff
- Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
- Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH – Weight of Hammer
WR – Weight of Rods
SPT – Standard Penetration Test Interval
ST – Undisturbed Thin-Walled (Shelby) Tube Sample
N60 – Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N140 – Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
M.N.C. – Natural Moisture Content (%)
U.W.W. – Unit Weight Wet (lbs./cu.ft.)
U.W.D. – Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. – Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 W. – Water Level and Date Recorded
T.O.R. – Top of Rock (Indicates the beginning of resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
R.C. – Begin Rock Core
R.Q.D. – Rock Quality Designation (%)
REC. – Recovery (%)
Refusal – Auger Refusal using a carbide-tipped tooth auger bit
No Refusal – No Refusal Encountered
* – Standard Penetration Test (SPT) terminated per ASTM D 1586-99. Refer to typed boring log.

SCALE: AS SHOWN
EXCEPT AS NOTED

YARD
–

GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – 50 FEET RIGHT

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2008 | 36 | C | 10W229-13 | R 0

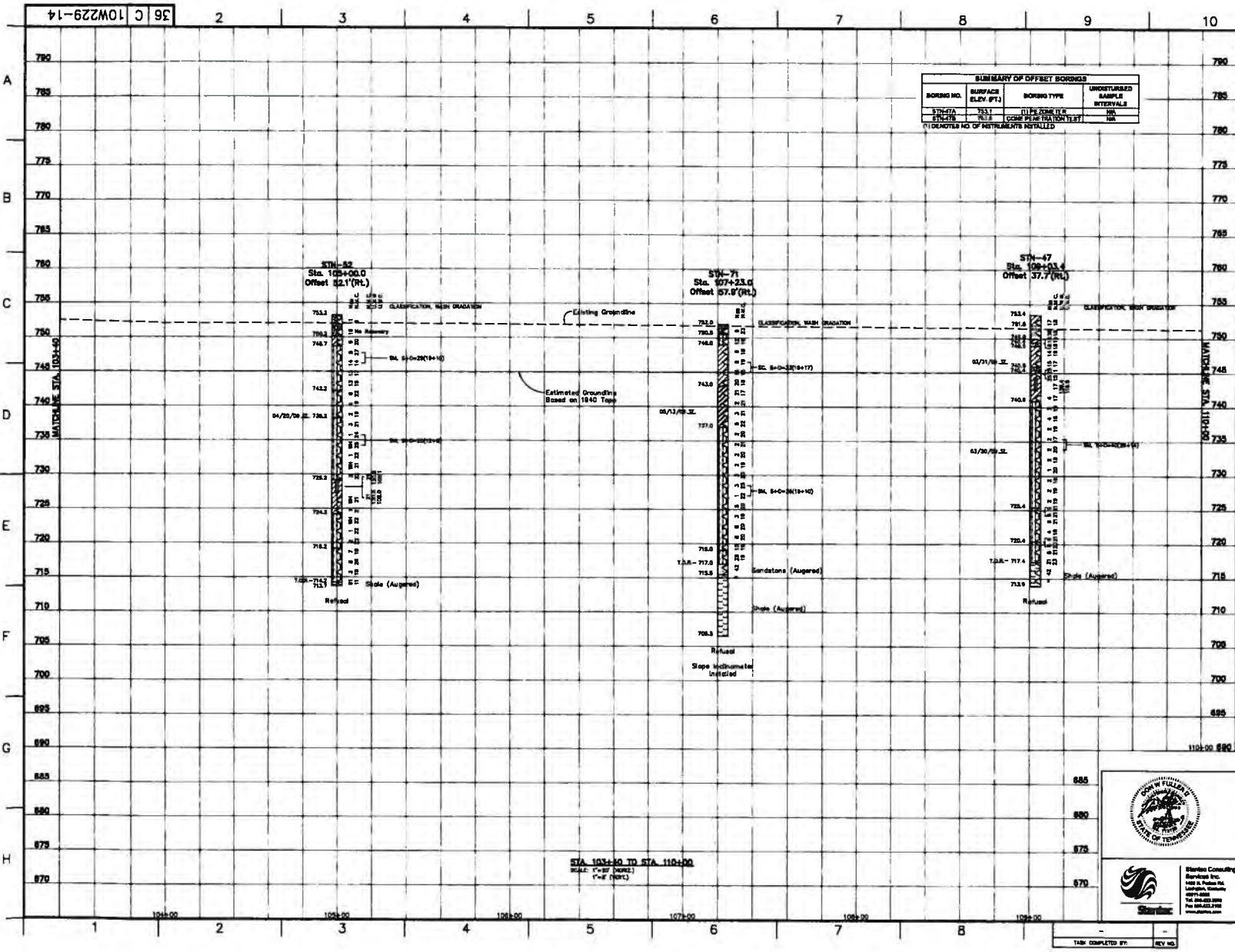
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TASK COMPLETED BY: – REV NO. –

10W229-13 | C | 36



SUMMARY OF OFFSET BORINGS

BORING NO.	SURFACE ELEV. (FT.)	BORING TYPE	UNDISTURBED SAMPLE INTERVALS
STN-47A	753.1	(1) PIEZOMETER	NA
STN-47B	753.1	CONE PENETRATION TEST	NA

(1) DENOTES NO. OF INSTRUMENTS INSTALLED

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NOT FOR CONSTRUCTION

LEGEND

- Overburden
- Topsoil
- Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
- Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
- Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
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- Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH	Weight of Hammer
WR	Weight of Rods
Δ	Standard Penetration Test Interval
≡	Undisturbed Thin-Walled (Shelby) Tube Sample
N=x	Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N=x	Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C.	Natural Moisture Content (%)
U.W.W.	Unit Weight Wet (lbs./cu.ft.)
U.W.D.	Unit Weight Dry (lbs./cu.ft.)
U.C./U.U.	Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
▽	Water Level and Date Recorded
03/31/09 ▽	
T.O.R.–	Top of Rock (Indicates the beginning of rock—the resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C.–	Begin Rock Core
R.Q.D.	Rock Quality Designation (%)
REC.	Recovery (%)
Refusal	Auger Refusal using a carbide-tipped tooth auger bit
No Refusal	No Refusal Encountered
*	Standard Penetration Test (SPT) terminated per ASTM D 1586-08. Refer to typed boring log.

SCALE: AS SHOWN EXCEPT AS NOTED

YARD
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GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – 50 FEET RIGHT

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

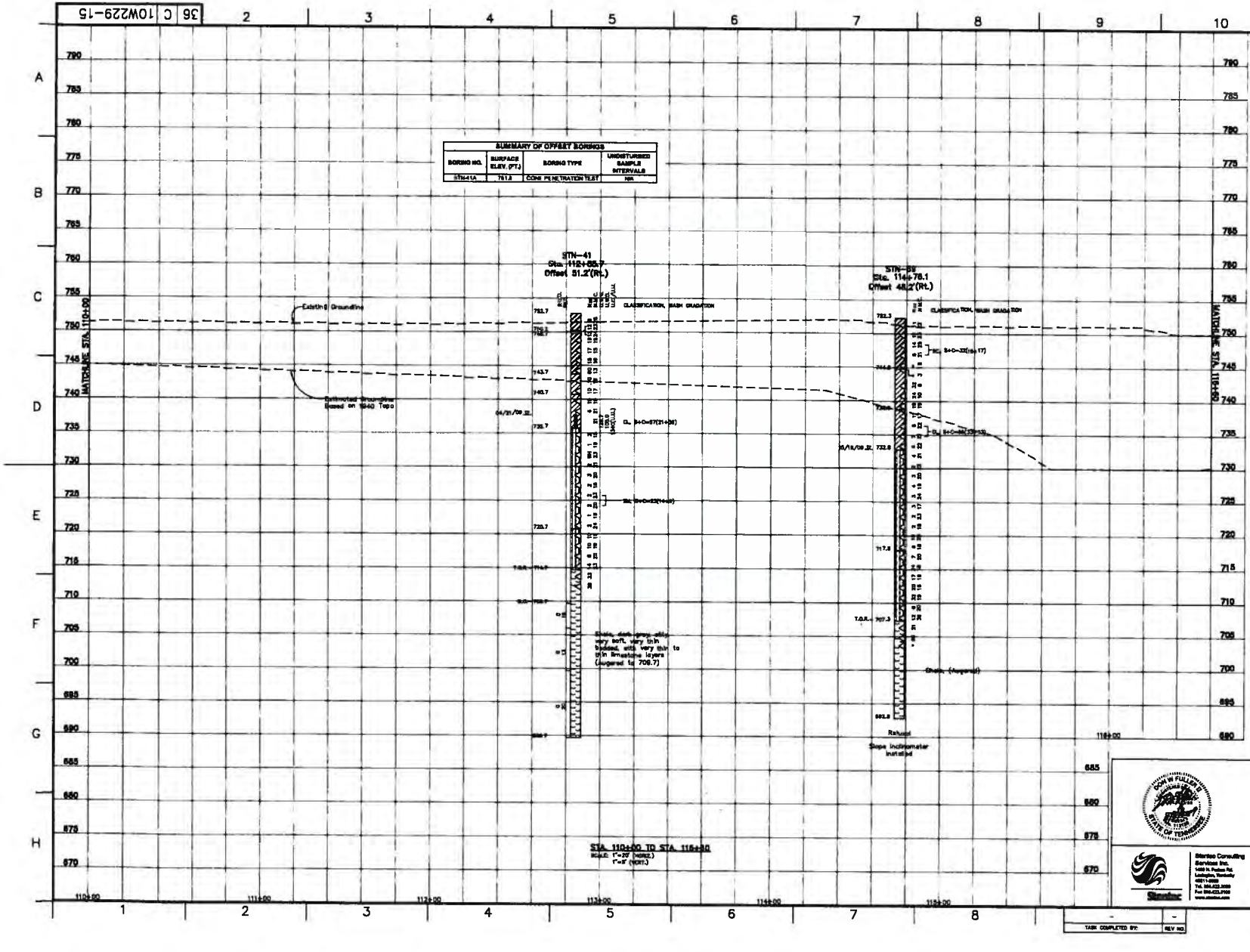
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NOT FOR CONSTRUCTION

LEGEND

- Overburden
- Topsoil
- Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
- Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
- Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
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- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH – Weight of Hammer
WR – Weight of Rods
SPT – Standard Penetration Test interval
U – Undisturbed Thin-Walled (Shelby) Tube Sample
Ncm – Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
Nam – Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. – Natural Moisture Content (%)
U.W.W. – Unit Weight Wet (lbs./cu.ft.)
U.W.D. – Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. – Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 ∇ – Water Level and Date Recorded
T.O.R. – Top of Rock (indicates the beginning of rock–like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C. – Begin Rock Core
R.Q.D. – Rock Quality Designation (%)
REC. – Recovery (%)
Refusal – Auger Refusal using a carbide–tipped tooth auger bit
No Refusal – No Refusal Encountered
* – Standard Penetration Test (SPT) terminated per ASTM D 1586–99. Refer to typed boring log.

SCALE: AS SHOWN
EXCEPT AS NOTED

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GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – 50 FEET RIGHT

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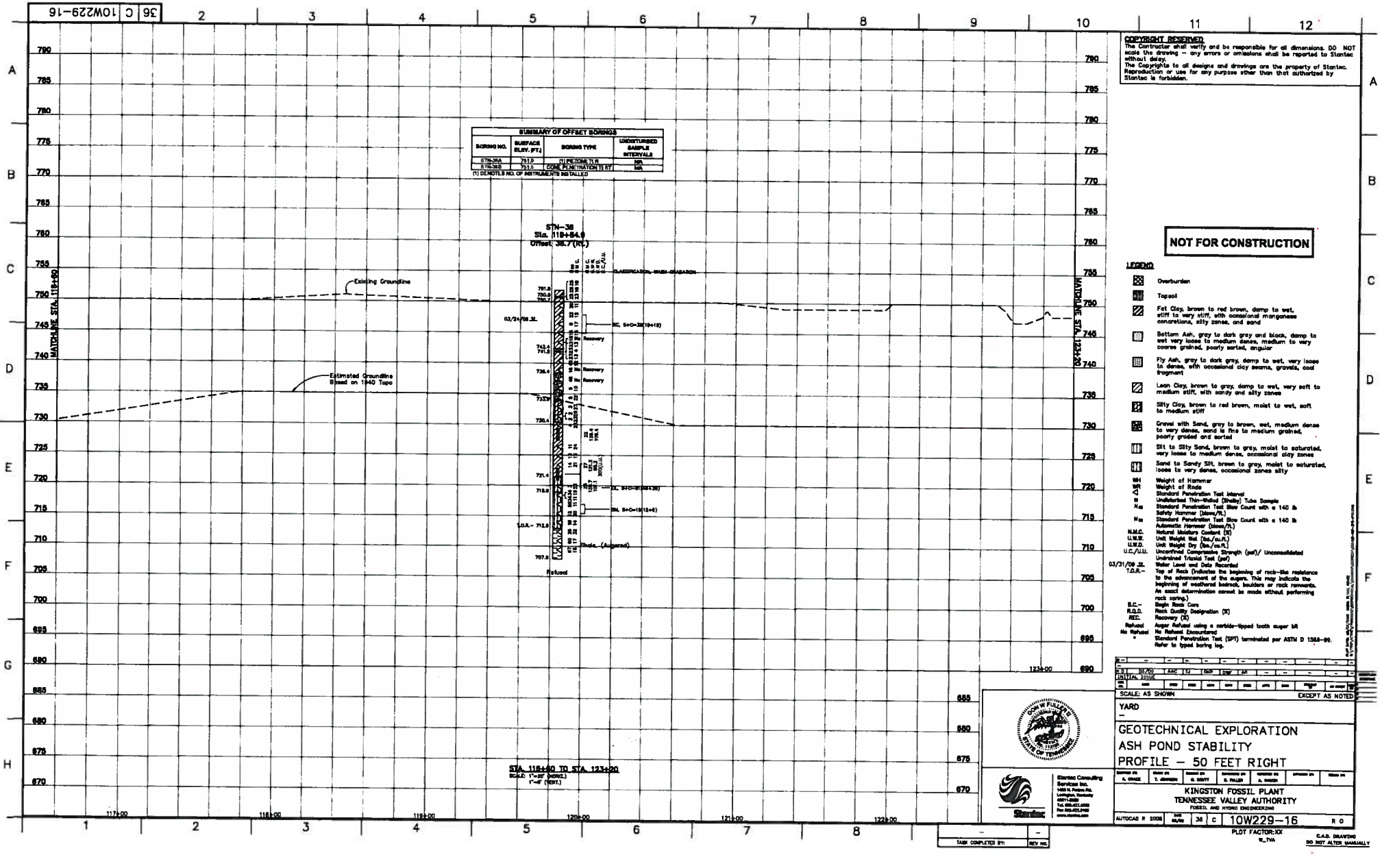
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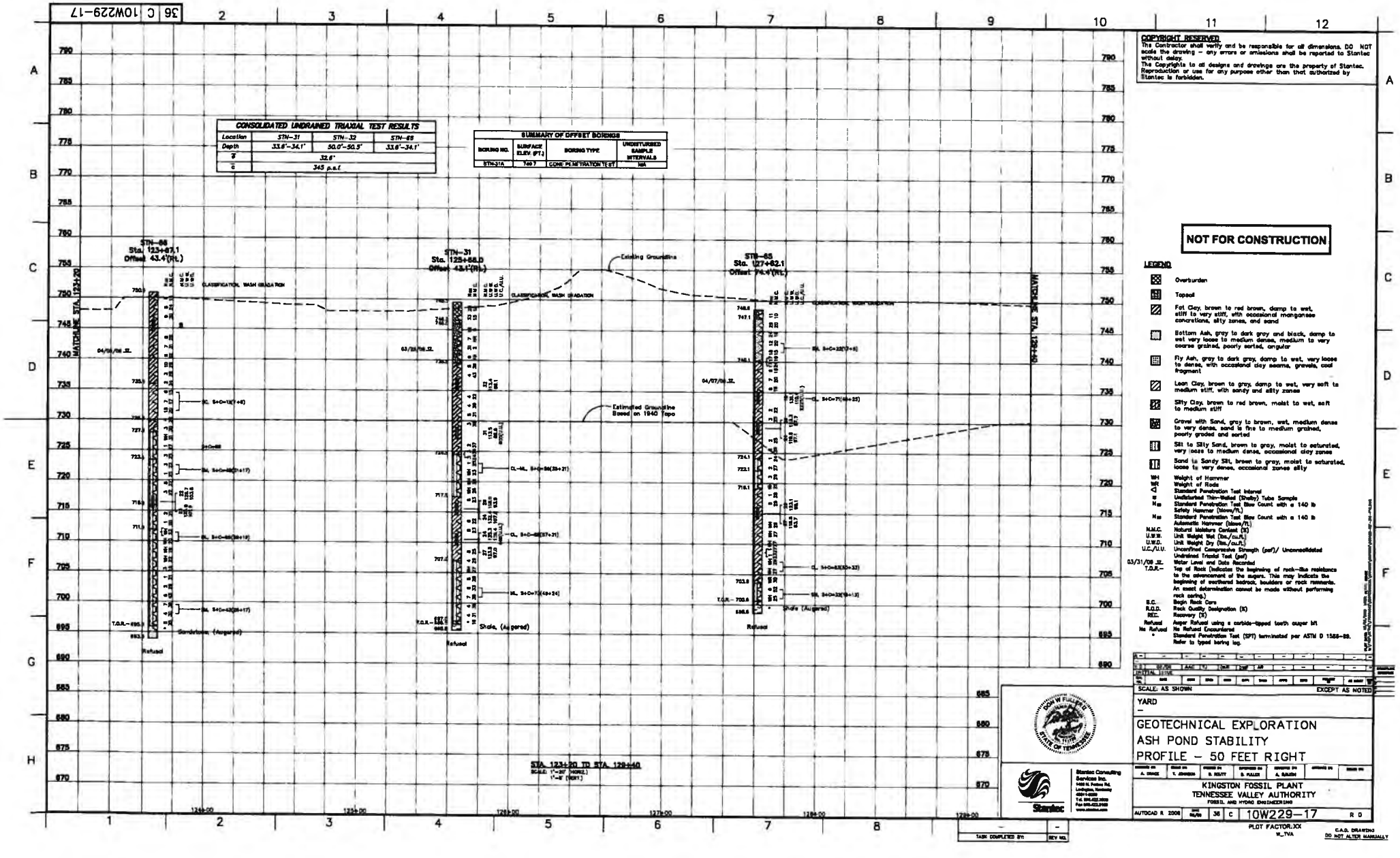
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DON W. FULLER
LICENSED PROFESSIONAL ENGINEER
STATE OF TENNESSEE

TASK COMPLETED BY: –
REV NO. –


36 C 10W229-16
SUMMARY OF OFFSET BORINGS
BORING NO. | SURFACE ELEV. (FT.) | BORING TYPE | UNDISTURBED SAMPLE INTERVALS
STN-36A | 731.9 | (1) PIEZOMETER | NA
STN-36B | 731.5 | CONE PENETRATION TEST | NA
(1) DENOTES NO. OF INSTRUMENTS INSTALLED
STN-38
Sta. 119+84.9
Offset: 36.7'(Rt.)
Existing Groundline
Estimated Groundline Based on 1940 Topo
MATCHLINE STA. 118+60
MATCHLINE STA. 123+20
Refusal
STA 118+60 TO STA 123+20
SCALE: 1"=20' (HORIZ.)
1"=2' (VERT.)
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NOT FOR CONSTRUCTION
LEGEND
Overburden
Topsoil
Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
Lean Clay, brown to gray, damp to wet, very soft to medium stiff, with sandy and silty zones
Silty Clay, brown to red brown, moist to wet, soft to medium stiff
Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty
WH Weight of Hammer
WR Weight of Rods
SPT Standard Penetration Test Interval
U Undisturbed Thin-Walled (Shelby) Tube Sample
N60 Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. Natural Moisture Content (%)
U.W.W. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/08 ▽ Water Level and Date Recorded
T.O.R.– Top of Rock (Indicates the beginning of rock–like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C.– Begin Rock Core
R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal Auger Refusal using a carbide–tipped tooth auger bit
No Refusal No Refusal Encountered
* Standard Penetration Test (SPT) terminated per ASTM D 1586–99. Refer to typed boring log.
SCALE: AS SHOWN
EXCEPT AS NOTED
YARD
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GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – 50 FEET RIGHT
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TENNESSEE VALLEY AUTHORITY
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36 C 10W229-16 R 0
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CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS

Location	STN-31	STN-32	STN-68
Depth	33.6'-34.1'	50.0'-50.5'	33.6'-34.1'
$\bar{\sigma}$	32.6°		
c	345 p.s.f.		

SUMMARY OF OFFSET BORINGS

BORING NO.	SURFACE ELEV. (FT.)	BORING TYPE	UNDISTURBED SAMPLE INTERVALS
STN-31A	749 ?	CONE PENETRATION TEST	NA

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NOT FOR CONSTRUCTION

LEGEND

- Overburden
- Topsoil
- Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
- Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
- Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
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- Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
- Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH	Weight of Hammer
WR	Weight of Rods
Δ	Standard Penetration Test Interval
▯	Undisturbed Thin-Walled (Shelby) Tube Sample
N_{60}	Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N_{AR}	Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C.	Natural Moisture Content (%)
U.W.W.	Unit Weight Wet (lbs./cu.ft.)
U.W.D.	Unit Weight Dry (lbs./cu.ft.)
U.C./U.U.	Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 ∑	Water Level and Date Recorded
T.O.R.–	Top of Rock (indicates the beginning of rock-like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
R.C.	Begin Rock Core
R.Q.D.	Rock Quality Designation (%)
REC.	Recovery (%)
Refusal	Auger Refusal using a carbide-tipped tooth auger bit
No Refusal	No Refusal Encountered
*	Standard Penetration Test (SPT) terminated per ASTM D 1586-99. Refer to typed boring log.

SCALE: AS SHOWN
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GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – 50 FEET RIGHT

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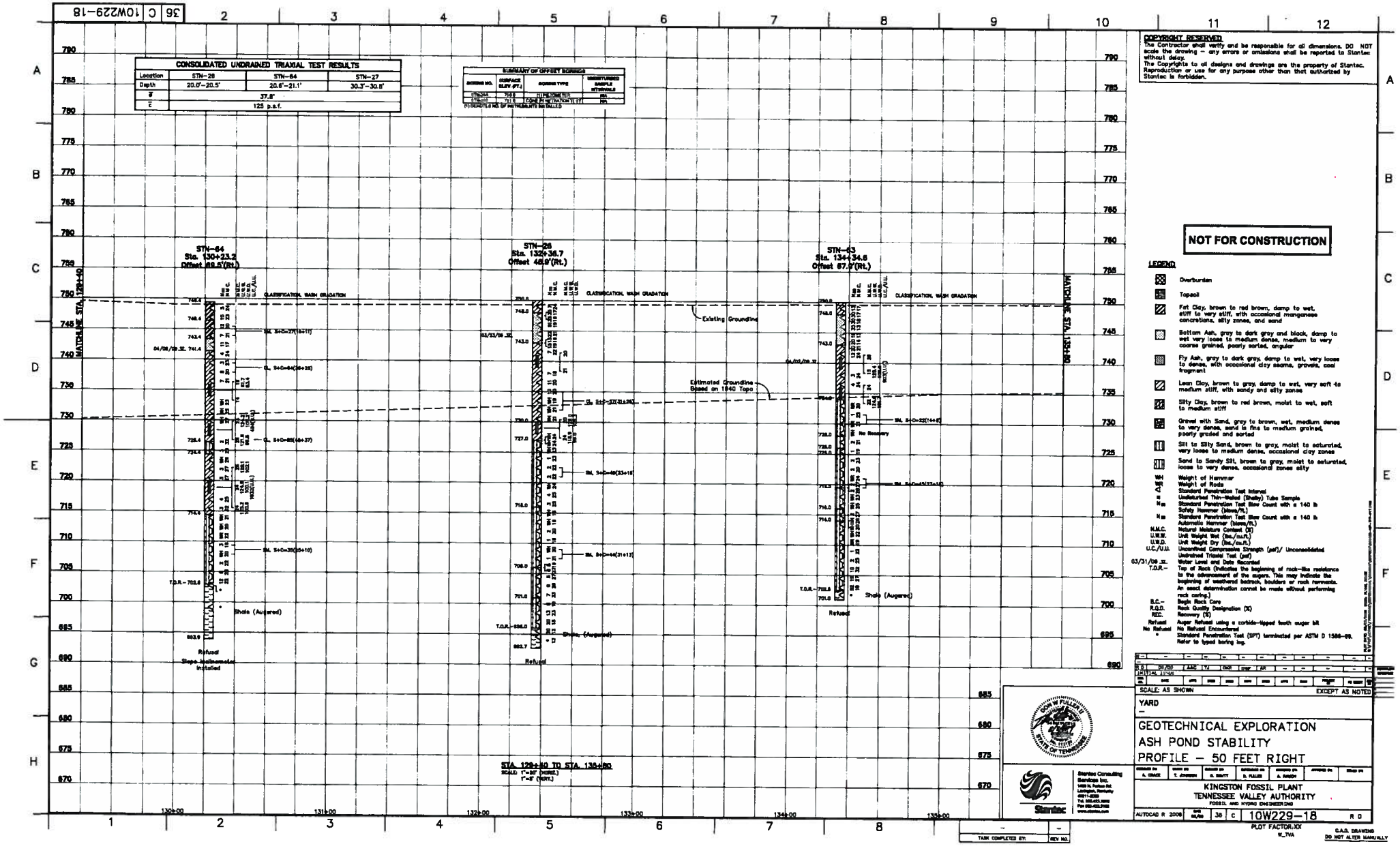
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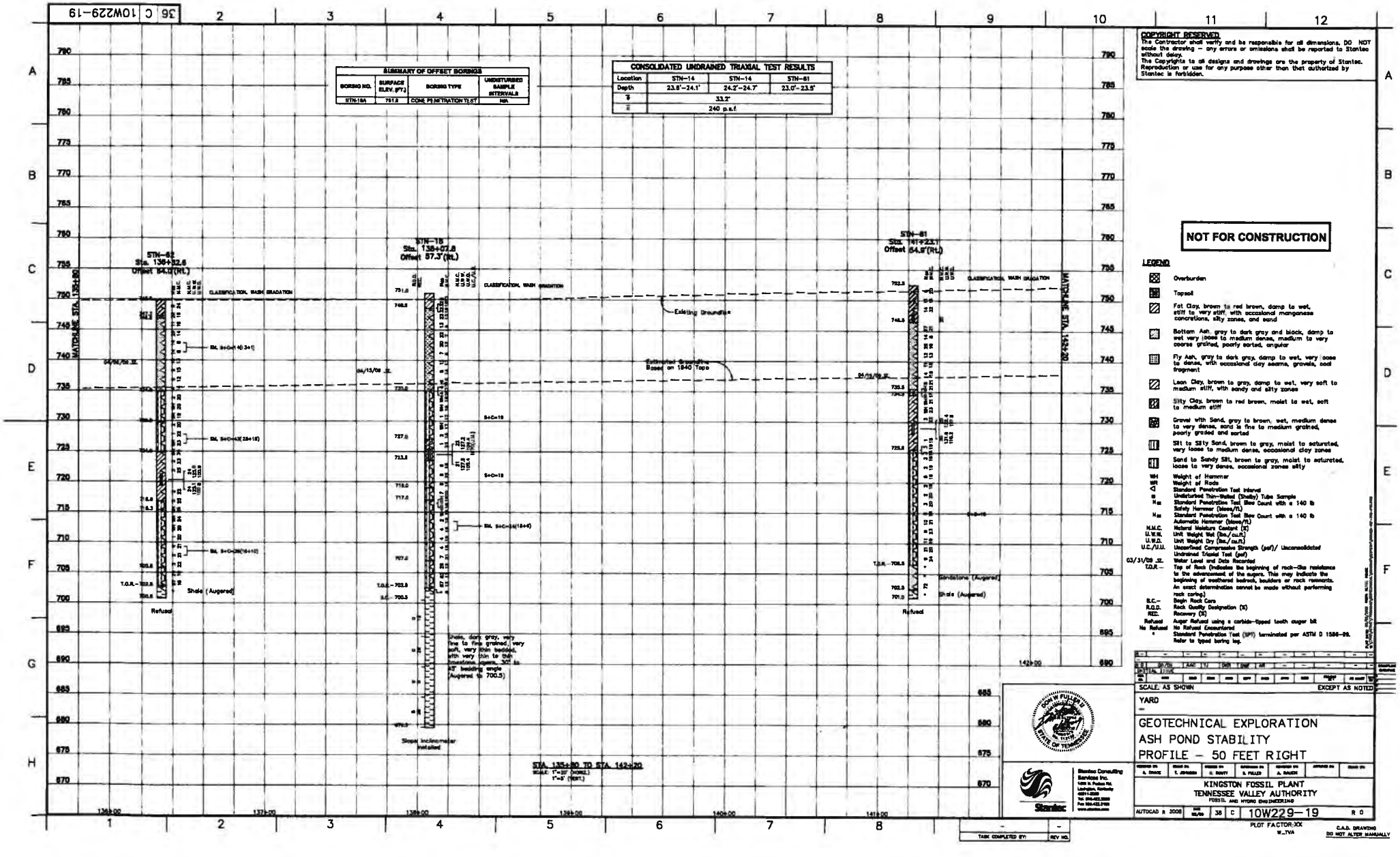
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TASK COMPLETED BY: – REV NO. –

36 C 10W229-17


36 C 10W229-18
CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS
Location | STN-28 | STN-64 | STN-27
Depth | 20.0'–20.5' | 20.6'–21.1' | 30.3'–30.8'
ϕ | 37.8°
c | 125 p.s.f.
SUMMARY OF OFFSET BORINGS
BORING NO. | SURFACE ELEV. (FT.) | BORING TYPE | UNDISTURBED SAMPLE INTERVALS
STN-28A | 748.0 | (1) PIEZOMETER | 20ft
STN-28B | 731.0 | CORE BORING (1) | 20ft
(1) DENOTES NO. OF INSTRUMENTS INSTALLED
STN-64
Sta. 130+23.2
Offset 69.5'(Rt.)
STN-28
Sta. 132+36.7
Offset 48.9'(Rt.)
STN-43
Sta. 134+34.6
Offset 67.9'(Rt.)
MATCH LINE STA. 129+50
MATCH LINE STA. 135+50
CLASSIFICATION, WASH GRADATION
Existing Groundline
Estimated Groundline Based on 1940 Topo
Shale (Augered)
Refused
Slope Inclinometer Installed
No Recovery
STA. 129+50 TO STA. 135+50
SCALE: 1"=50' (HORIZ.)
1"=5' (VERT.)
NOT FOR CONSTRUCTION
LEGEND
Overburden
Topsoil
Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
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Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
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WH Weight of Hammer
WR Weight of Rods
Δ Standard Penetration Test Interval
Undisturbed Thin-Walled (Shelby) Tube Sample
Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. Natural Moisture Content (%)
U.W.W. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
Water Level and Date Recorded
03/31/09 ∇
T.O.R.– Top of Rock (Indicates the beginning of rock–like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock formants. An exact determination cannot be made without performing rock coring.)
B.C.– Begin Rock Core
R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal Auger Refusal using a carbide–tipped tooth auger bit
No Refusal Encountered
* Standard Penetration Test (SPT) terminated per ASTM D 1586–99. Refer to typed boring log.
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ASH POND STABILITY
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Stantec
AUTOCAD R 2008
36 C 10W229-18 R 0
PLOT FACTOR:XX
W_TVA
C.A.D. DRAWING
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TASK COMPLETED BY:
REV NO.


SUMMARY OF OFFSET BORINGS
BORING NO. | SURFACE ELEV. (FT.) | BORING TYPE | UNDISTURBED SAMPLE INTERVALS
STN-16A | 751.8 | CORE PENETRATION 71.5' | NA
CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS
Location | STN-14 | STN-14 | STN-81
Depth | 23.6'-24.1' | 24.2'-24.7' | 23.0'-23.5'
ϕ | 33.2°
c | 240 p.s.f.
STN-82
Sta. 136+32.6
Offset 84.0'(Rt.)
STN-16
Sta. 138+07.8
Offset 87.3'(Rt.)
STN-81
Sta. 141+23.1
Offset 84.8'(Rt.)
CLASSIFICATION, MAIN EMBANKMENT
Existing Groundline
Estimated Groundline Based on 1940 Topo
Shale (Augered)
Refusal
Sandstone (Augered)
Shale, dark gray, very fine to fine grained, very soft, very thin bedded, with very thin to thin limestone seams, 30° to 40° bedding angle (Augered to 700.5)
Slope Inclinometer Installed
STA. 135+80 TO STA. 142+20
SCALE: 1"=50' (HORIZ.)
1"=5' (VERT.)
MATCHLINE STA. 135+80
MATCHLINE STA. 142+20
NOT FOR CONSTRUCTION
LEGEND
Overburden
Topsoil
Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
Lean Clay, brown to gray, damp to wet, very soft to medium stiff, with sandy and silty zones
Silty Clay, brown to red brown, moist to wet, soft to medium stiff
Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty
WH Weight of Hammer
WR Weight of Rods
Standard Penetration Test Interval
Undisturbed Thin-Walled (Shelby) Tube Sample
Standard Penetration Test, Blow Count with a 140 lb Safety Hammer (blows/ft)
Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft)
N.M.C. Natural Moisture Content (%)
U.W.M. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 ⊻ Water Level and Date Recorded
T.O.R. Top of Rock (Indicates the beginning of rock--Has resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring)
R.C. Begin Rock Core
R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal No Refusal Auger Refusal using a carbide-tipped tooth auger bit
Standard Penetration Test (SPT) terminated per ASTM D 1586-99. Refer to typed boring log.
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GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – 50 FEET RIGHT
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
AUTOCAD R 2008
36 C 10W229-19 R 0
PLOT FACTOR:XX
W_TVA
C.A.D. DRAWING
DO NOT ALTER MANUALLY
TASK COMPLETED BY:
REV NO.

SUMMARY OF OFFSET BORINGS			
BORING NO.	SURFACE ELEV. (FT.)	BORING TYPE	UNDISTURBED SAMPLE INTERVALS
STN-14A	731.1	CORE DRILLING THRU 15.0	5ft
STN-14B	731.1	CORE DRILLING THRU 15.0	NA

(1) DEPTH/NO. OF UNDISTURBED SAMPLES

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS			
Location	STN-14	STN-14	STN-81
Depth	23.6'-24.1'	24.2'-24.7'	23.0'-23.5'
ϕ	33.2°		
c	240 p.s.f.		

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NOT FOR CONSTRUCTION

LEGEND

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- Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
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- Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH Weight of Hammer
WR Weight of Rods
Δ Standard Penetration Test Interval
ø Undisturbed Thin-Walled (Shelby) Tube Sample
N60 Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. Natural Moisture Content (%)
U.W.W. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 ∑ Water Level and Date Recorded
T.O.R.– Top of Rock (Indicates the beginning of rock–like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulder or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C.– Begin Rock Core
R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal Auger Refusal using a carbide–tipped tooth auger bit
No Refusal No Refusal Encountered
* Standard Penetration Test (SPT) terminated per ASTM D 1586–99. Refer to typed boring log.

STN-14
Sta. 143+05.
Offset 55.1'(Rt.)

STN-80
Sta. 145+18.2
Offset 65.3'(Rt.)

STN-81
Sta. 147+18.2
Offset 80.5'(Rt.)

MATCHLINE STA. 142+20

MATCHLINE STA. 148+40

CLASSIFICATION, WASH GRADATION

Existing Groundline

Estimated Groundline Based on 1940 Topo

CL, S+C=83(39+44)

CL-ML, S+C=86(43+43)

ML, S+C=82(39+43)

ML, S+C=81(33+48)

S+C=83

S+C=44

S+C=86(30+56)

S+C=81(33+48)

S+C=82(39+43)

03/26/09 ∑

04/08/09 ∑

03/27/09 ∑

No Recovery

T.O.R.– 706.9

T.O.R.– 701.0

T.O.R.– 701.9

Bottom Ash (Augered)

Shale (Augered)

Refusal

STA. 142+20 TO STA. 148+40
SCALE: 1"=20' (HORIZ.)
1"=5' (VERT.)

SCALE: AS SHOWN
EXCEPT AS NOTED

YARD
–

GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – 50 FEET RIGHT

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

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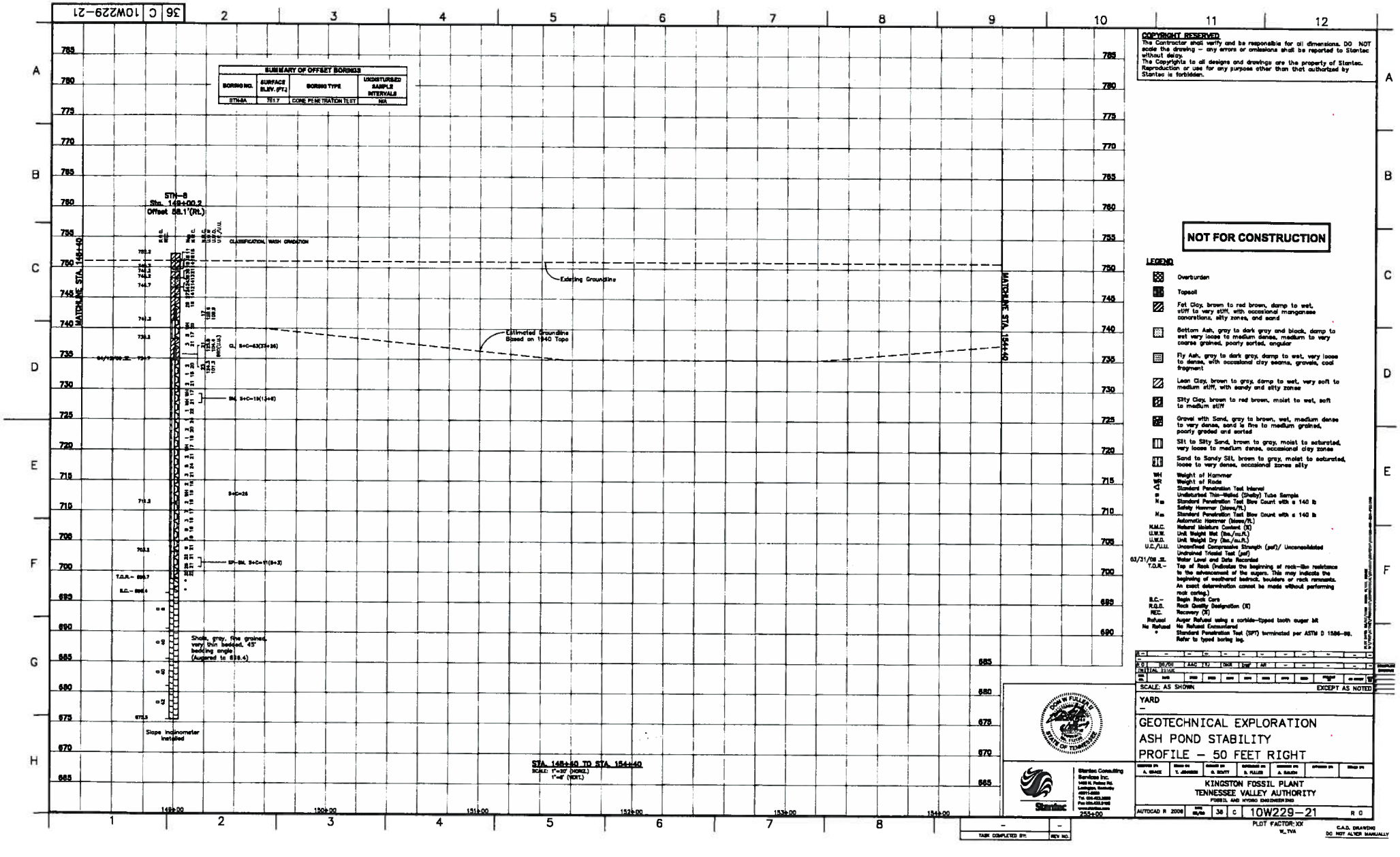
PLOT FACTOR:XX
W_TVA

C.A.D. DRAWING
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JOHN W. FULLER
LICENSED PROFESSIONAL ENGINEER
STATE OF TENNESSEE

Stantec Consulting Services Inc.
1409 N. Peters Rd.
Knoxville, Tennessee
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Tel. 865-691-2500
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NOT FOR CONSTRUCTION

LEGEND

- Overburden
- Topsoil
- Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
- Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
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- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH – Weight of Hammer
WR – Weight of Rods
Δ – Standard Penetration Test Interval
ST – Undisturbed Thin-Walled (Shelby) Tube Sample
N60 – Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N60 – Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
M.N.C. – Natural Moisture Content (%)
U.W.W. – Unit Weight Wet (lbs./cu.ft.)
U.W.D. – Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. – Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/08 ▽ – Water Level and Date Recorded
T.O.R.– Top of Rock (indicates the beginning of rock–like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C.– Begin Rock Core
R.Q.D. – Rock Quality Designation (%)
REC. – Recovery (%)
Refusal – Auger Refusal using a carbide–tipped tooth auger bit
No Refusal – No Refusal Encountered
* – Standard Penetration Test (SPT) terminated per ASTM D 1586–99. Refer to typed boring log.

R	DESCRIPTION	BY	CHK	YY-MM-DD
0	INITIAL ISSUE	AAC	TJ	08/05

SCALE: AS SHOWN EXCEPT AS NOTED

YARD

GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE – 50 FEET RIGHT

DESIGNED BY	DRAWN BY	CHECKED BY	QUALITY CHECKED BY	APPROVED BY	SIGNED BY
A. ISAACS	T. JOHNSON	A. SCOTT	A. FULLER	A. BAUER	

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

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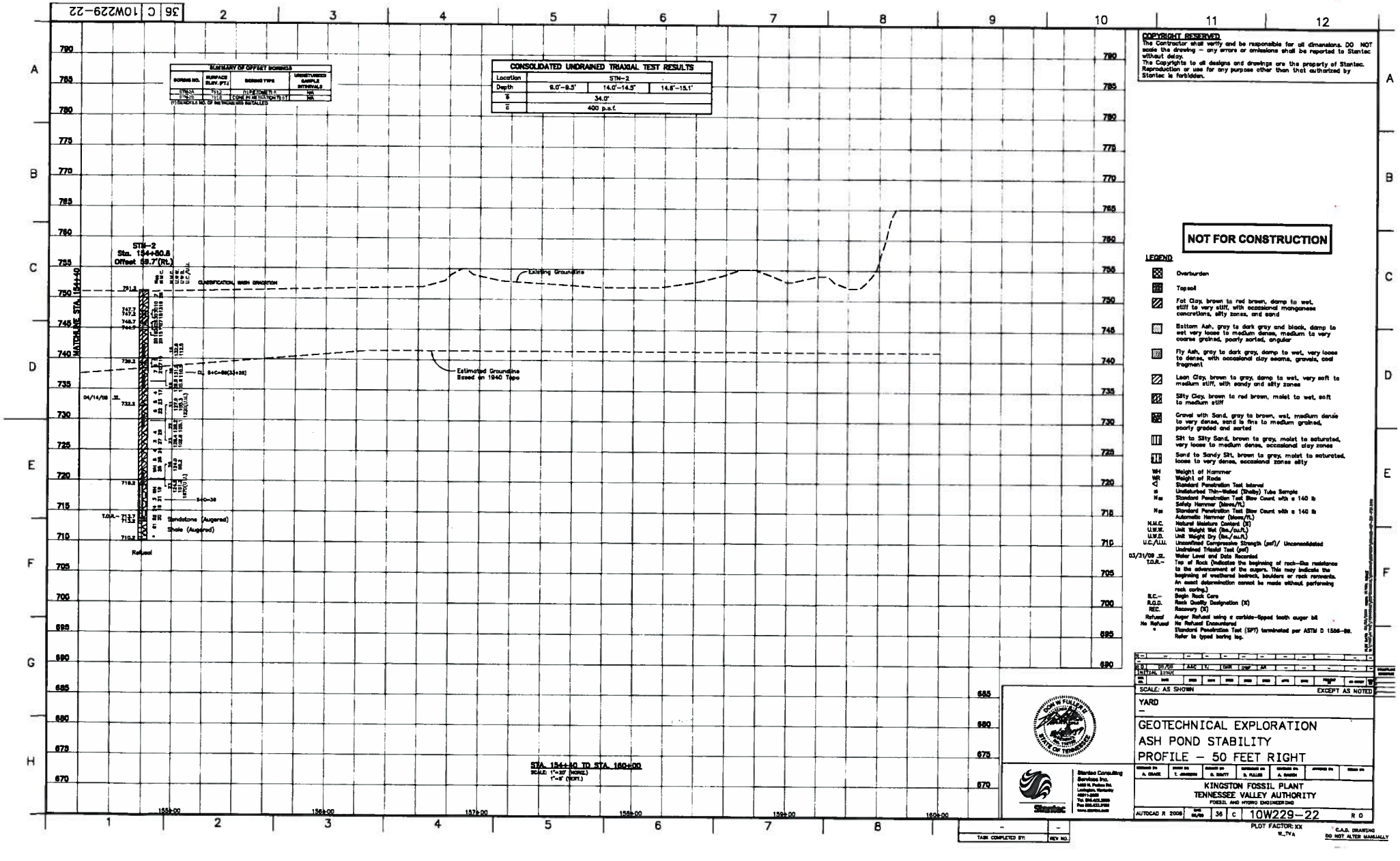
PLOT FACTOR: XX
W_TVA

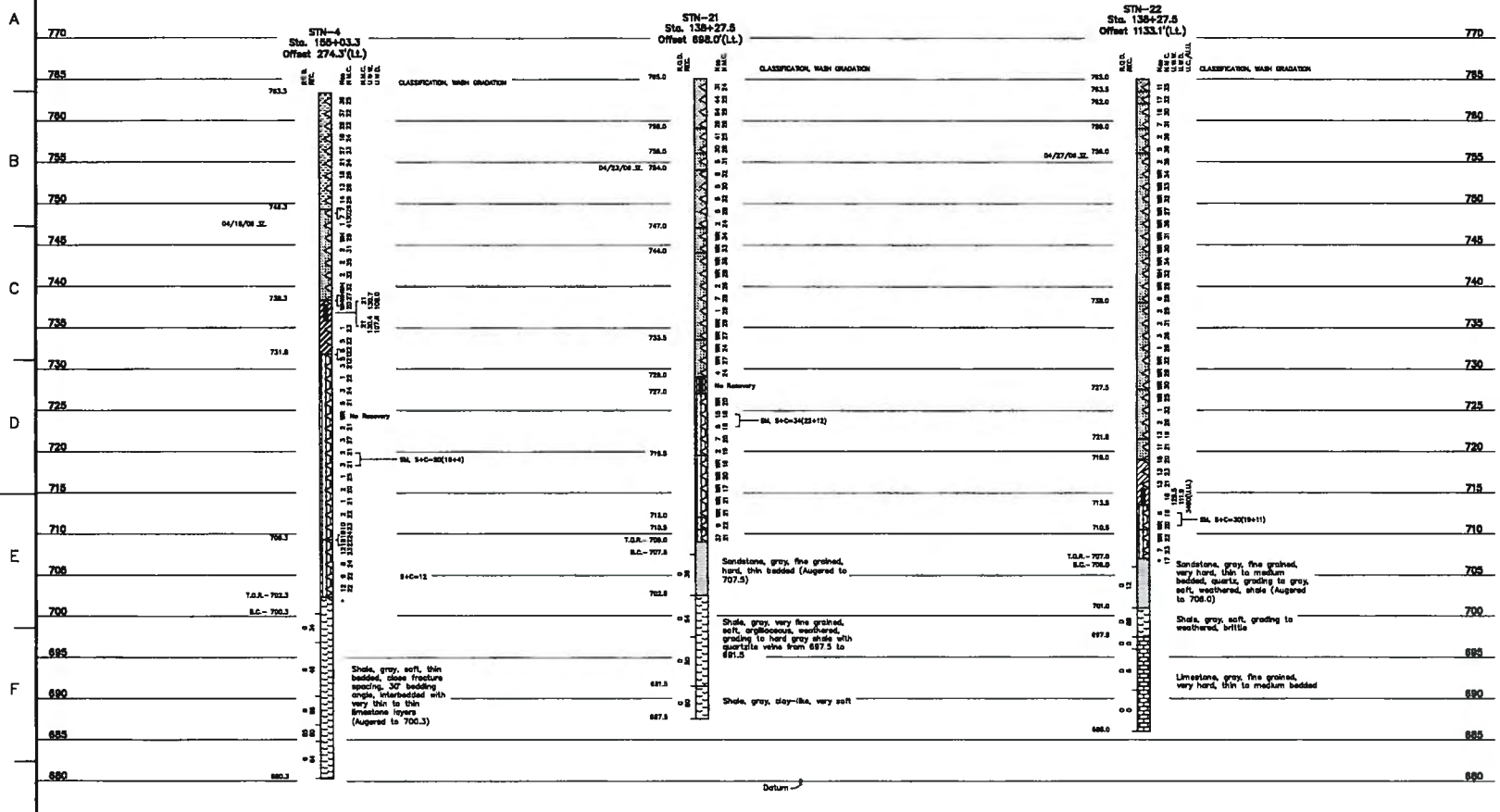
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Knoxville, Tennessee
37923-4819
Tel. 865-470-5555
Fax 865-470-5973
www.stantec.com

255+00


SUMMARY OF OFFSET BORINGS
CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS
STN-2
Sta. 154+80.8
Offset 69.7'(Rt.)
Existing Groundline
Estimated Groundline
Based on 1940 Tops
CLASSIFICATION, MAIN EXCAVATION
MAINLINE STA. 154+40
Standstone (Augered)
Shale (Augered)
Refusal
STA. 154+40 TO STA. 160+00
SCALE: 1"=20' (HORIZ.)
1"=5' (VERT.)
NOT FOR CONSTRUCTION
LEGEND
Overburden
Topsoil
Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
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Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty
WH Weight of Hammer
WR Weight of Rods
Standard Penetration Test Interval
Undisturbed Thin-Walled (Shelby) Tube Sample
Standard Penetration Test, Blow Count with a 140 lb Safety Hammer (blows/ft.)
Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
M.N.C. Natural Moisture Content (%)
U.W.W. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
Water Level and Date Recorded
Top of Rock (Indicates the beginning of rock-like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C.- Begin Rock Core
R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal No Refusal
Auger Refusal using a carbide-tipped tooth auger bit
No Refusal Encountered
Standard Penetration Test (SPT) terminated per ASTM D 1586-99. Refer to typed boring log.
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SCALE: AS SHOWN
EXCEPT AS NOTED
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GEOTECHNICAL EXPLORATION
ASH POND STABILITY
PROFILE - 50 FEET RIGHT
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING
AUTOCAD R 2008
36 C 10W229-22 R 0
PLOT FACTOR: XX
C.A.D. DRAWING
DO NOT ALTER MANUALLY



LOGS OF BORINGS
SCALE: 1"=5' (VERTICAL ONLY)

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NOT FOR CONSTRUCTION

LEGEND

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WH – Weight of Hammer
WR – Weight of Rods
SPT – Standard Penetration Test Interval
ST – Undisturbed Thin-Walled (Shelby) Tube Sample
N60 – Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
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03/31/08 ⊻ – Water Level and Date Recorded
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SCALE AS SHOWN
EXCEPT AS NOTED

YARD
–

GEOTECHNICAL EXPLORATION
ASH POND STABILITY
LOGS OF BORINGS

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

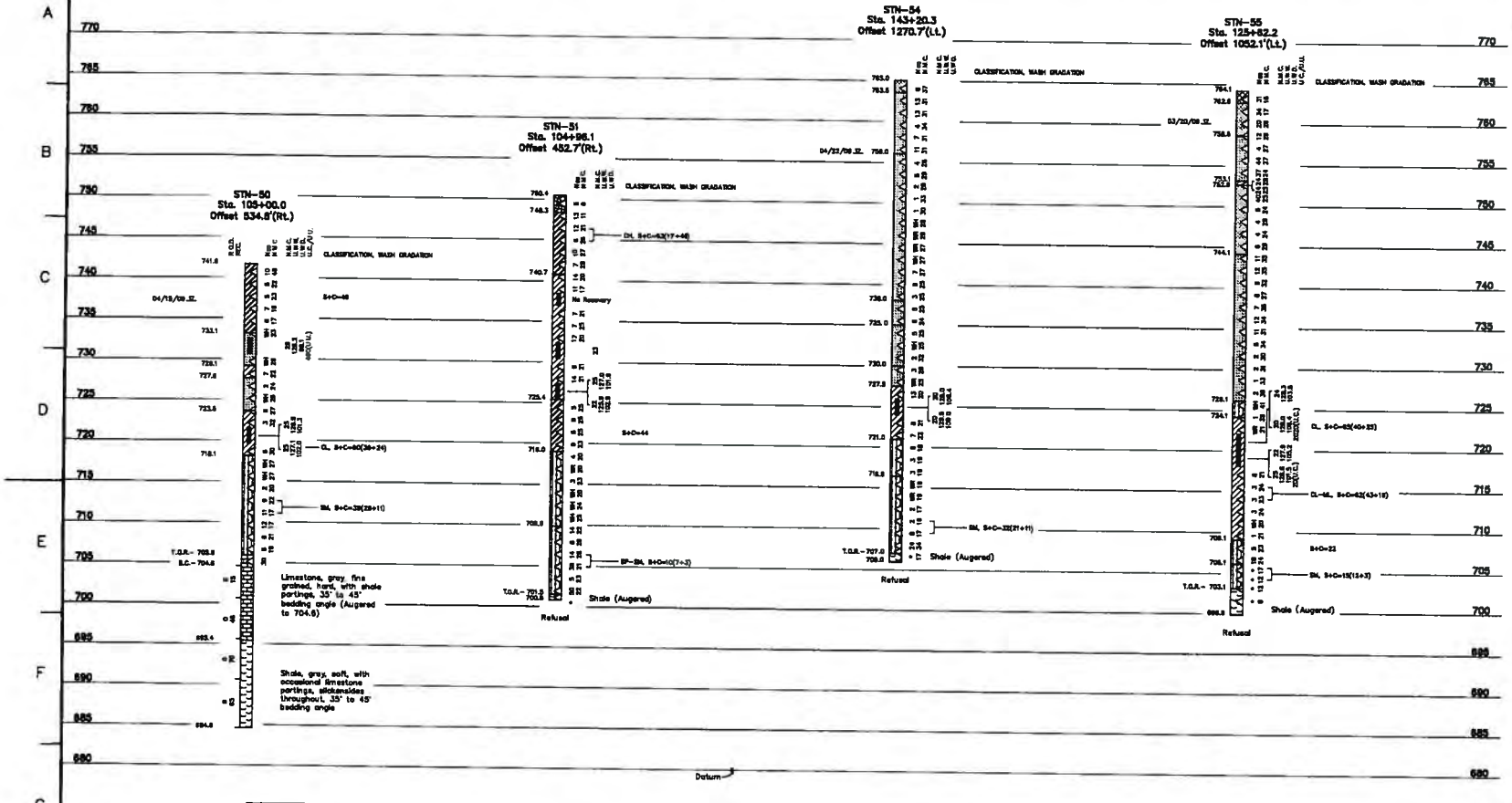
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PLOT FACTOR:XX
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WH Weight of Hammer
WR Weight of Rods
Δ Standard Penetration Test Interval
U Undisturbed Thin-Walled (Shelby) Tube Sample
N60 Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. Natural Moisture Content (%)
U.W. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 ∑ Water Level and Date Recorded
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R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal Auger Refusal using a carbide–tipped tooth auger bit
No Refusal No Refusal Encountered
* Standard Penetration Test (SPT) terminated per ASTM D 1586–99. Refer to typed boring log.

CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS

Location	STN-50	STN-50	STN-51
Depth	20.0'–20.5'	20.5'–21.0'	23.5'–24.0'
ϕ	38.6°		
c	80 p.s.f.		

LOGS OF BORINGS
SCALE: 1"=0' (VERTICAL ONLY)

SCALE: AS SHOWN EXCEPT AS NOTED

YARD
–

GEOTECHNICAL EXPLORATION
ASH POND STABILITY
LOGS OF BORINGS

Stantec Consulting Services Inc.
1409 N. Peters Rd.
Lexington, Kentucky
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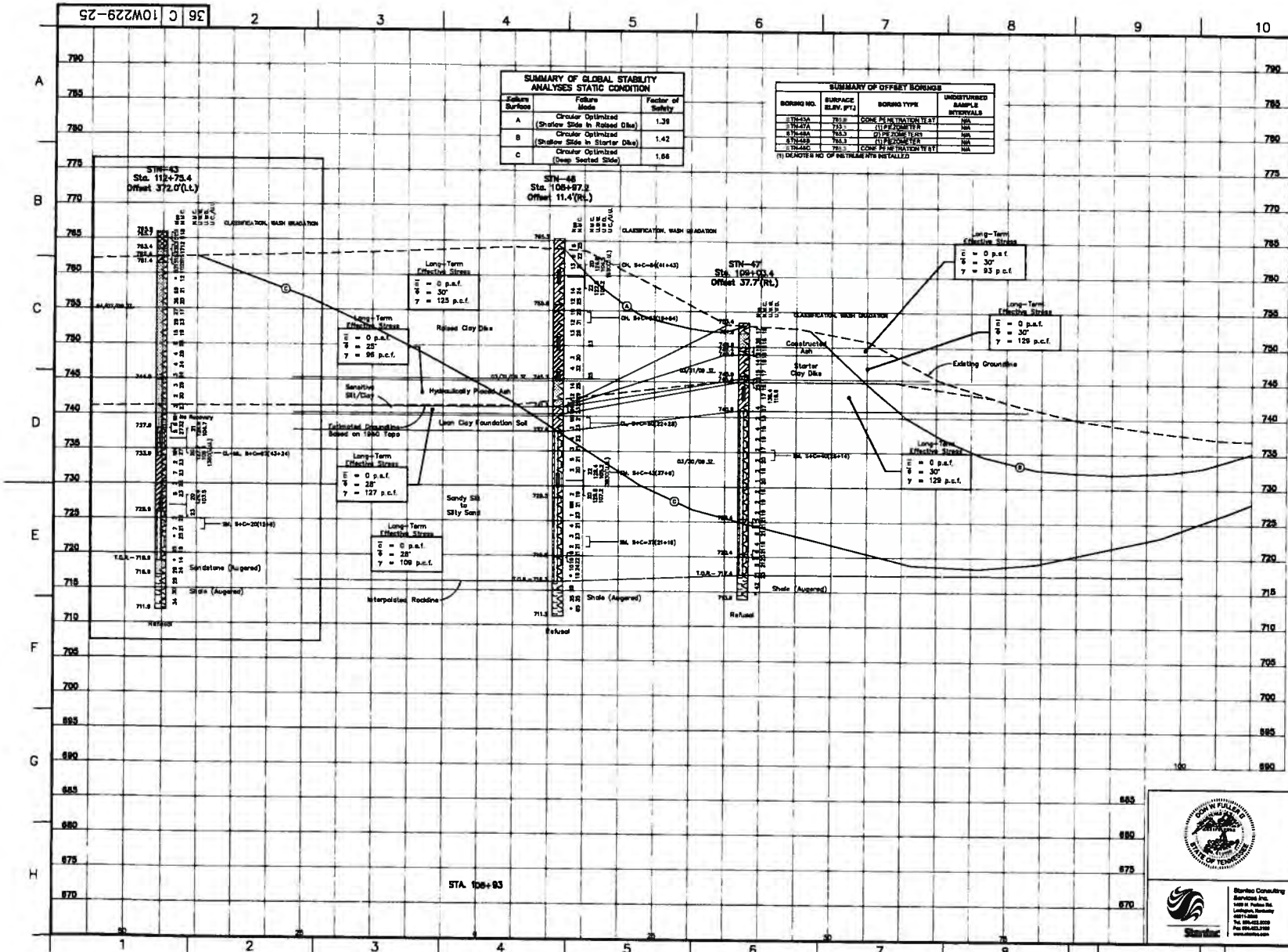
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PLOT FACTOR: XX
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

TASK COMPLETED BY: – REV NO. –



SUMMARY OF GLOBAL STABILITY ANALYSES STATIC CONDITION

Failure Surfaces	Failure Mode	Factor of Safety
A	Circular Optimized (Shallow Slide in Raised Dike)	1.39
B	Circular Optimized (Shallow Slide in Starter Dike)	1.42
C	Circular Optimized (Deep Seated Slide)	1.66

SUMMARY OF OFFSET BORINGS

BORING NO.	SURFACE ELEV. (FT.)	BORING TYPE	UNDISTURBED SAMPLE INTERVALS
① TN-43A	781.8	CONE PENETRATION TEST	N/A
① TN-47A	733.1	(1) PIEZOMETER	N/A
① TN-48A	765.3	(2) PIEZOMETERS	N/A
① TN-48B	765.3	(1) PIEZOMETER	N/A
① TN-48C	781.3	CONE PENETRATION TEST	N/A

(1) DENOTES NO. OF INSTRUMENTS INSTALLED

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NOT FOR CONSTRUCTION

LEGEND

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- Topsoil
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- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH	Weight of Hammer
WR	Weight of Rods
Δ	Standard Penetration Test Interval
≡	Undisturbed Thin-Walled (Shelby) Tube Sample
N60	Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N60	Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C.	Natural Moisture Content (%)
U.W.W.	Unit Weight Wet (lbs./cu.ft.)
U.W.D.	Unit Weight Dry (lbs./cu.ft.)
U.C./U.U	Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 ⌵	Water Level and Date Recorded
T.O.R.–	Top of Rock (Indicates the beginning of rock—the resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C.–	Begin Rock Core
R.Q.D.	Rock Quality Designation (%)
REC.	Recovery (%)
Refusal	Auger Refusal using a carbide-tipped tooth auger bit
No Refusal	No Refusal Encountered
*	Standard Penetration Test (SPT) terminated per ASTM D 1586-98. Refer to typed boring log.

SCALE: 1"=5' EXCEPT AS NOTED

YARD

GEOTECHNICAL EXPLORATION
ASH POND STABILITY
CROSS SECTION

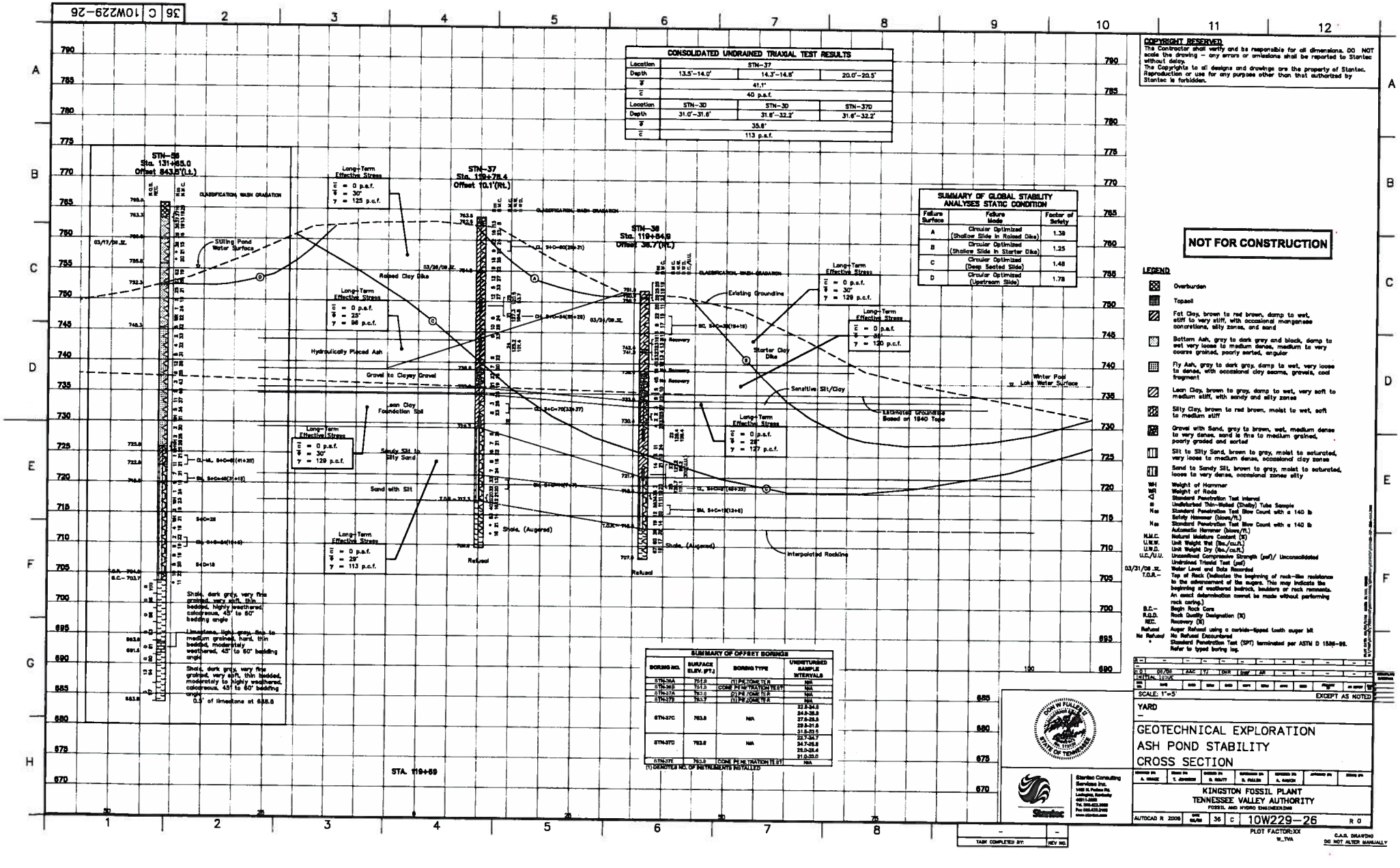
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PLOT FACTOR.XX
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

TASK COMPLETED BY: | REV NO.



CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS

Location	STN-37		
Depth	13.5'–14.0'	14.3'–14.8'	20.0'–20.5'
$\bar{\phi}$	41.1°		
$\bar{c}$	40 p.s.f.		
Location	STN-30	STN-30	STN-370
Depth	31.0'–31.6'	31.6'–32.2'	31.6'–32.2'
$\bar{\phi}$	35.6°		
$\bar{c}$	113 p.s.f.		

SUMMARY OF GLOBAL STABILITY ANALYSES STATIC CONDITION

Failure Surface	Failure Mode	Factor of Safety
A	Circular Optimized (Shallow Slide in Raised Dike)	1.38
B	Circular Optimized (Shallow Slide in Starter Dike)	1.25
C	Circular Optimized (Deep Seated Slide)	1.48
D	Circular Optimized (Upstream Slide)	1.78

SUMMARY OF OFFSET BORINGS

BORING NO.	SURFACE ELEV. (FT.)	BORING TYPE	UNDISTURBED SAMPLE INTERVALS
STN-36A	757.8	(1) PIEZOMETER	NA
STN-36B	757.3	CONE PENETRATION TEST	NA
STN-37A	783.6	(2) PIEZOMETER	NA
STN-37B	783.7	(1) PIEZOMETER	NA
STN-37C	783.8	NA	22.8-24.3 24.8-26.3 27.8-29.3 29.8-31.8 31.8-33.5
STN-37D	783.8	NA	22.7-24.7 24.7-26.8 29.3-32.4 31.0-33.0
STN-37E	783.8	CONE PENETRATION TEST	NA

(1) DENOTES NO. OF INSTRUMENTS INSTALLED

NOT FOR CONSTRUCTION

LEGEND

- Overburden
- Topsoil
- Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
- Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
- Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
- Lean Clay, brown to gray, damp to wet, very soft to medium stiff, with sandy and silty zones
- Silty Clay, brown to red brown, moist to wet, soft to medium stiff
- Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
- Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH — Weight of Hammer
WR — Weight of Rods
ΔS — Standard Penetration Test Interval
ø — Undisturbed Thin-Walled (Shelby) Tube Sample
N60 — Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N60 — Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
M.N.C. — Natural Moisture Content (%)
U.W.W. — Unit Weight Wet (lbs./cu.ft.)
U.W.D. — Unit Weight Dry (lbs./cu.ft.)
UC/U.U. — Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 W. — Water Level and Date Recorded
T.O.R. — Top of Rock (indicates the beginning of rock-like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C. — Begin Rock Core
R.Q.D. — Rock Quality Designation (%)
REC. — Recovery (%)
Refusal — Auger Refusal using a carbide-tipped tooth auger bit
No Refusal — No Refusal Encountered
* — Standard Penetration Test (SPT) terminated per ASTM D 1586-99. Refer to typed boring log.

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SCALE: 1"=5' EXCEPT AS NOTED

YARD

GEOTECHNICAL EXPLORATION
ASH POND STABILITY
CROSS SECTION

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2008 | 36 | C | 10W229-26 | R 0

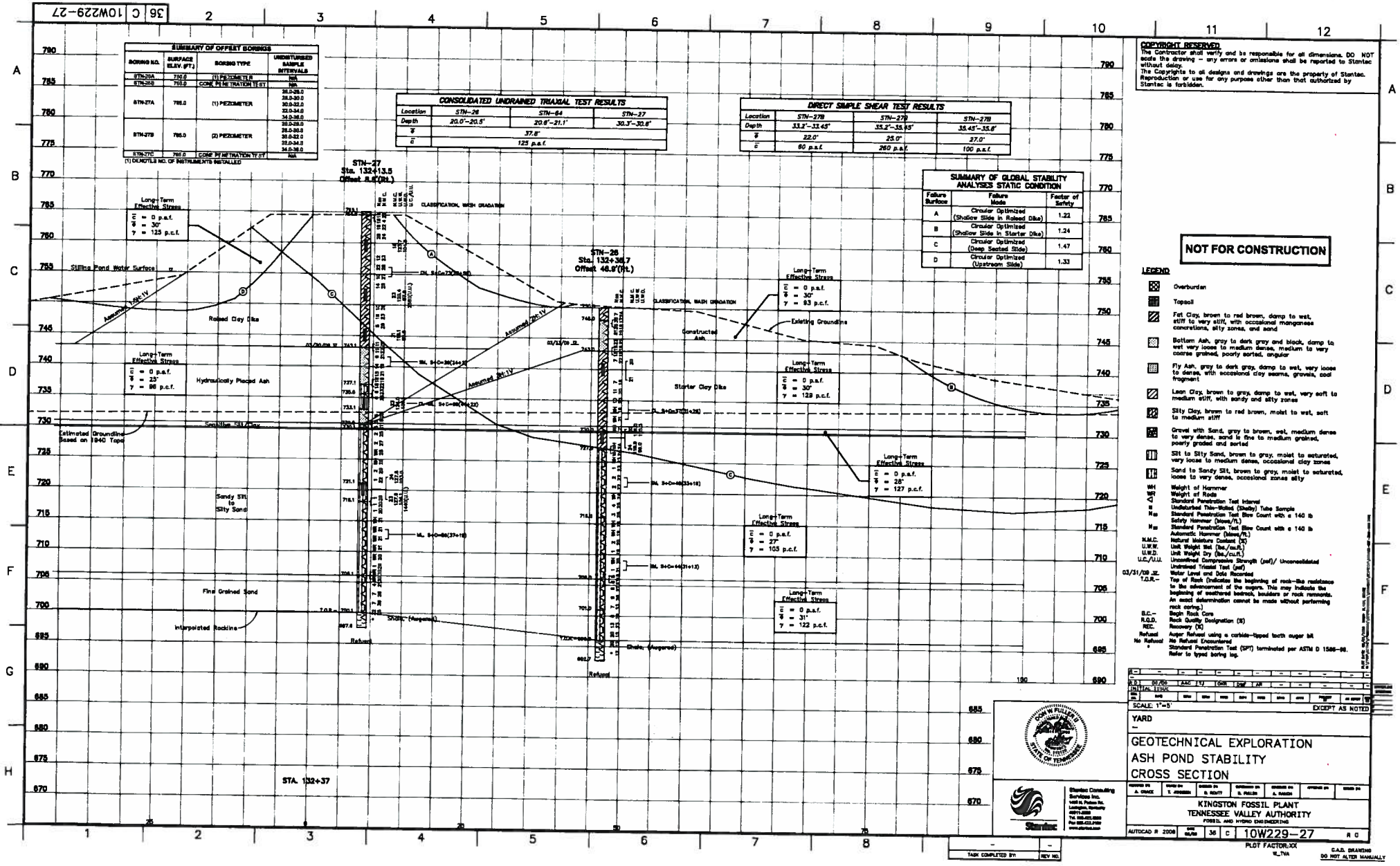
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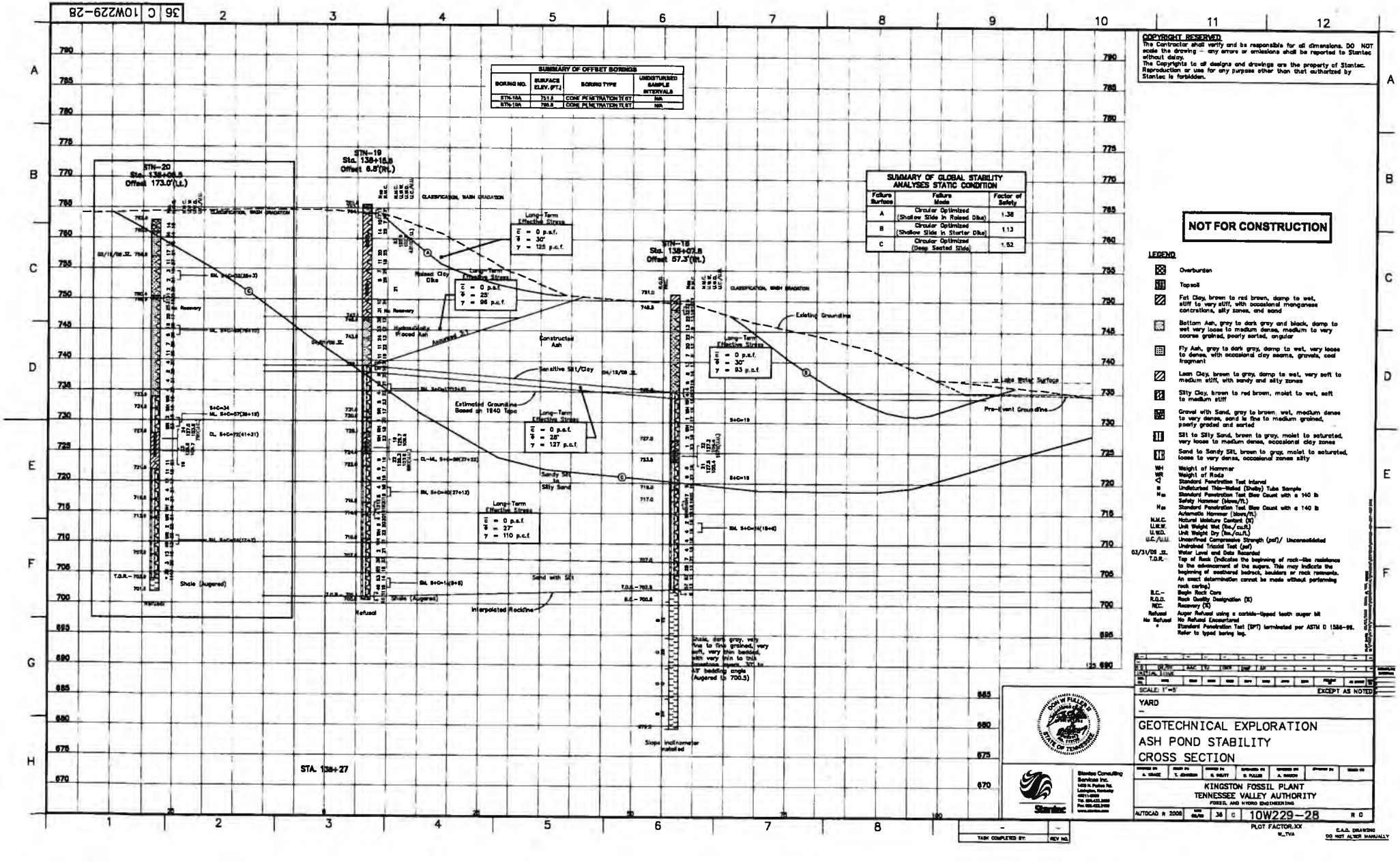
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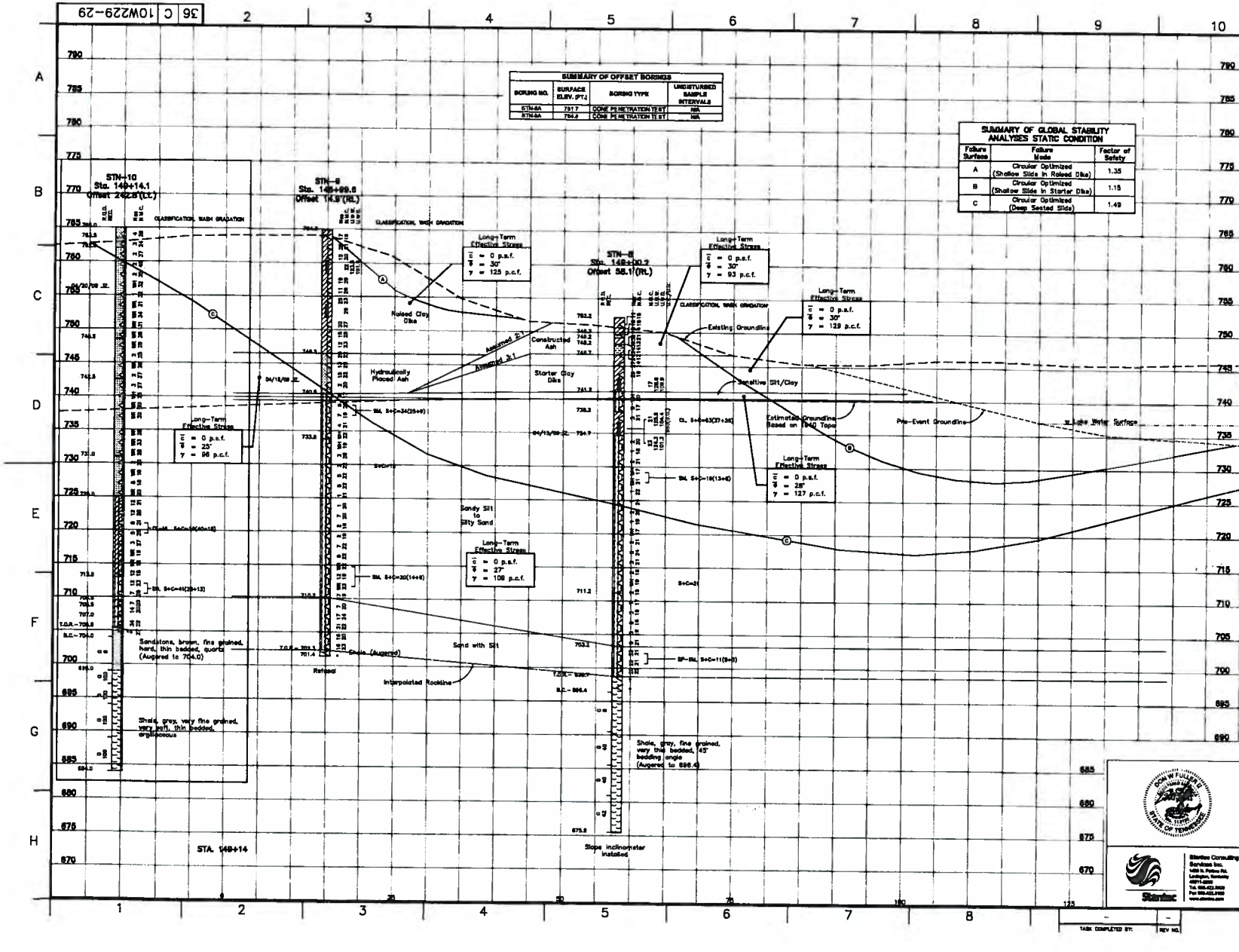
Stantec Consulting Services Inc.
1600 N. Parkes Rd.
Lexington, Kentucky
40511-2050
Tel. 859.422.3000
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www.stantec.com

TASK COMPLETED BY: — REV NO. —

36 C 10W229-26


SUMMARY OF OFFSET BORINGS
BORING NO. | SURFACE ELEV. (FT.) | BORING TYPE | UNDISTURBED SAMPLE INTERVALS
STN-26A | 760.0 | (1) PIEZOMETER | NA
STN-26B | 760.0 | CONE PENETRATION TEST | NA
STN-27A | 765.0 | (1) PIEZOMETER | 26.0-28.0, 28.0-30.0, 30.0-32.0, 32.0-34.0, 34.0-36.0
STN-27B | 765.0 | (2) PIEZOMETER | 30.0-28.0, 28.0-30.0, 30.0-32.0, 32.0-34.0, 34.0-36.0
STN-27C | 765.0 | CONE PENETRATION TEST | NA
(1) DENOTES NO. OF INSTRUMENTS INSTALLED
CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS
Location | STN-26 | STN-64 | STN-27
Depth | 20.0'-20.5' | 20.6'-21.1' | 30.3'-30.8'
ϕ' | 37.6°
c' | 125 p.s.f.
DIRECT SIMPLE SHEAR TEST RESULTS
Location | STN-27B | STN-27B | STN-27B
Depth | 33.2'-33.45' | 35.2'-35.45' | 35.45'-35.8'
ϕ' | 22.0° | 25.0° | 27.0°
c' | 60 p.s.f. | 260 p.s.f. | 100 p.s.f.
SUMMARY OF GLOBAL STABILITY ANALYSES STATIC CONDITION
Failure Surface | Failure Mode | Factor of Safety
A | Circular Optimized (Shallow Slide in Raised Dike) | 1.22
B | Circular Optimized (Shallow Slide in Starter Dike) | 1.24
C | Circular Optimized (Deep Seated Slide) | 1.47
D | Circular Optimized (Upstream Slide) | 1.33
STN-27 Sta. 132+13.5 Offset 8.6'(Rt.)
STN-28 Sta. 132+36.7 Offset 46.8'(Rt.)
Long-Term Effective Stress c' = 0 p.s.f. ϕ' = 30° γ = 123 p.c.f.
Stilling Pond Water Surface
Assumed 2(H):1(V)
Raised Clay Dike
Long-Term Effective Stress c' = 0 p.s.f. ϕ' = 25° γ = 86 p.c.f.
Hydraulically Placed Ash
Estimated Groundline Based on 1940 Topo
Sandy Silt to Silty Sand
Fine Grained Sand
Interpolated Rockline
Refusal
Shale (Augered)
Long-Term Effective Stress c' = 0 p.s.f. ϕ' = 30° γ = 93 p.c.f.
Existing Groundline
Constructed Ash
Starter Clay Dike
Long-Term Effective Stress c' = 0 p.s.f. ϕ' = 30° γ = 129 p.c.f.
Long-Term Effective Stress c' = 0 p.s.f. ϕ' = 28° γ = 127 p.c.f.
Long-Term Effective Stress c' = 0 p.s.f. ϕ' = 27° γ = 105 p.c.f.
Long-Term Effective Stress c' = 0 p.s.f. ϕ' = 31° γ = 122 p.c.f.
STA. 132+37
CLASSIFICATION, WASH GRADATION
03/31/09 W.L.
T.O.R.
B.C.
R.C.D.
Refusal
No Refusal
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NOT FOR CONSTRUCTION
LEGEND
Overburden
Topsoil
Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
Lean Clay, brown to gray, damp to wet, very soft to medium stiff, with sandy and silty zones
Silty Clay, brown to red brown, moist to wet, soft to medium stiff
Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty
WH Weight of Hammer
WR Weight of Rods
Δ Standard Penetration Test Interval
Ⅱ Undisturbed Thin-Walled (Shelby) Tube Sample
N60 Standard Penetration Test Blow Count with a 140 lb Safety Hammer (Blows/ft.)
N Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (Blows/ft.)
N.M.C. Natural Moisture Content (%)
U.W.W. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 W. Water Level and Date Recorded
T.O.R. Top of Rock (Indicates the beginning of rock-like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C. Begin Rock Core
R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal Auger Refusal using a carbide-tipped tooth auger bit
No Refusal No Refusal Encountered
* Standard Penetration Test (SPT) terminated per ASTM D 1586-99. Refer to typed boring log.
SCALE: 1"=5'
EXCEPT AS NOTED
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GEOTECHNICAL EXPLORATION
ASH POND STABILITY
CROSS SECTION
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TASK COMPLETED BY
REV NO.


SUMMARY OF OFFSET BORINGS
BORING NO. | SURFACE ELEV. (FT.) | BORING TYPE | UNDISTURBED SAMPLE INTERVALS
STN-18A | 751.9 | CONE PENETRATION TEST | NA
STN-18A | 780.6 | CONE PENETRATION TEST | NA
SUMMARY OF GLOBAL STABILITY ANALYSES STATIC CONDITION
Failure Surface | Failure Mode | Factor of Safety
A | Circular Optimized (Shallow Slide in Raised Dike) | 1.38
B | Circular Optimized (Shallow Slide in Starter Dike) | 1.13
C | Circular Optimized (Deep Seated Slide) | 1.52
STN-20
Sta. 138+08.5
Offset 173.0'(Lt.)
STN-19
Sta. 138+15.5
Offset 6.5'(Rt.)
STN-18
Sta. 138+07.5
Offset 57.3'(Rt.)
Raised Clay Dike
Hydraulically Placed Ash
Constructed Ash
Sensitive Silt/Clay
Estimated Groundline Based on 1940 Topo
Existing Groundline
Pre-Event Groundline
Sandy Silt to Silty Sand
Shale (Augered)
Refusal
Interpolated Rockline
Long-Term Effective Stress
c' = 0 p.s.f.
φ' = 30°
γ = 125 p.c.f.
c' = 0 p.s.f.
φ' = 25°
γ = 98 p.c.f.
c' = 0 p.s.f.
φ' = 30°
γ = 83 p.c.f.
c' = 0 p.s.f.
φ' = 28°
γ = 127 p.c.f.
c' = 0 p.s.f.
φ' = 27°
γ = 110 p.c.f.
STA. 138+27
NOT FOR CONSTRUCTION
LEGEND
Overburden
Topsoil
Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
Lean Clay, brown to gray, damp to wet, very soft to medium stiff, with sandy and silty zones
Silty Clay, brown to red brown, moist to wet, soft to medium stiff
Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty
WH Weight of Hammer
WR Weight of Rods
Standard Penetration Test Interval
Undisturbed Thin-Walled (Shelby) Tube Sample
Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. Natural Moisture Content (%)
U.W.W. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/05 ▽ Water Level and Date Recorded
T.O.R. Top of Rock (indicates the beginning of rock-like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring)
B.C. Begin Rock Core
R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal Auger Refusal using a carbide-tipped tooth auger bit
No Refusal No Refusal Encountered
* Standard Penetration Test (SPT) terminated per ASTM D 1586-98. Refer to typed boring log.
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PLOT FACTOR.XX
W_TVA
C.A.D. DRAWING DO NOT ALTER MANUALLY
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SUMMARY OF OFFSET BORINGS

BORING NO.	SURFACE ELEV. (FT.)	BORING TYPE	UNDISTURBED SAMPLE INTERVALS
STN-8A	781.7	CONE PENETRATION TEST	N/A
STN-9A	784.5	CONE PENETRATION TEST	N/A

SUMMARY OF GLOBAL STABILITY ANALYSES STATIC CONDITION

Failure Surface	Failure Mode	Factor of Safety
A	Circular Optimized (Shallow Slide in Raised Dike)	1.35
B	Circular Optimized (Shallow Slide in Starter Dike)	1.15
C	Circular Optimized (Deep Seated Slide)	1.49

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NOT FOR CONSTRUCTION

LEGEND

- Overburden
- Topsoil
- Fat Clay, brown to red brown, damp to wet, stiff to very stiff, with occasional manganese concretions, silty zones, and sand
- Bottom Ash, gray to dark gray and black, damp to wet very loose to medium dense, medium to very coarse grained, poorly sorted, angular
- Fly Ash, gray to dark gray, damp to wet, very loose to dense, with occasional clay seams, gravels, coal fragment
- Lean Clay, brown to gray, damp to wet, very soft to medium stiff, with sandy and silty zones
- Silty Clay, brown to red brown, moist to wet, soft to medium stiff
- Gravel with Sand, gray to brown, wet, medium dense to very dense, sand is fine to medium grained, poorly graded and sorted
- Silt to Silty Sand, brown to gray, moist to saturated, very loose to medium dense, occasional clay zones
- Sand to Sandy Silt, brown to gray, moist to saturated, loose to very dense, occasional zones silty

WH Weight of Hammer
WR Weight of Rods
Δ Standard Penetration Test Interval
U Undisturbed Thin-Walled (Shelby) Tube Sample
N60 Standard Penetration Test Blow Count with a 140 lb Safety Hammer (blows/ft.)
N60 Standard Penetration Test Blow Count with a 140 lb Automatic Hammer (blows/ft.)
N.M.C. Natural Moisture Content (%)
U.W.M. Unit Weight Wet (lbs./cu.ft.)
U.W.D. Unit Weight Dry (lbs./cu.ft.)
U.C./U.U. Unconfined Compressive Strength (psf)/ Unconsolidated Undrained Triaxial Test (psf)
03/31/09 ∇ Water Level and Date Recorded
T.O.R.- Top of Rock (Indicates the beginning of rock-like resistance to the advancement of the augers. This may indicate the beginning of weathered bedrock, boulders or rock remnants. An exact determination cannot be made without performing rock coring.)
B.C.- Begin Rock Core
R.Q.D. Rock Quality Designation (%)
REC. Recovery (%)
Refusal No Refusal Auger Refusal using a carbide-tipped tooth auger bit
No Refusal Encountered
* Standard Penetration Test (SPT) terminated per ASTM D 1586-98. Refer to typed boring log.

SCALE: 1"=5' EXCEPT AS NOTED

YARD

GEOTECHNICAL EXPLORATION
ASH POND STABILITY
CROSS SECTION

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2008 | 36 | C | 10W229-29 | R 0

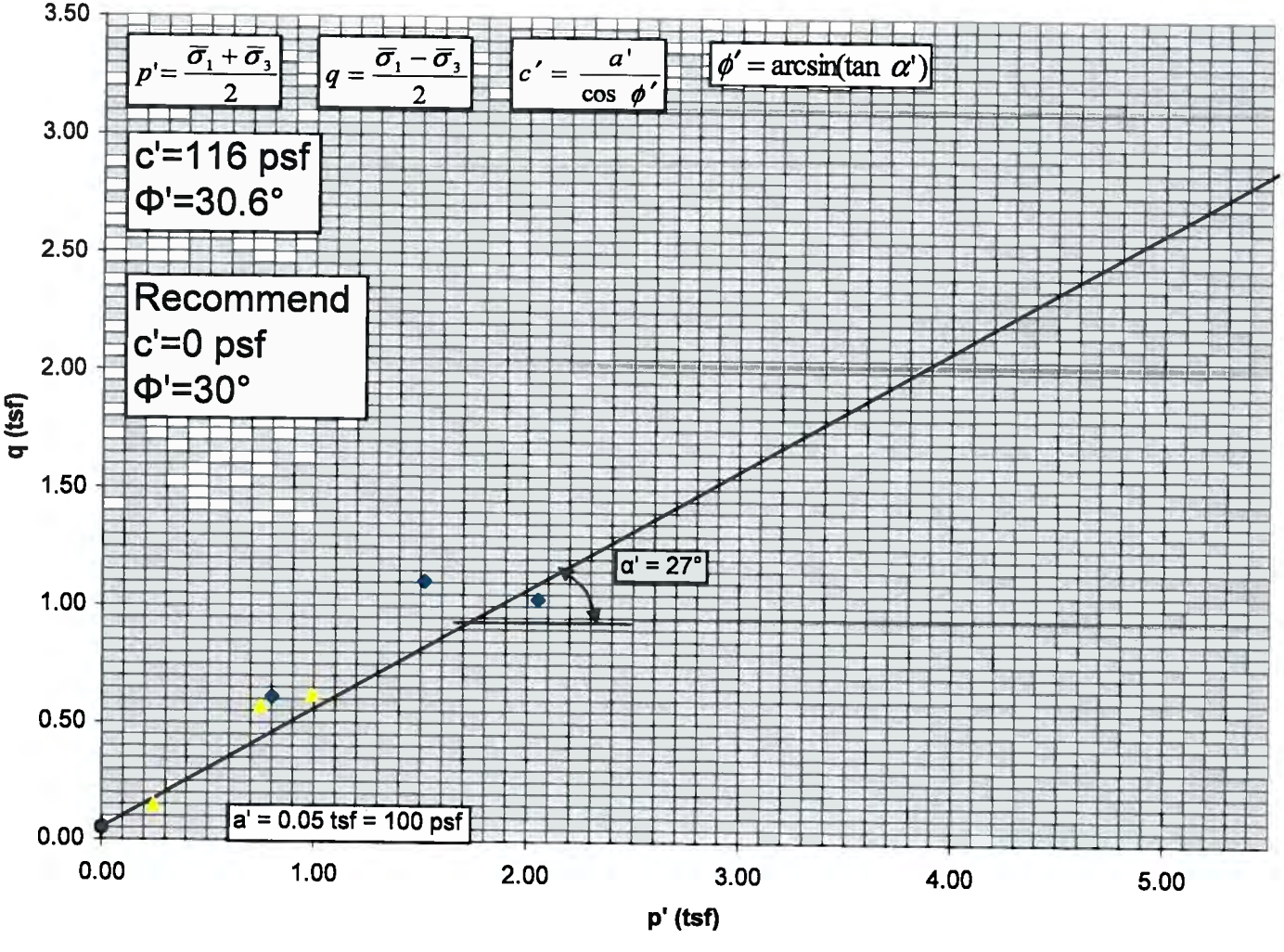
PLOT FACTOR.XX
W_TVA

C.A.D. DRAWING DO NOT ALTER MANUALLY

Appendix G

Strength Parameter Selection Charts

Raised Clay Dike
Effective Stress from CU Triaxial Tests

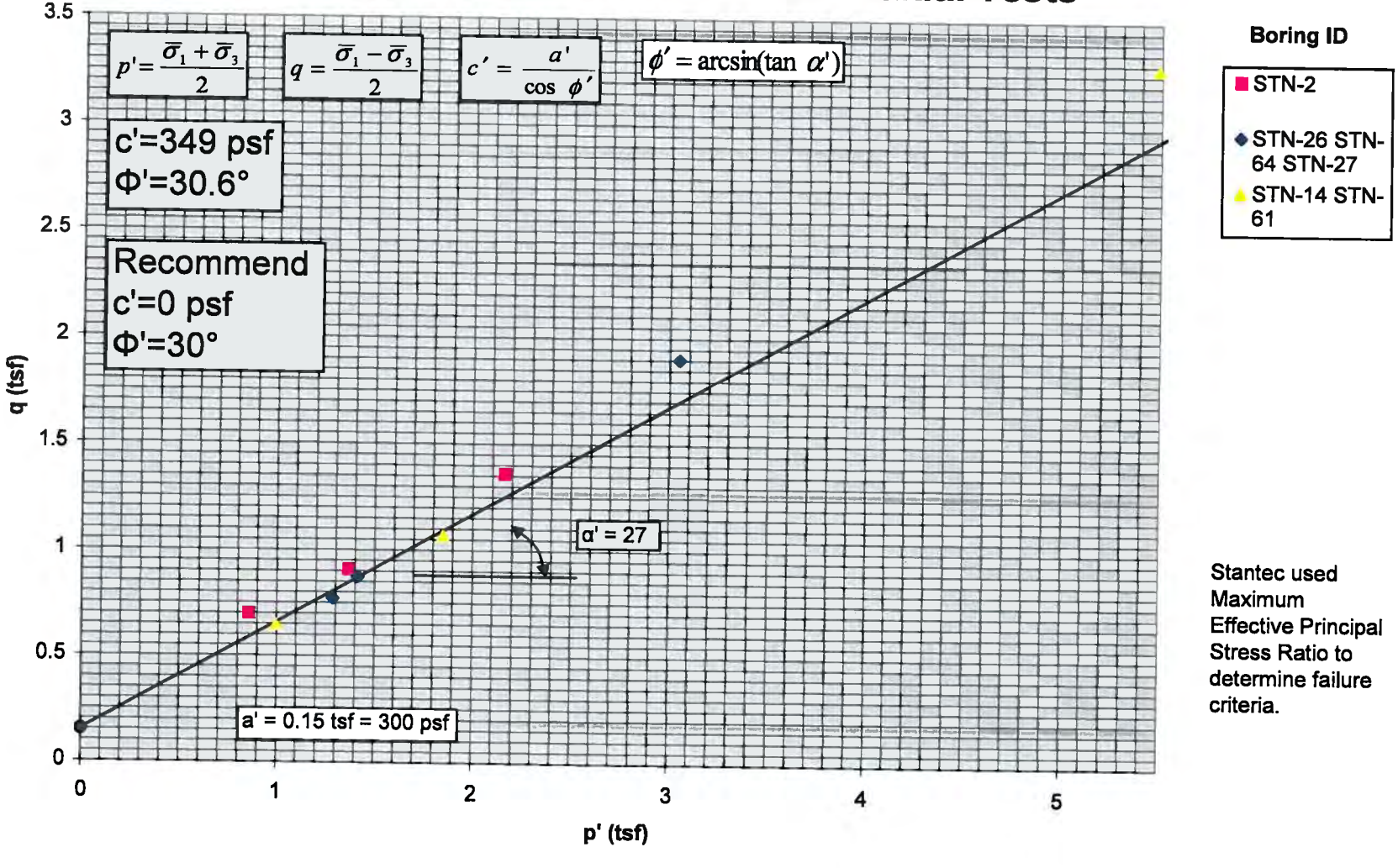


Boring ID

- STN-3 STN-5
- STN-37D

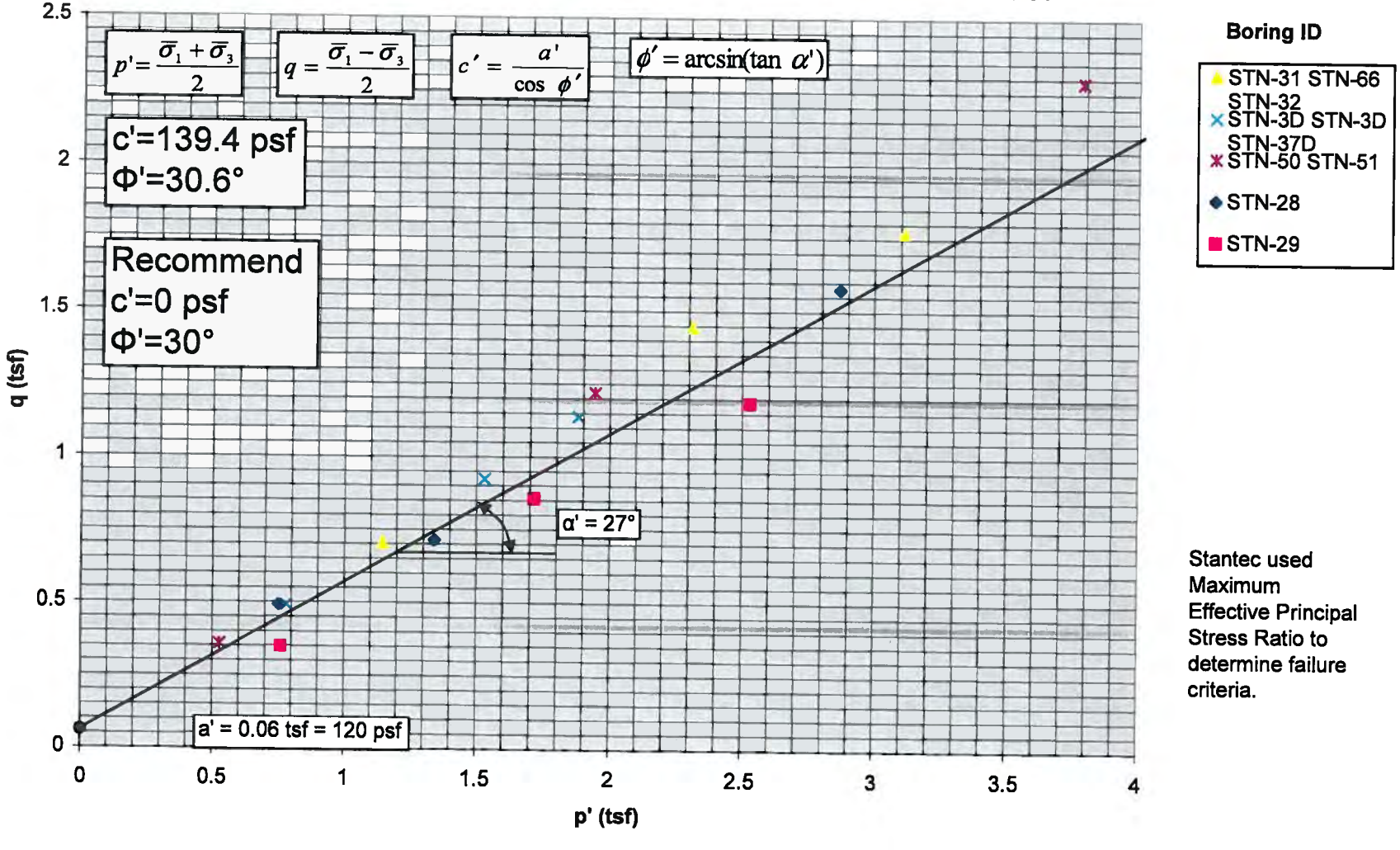
Stantec used Maximum Effective Principal Stress Ratio to determine failure criteria.

Starter Clay Dike
Effective Stress from CU Triaxial Tests



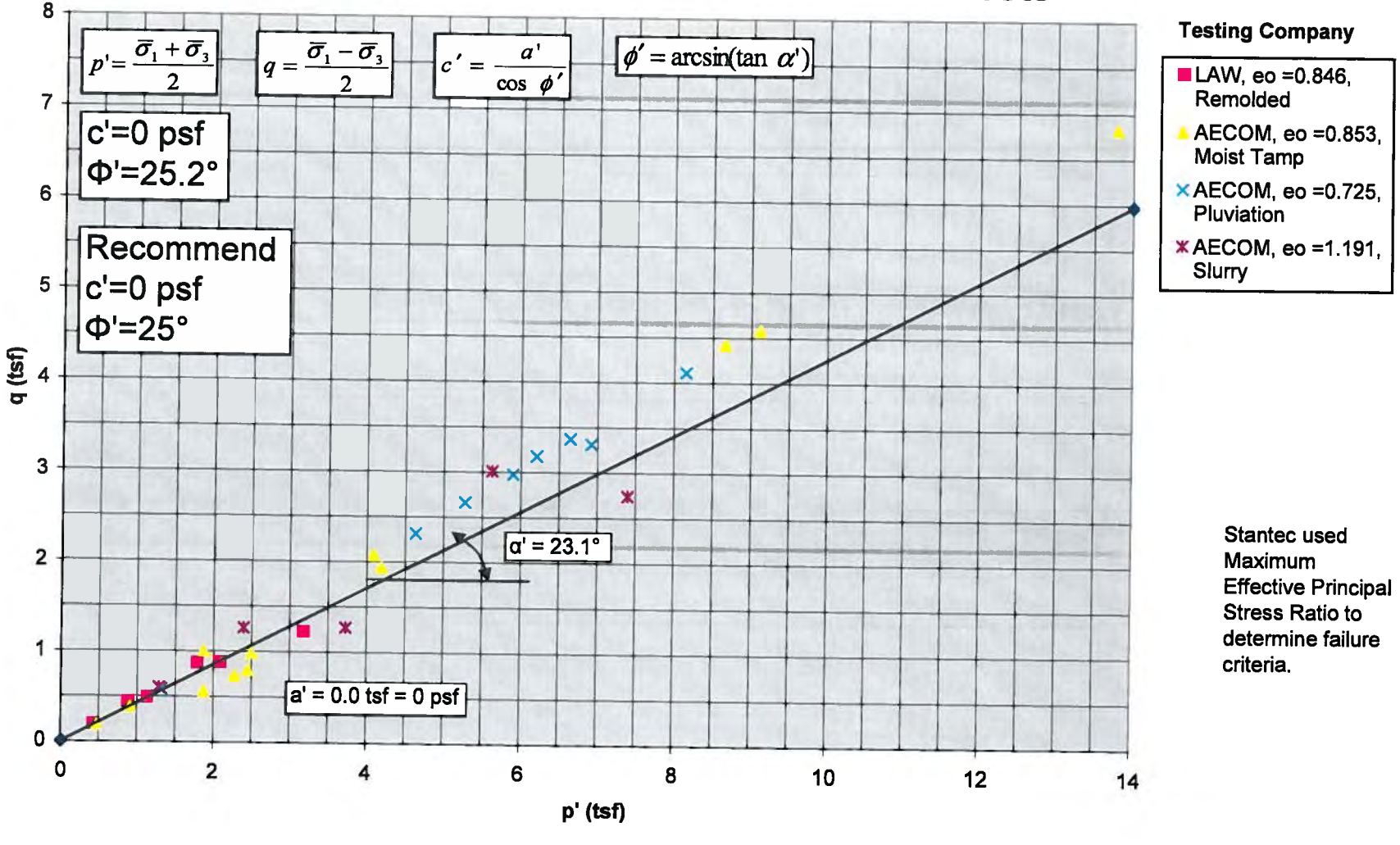
Stantec used Maximum Effective Principal Stress Ratio to determine failure criteria.

Foundation Clay
Effective Stress from CU Triaxial Tests



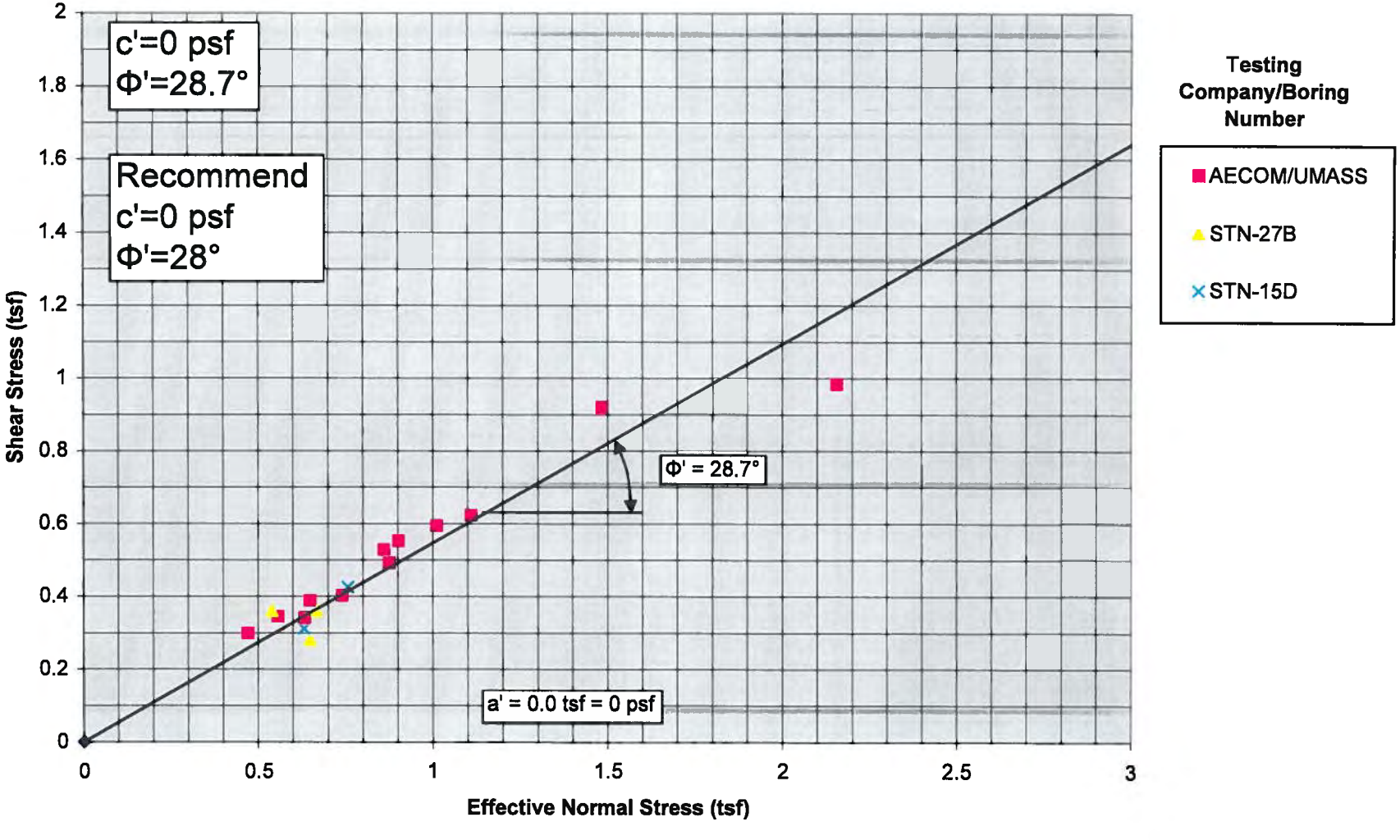
Stantec used Maximum Effective Principal Stress Ratio to determine failure criteria.

Hydraulically Placed Ash
Effective Stress from CU Triaxial Tests



Stantec used Maximum Effective Principal Stress Ratio to determine failure criteria.

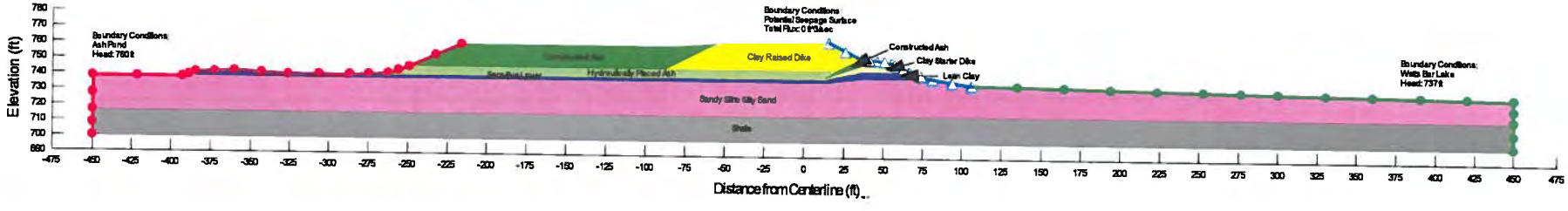
DSS Testing on Sensitive Silt/Clay
Effective Stress from DSS Tests



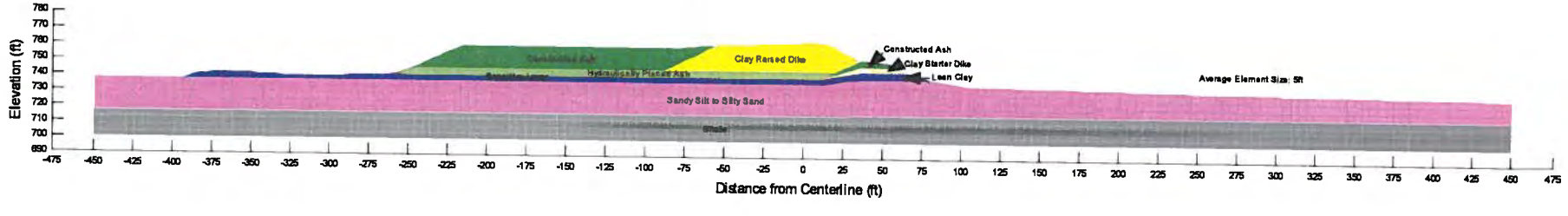
Appendix H

Pore Water Pressure Contours

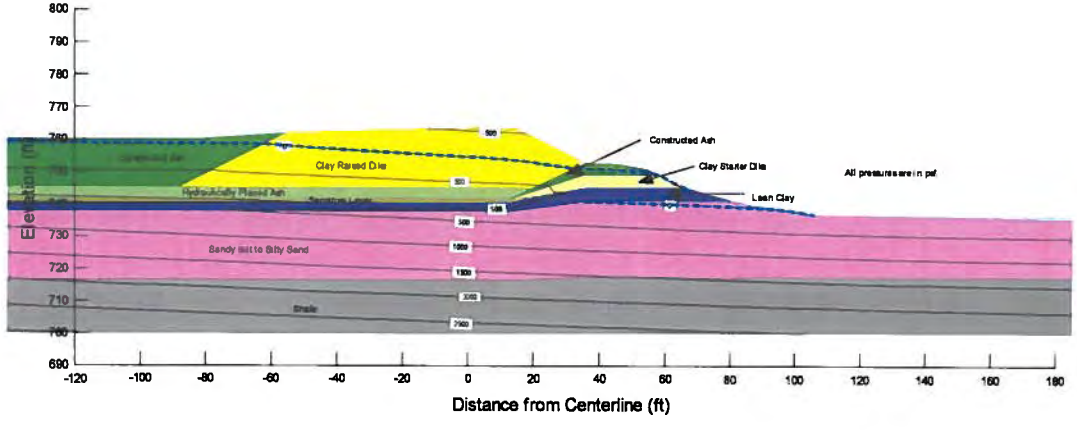
Kingston Fossil Plant
Dike C - Station 108+93
Subsurface Profile and Boundary Conditions



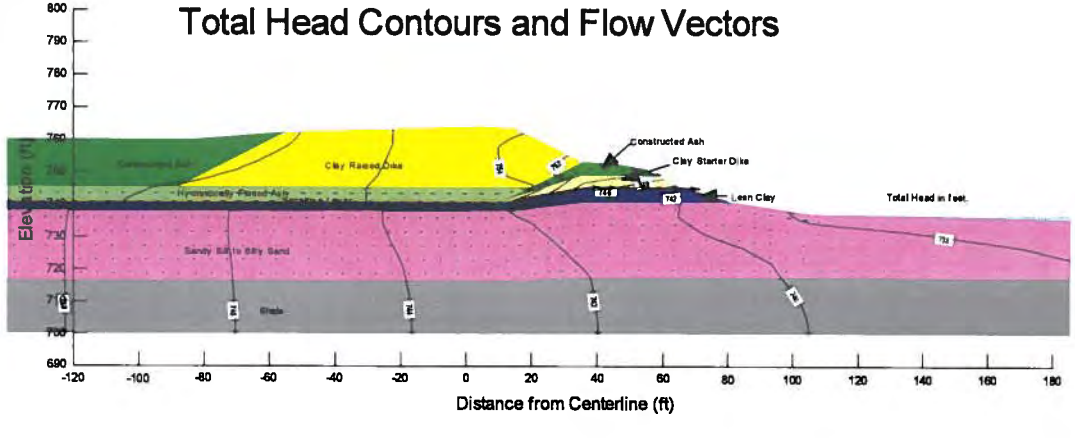
Kingston Fossil Plant
Dike C - Station 108+93
Finite Element Mesh



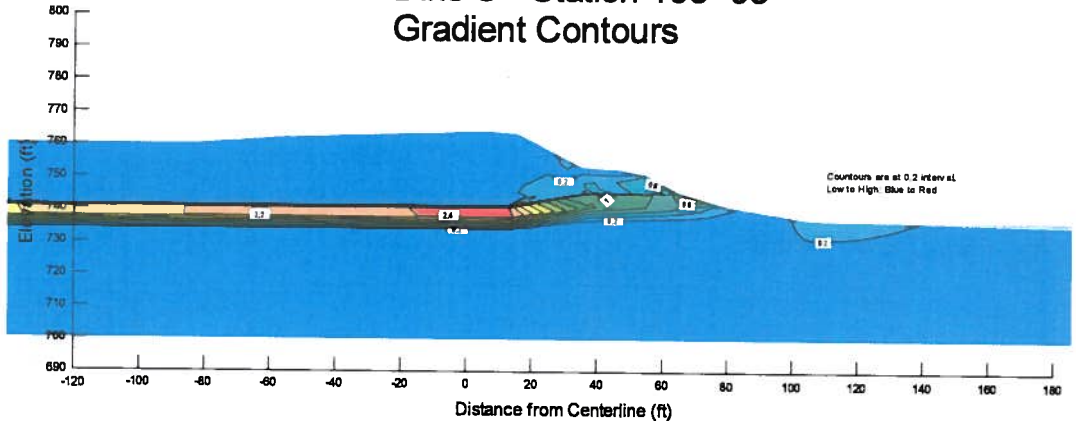
Kingston Fossil Plant
Dike C - Station 108+93
Pore Water Pressure Contours



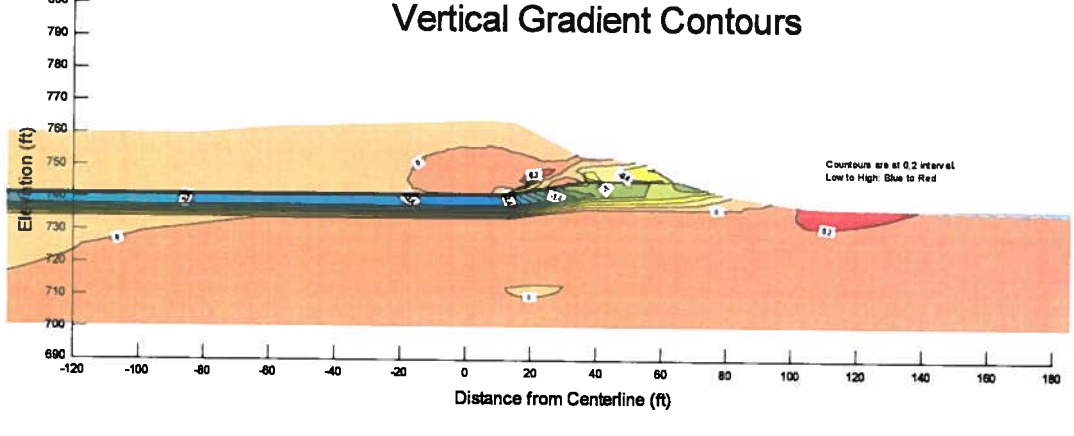
Kingston Fossil Plant
Dike C - Station 108+93
Total Head Contours and Flow Vectors



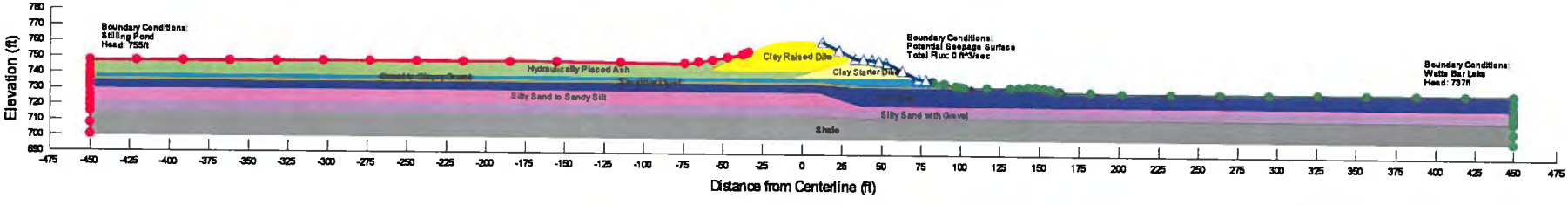
Kingston Fossil Plant
Dike C - Station 108+93
Gradient Contours



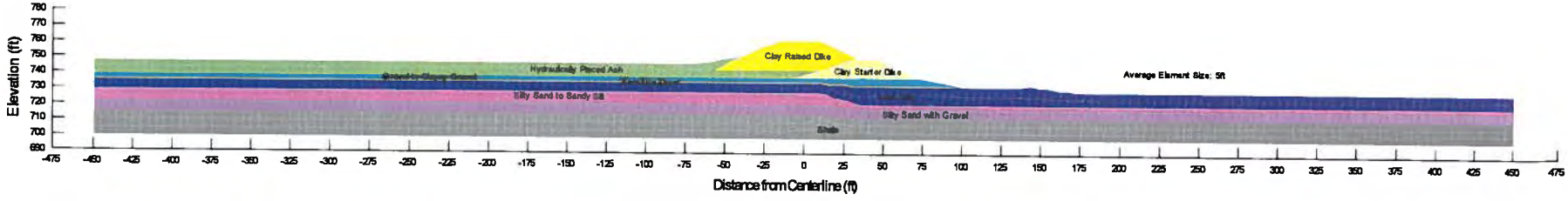
Kingston Fossil Plant
Dike C - Station 108+93
Vertical Gradient Contours



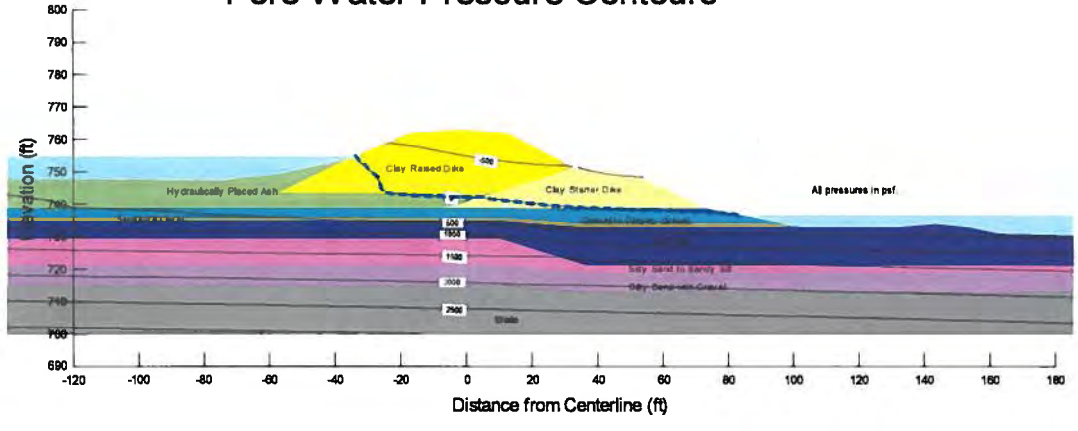
Kingston Fossil Plant
Dike C - Station 119+69
Subsurface Profile and Boundary Conditions



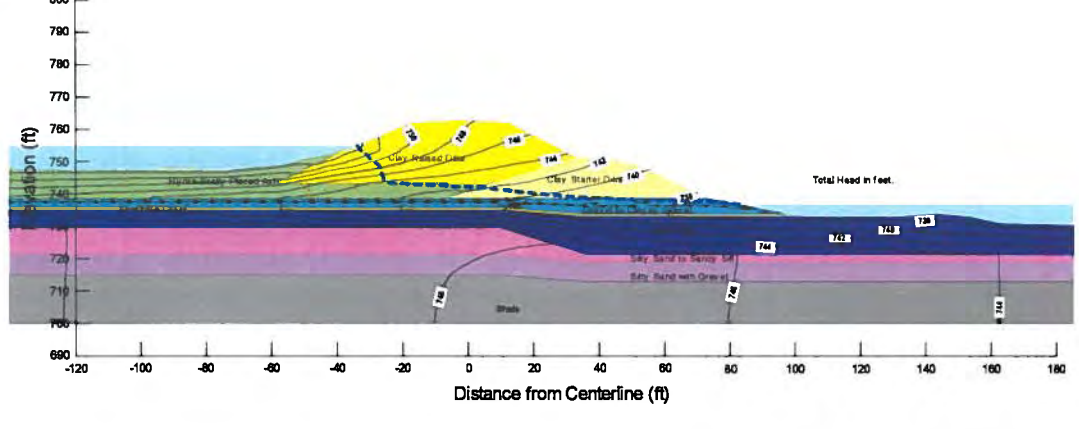
Kingston Fossil Plant
Dike C - Station 119+69
Finite Element Mesh



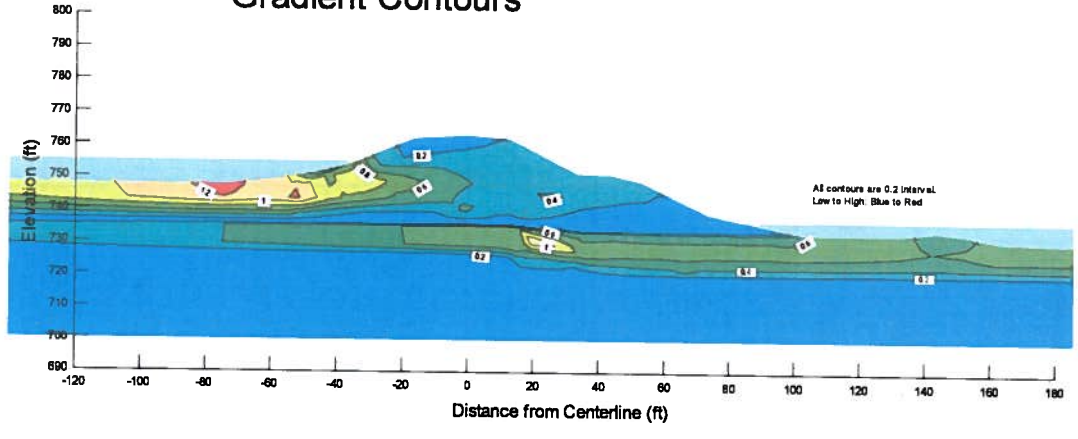
Kingston Fossil Plant
Dike C - Station 119+69
Pore Water Pressure Contours



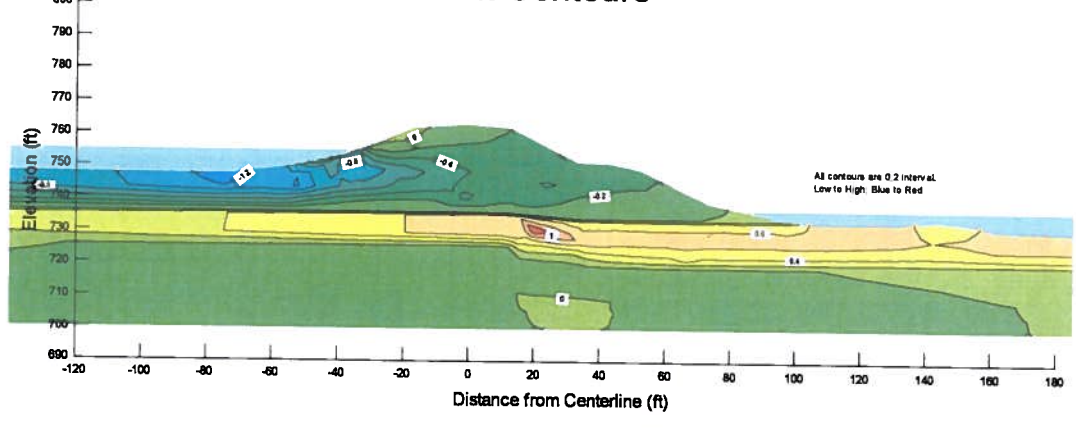
Kingston Fossil Plant
Dike C - Station 119+69
Total Head Contours and Flow Vectors



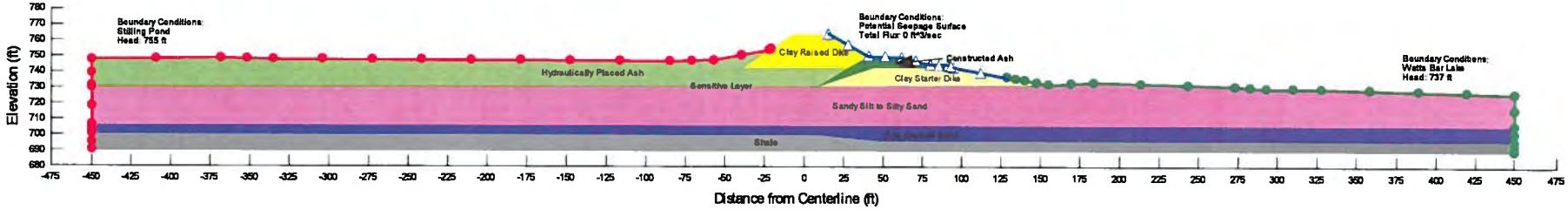
Kingston Fossil Plant
Dike C - Station 119+69
Gradient Contours



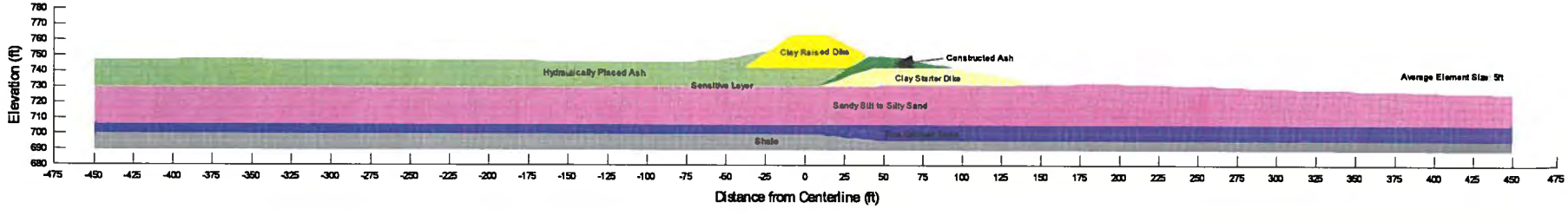
Kingston Fossil Plant
Dike C - Station 119+69
Vertical Gradient Contours



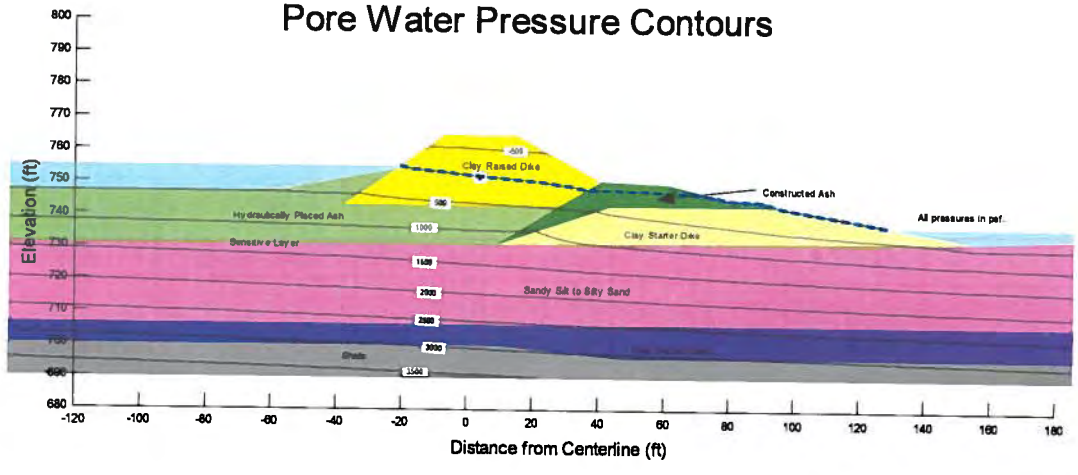
Kingston Fossil Plant
Dike C - Station 132+37
Subsurface Profile and Boundary Conditions



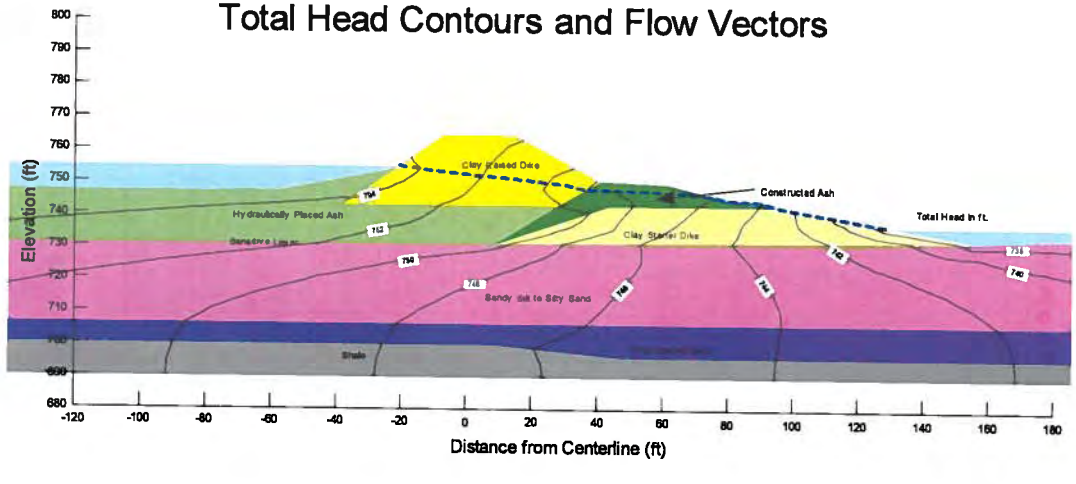
Kingston Fossil Plant
Dike C - Station 132+37
Finite Element Mesh



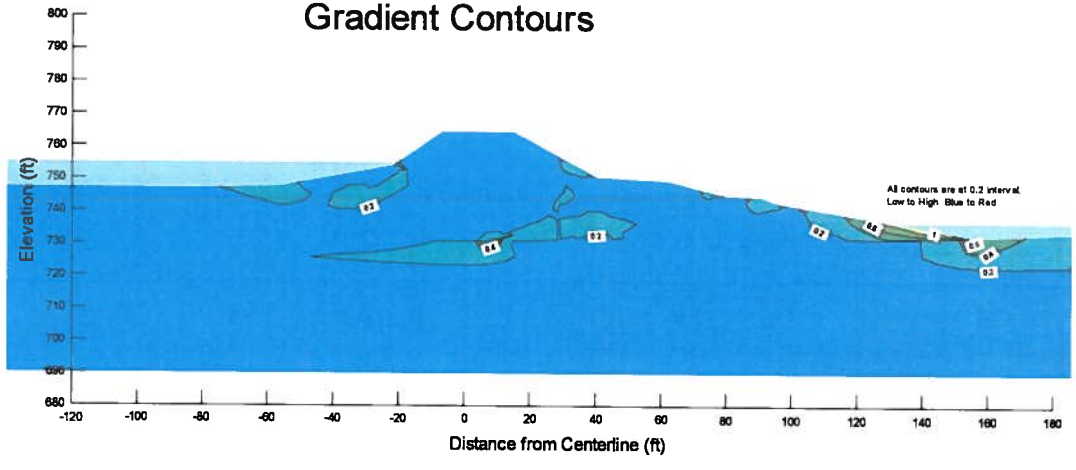
Kingston Fossil Plant
Dike C - Station 132+37
Pore Water Pressure Contours



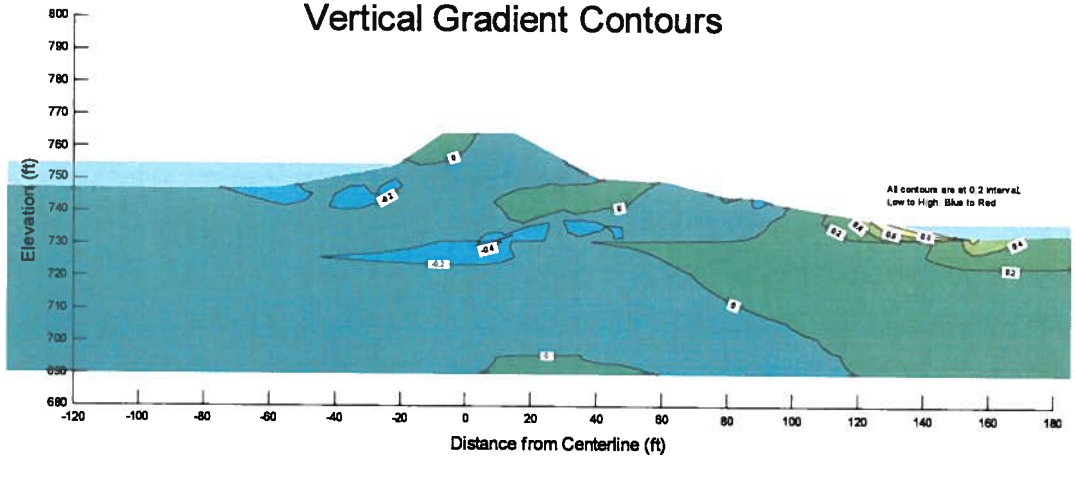
Kingston Fossil Plant
Dike C - Station 132+37
Total Head Contours and Flow Vectors



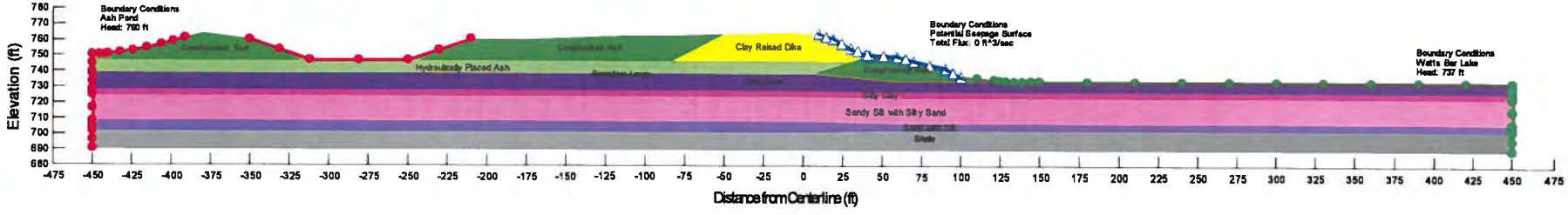
Kingston Fossil Plant
Dike C - Station 132+37
Gradient Contours



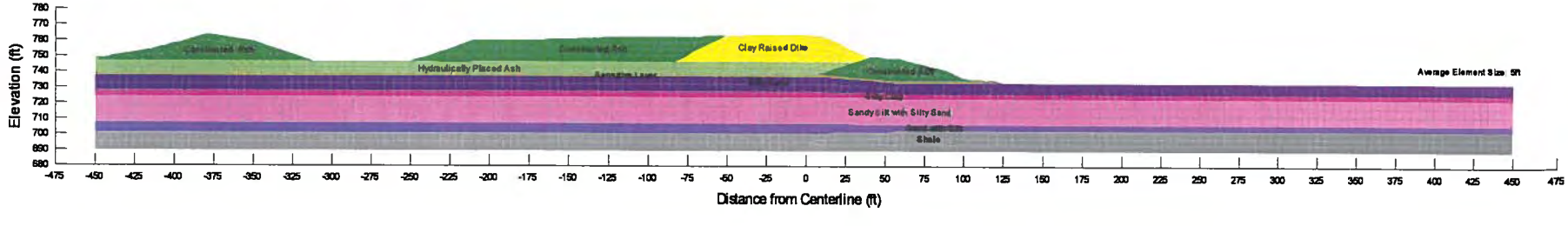
Kingston Fossil Plant
Dike C - Station 132+37
Vertical Gradient Contours



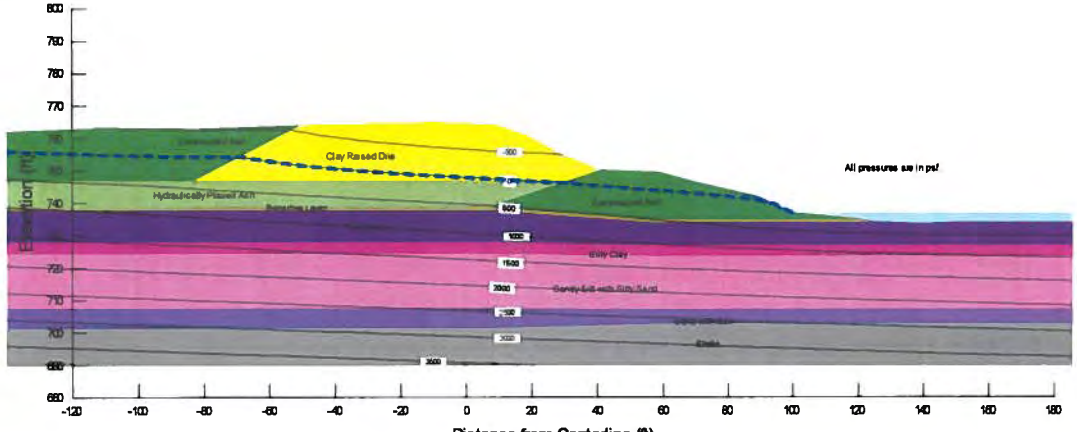
Kingston Fossil Plant
Dike C - Station 138+27
Subsurface Profile and Boundary Conditions



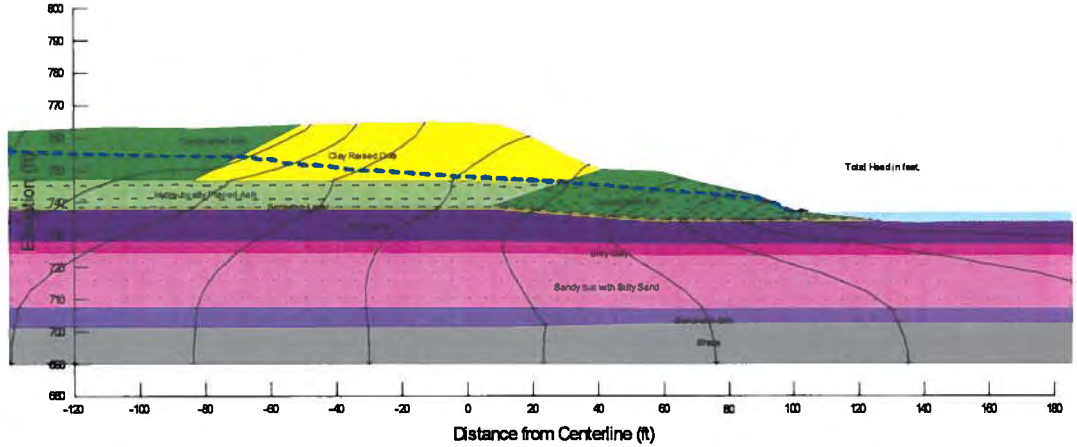
Kingston Fossil Plant
Dike C - Station 138+27
Finite Element Mesh



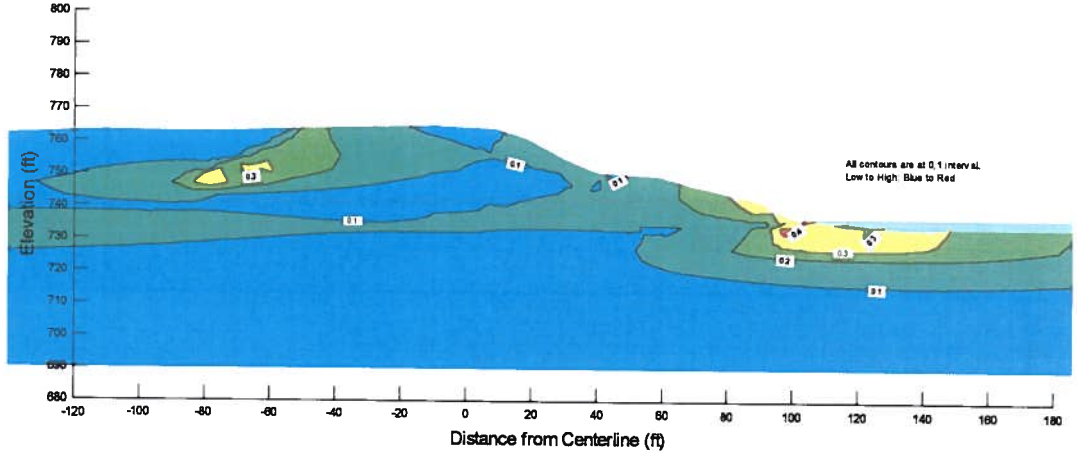
Kingston Fossil Plant
Dike C - Station 138+27
Pore Water Pressure Contours



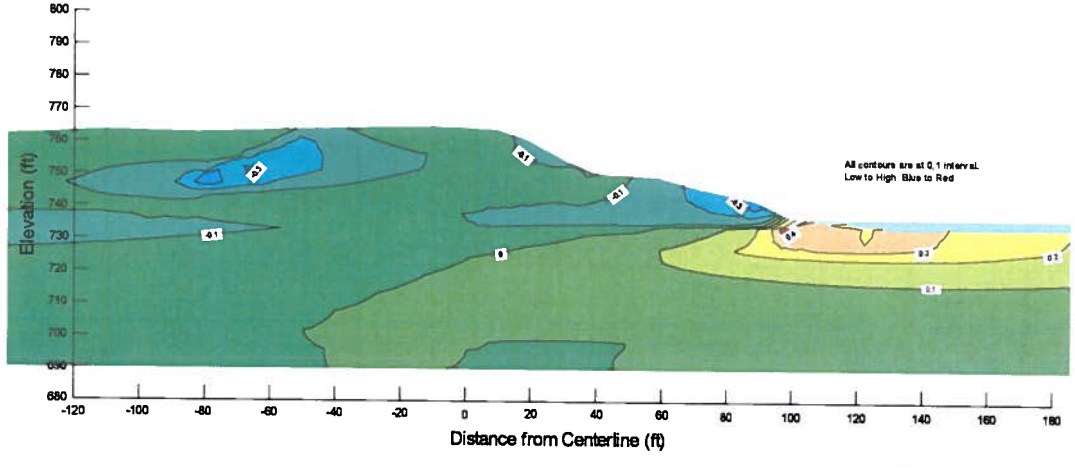
Kingston Fossil Plant
Dike C - Station 138+27
Total Head Contours and Flow Vectors



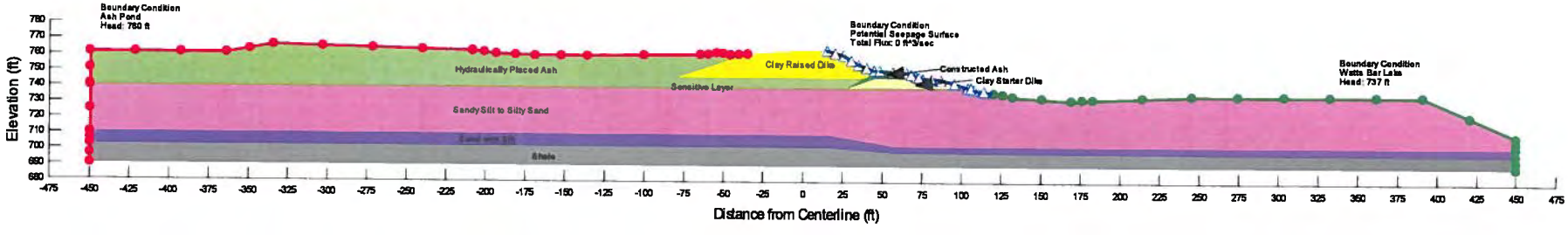
Kingston Fossil Plant
Dike C - Station 138+27
Gradient Contours



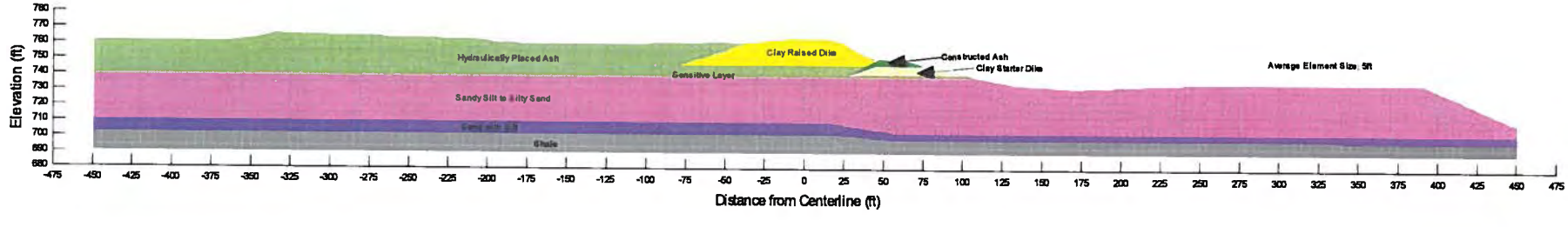
Kingston Fossil Plant
Dike C - Station 138+27
Vertical Gradient Contours



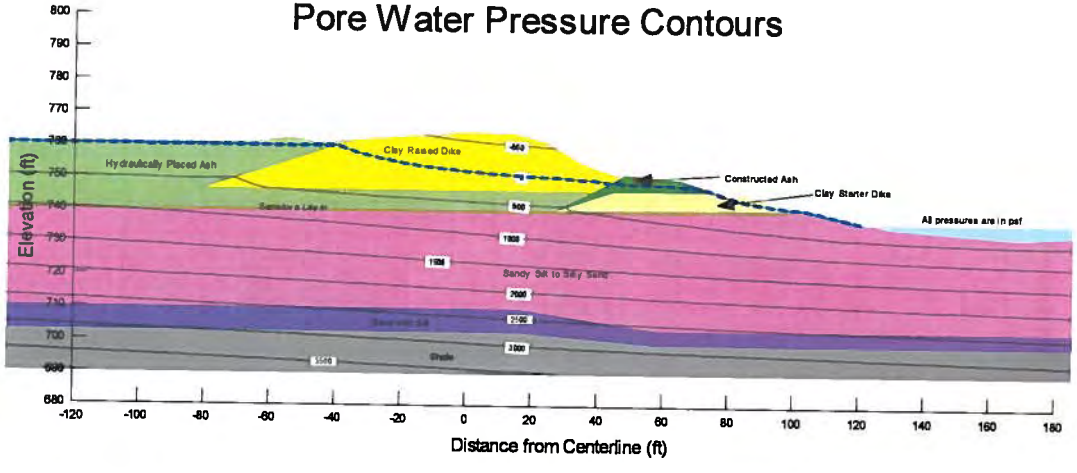
Kingston Fossil Plant
Dike C - Station 149+14
Subsurface Profile and Boundary Conditions



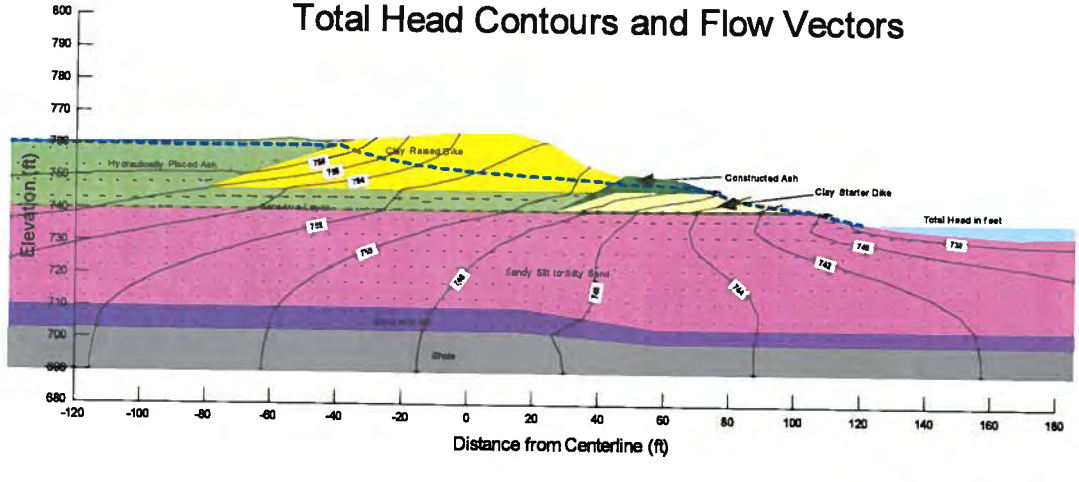
Kingston Fossil Plant
Dike C - Station 149+14
Finite Element Mesh



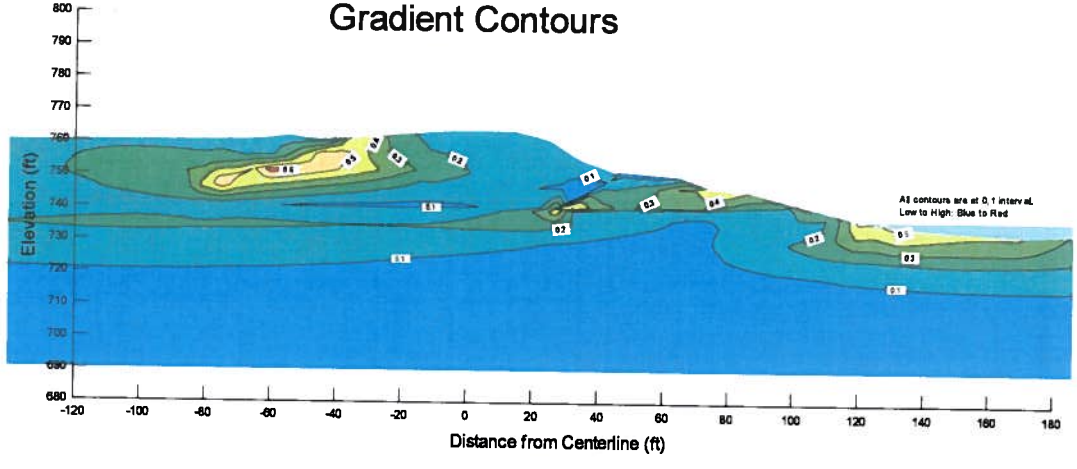
Kingston Fossil Plant
Dike C - Station 149+14
Pore Water Pressure Contours



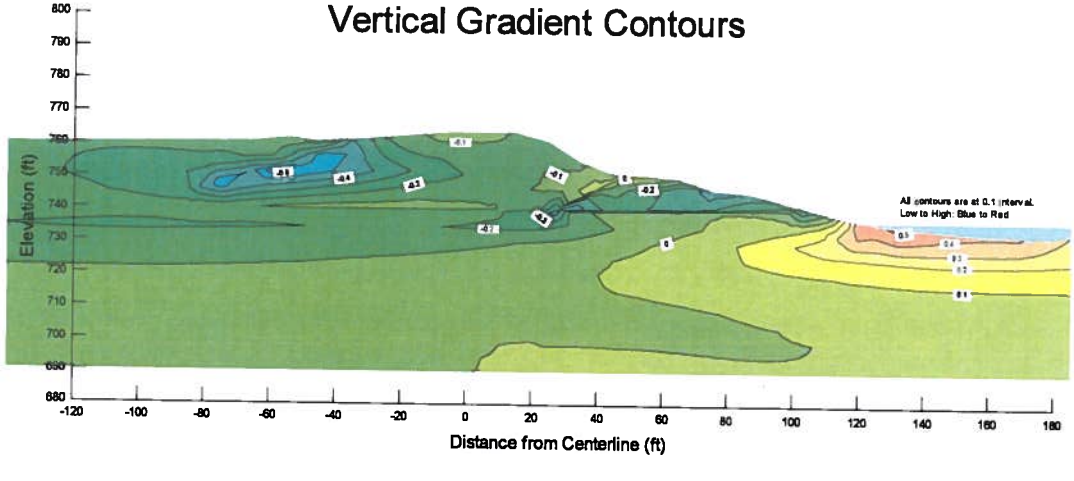
Kingston Fossil Plant
Dike C - Station 149+14
Total Head Contours and Flow Vectors



Kingston Fossil Plant
Dike C - Station 149+14
Gradient Contours



Kingston Fossil Plant
Dike C - Station 149+14
Vertical Gradient Contours



US EPA ARCHIVE DOCUMENT

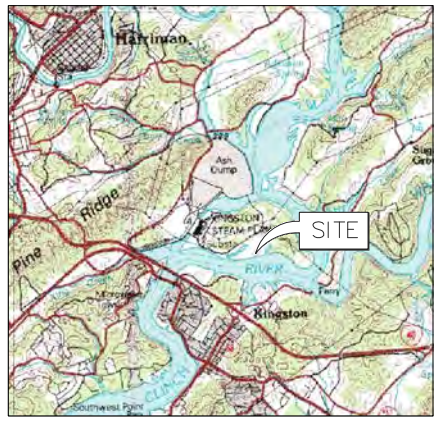
APPENDIX A

Document 2

Construction Drawings, "Coal Combustion By-Products (Gypsum) Disposal Facility, Kingston Fossil Plant – Peninsula Site" TVA Drawing Numbers 10W428-1 through 10W428-22

DRAWING INDEX

10W428–1	COVER SHEET
10W428–2	EXISTING SITE CONDITIONS
10W428–3	SITE DEVELOPMENT PLAN
10W428–4	EROSION CONTROL PLAN
10W428–5	EROSION CONTROL DETAILS SHEET 1 OF 2
10W428–6	EROSION CONTROL DETAILS SHEET 2 OF 2
10W428–7	PHASE I INITIAL GRADING PLAN AND SOIL DIKES
10W428–8	PHASE I TOP OF GEOLOGIC BUFFER
10W428–9	CROSS SECTIONS
10W428–10	EXISTING BORROW AREA
10W428–11	PROPOSED BORROW AREA GRADING PLAN
10W428–12	PERIMETER BERM CROSS SECTIONS SHEET 1 OF 2
10W428–13	PERIMETER BERM CROSS SECTIONS SHEET 2 OF 2
10W428–14	STORM DETENTION POND SECTIONS
10W428–15	PROPOSED ROAD PLAN AND PROFILE
10W428–16	PROPOSED ROAD CROSS SECTIONS SHEET 1 OF 3
10W428–17	PROPOSED ROAD CROSS SECTIONS SHEET 2 OF 3
10W428–18	PROPOSED ROAD CROSS SECTIONS SHEET 3 OF 3
10W428–19	PUMP STATION PLAN, SECTIONS, AND DETAILS
15W703	PUMP STATION ELECTRICAL
10W427–20	STORM DRAINAGE PLAN
10W427–21	DRAINAGE SYSTEM DETAILS SHEET 1 OF 2
10W427–22	DRAINAGE SYSTEM DETAILS SHEET 2 OF 2



LOCATION MAP



VICINITY MAP

COAL-COMBUSTION BY PRODUCT (GYPSUM) DISPOSAL FACILITY
KINGSTON FOSSIL PLANT - PENINSULA SITE

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV NO.
R 0	02/08	JTS	SAH	JTS	HLP	BAK					D
INITIAL ISSUE, FOR W.O.# KIF 08–500095–000 R0											

DISCIPLINE INTERFACE

SCALE: NTS EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
COVER SHEET

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

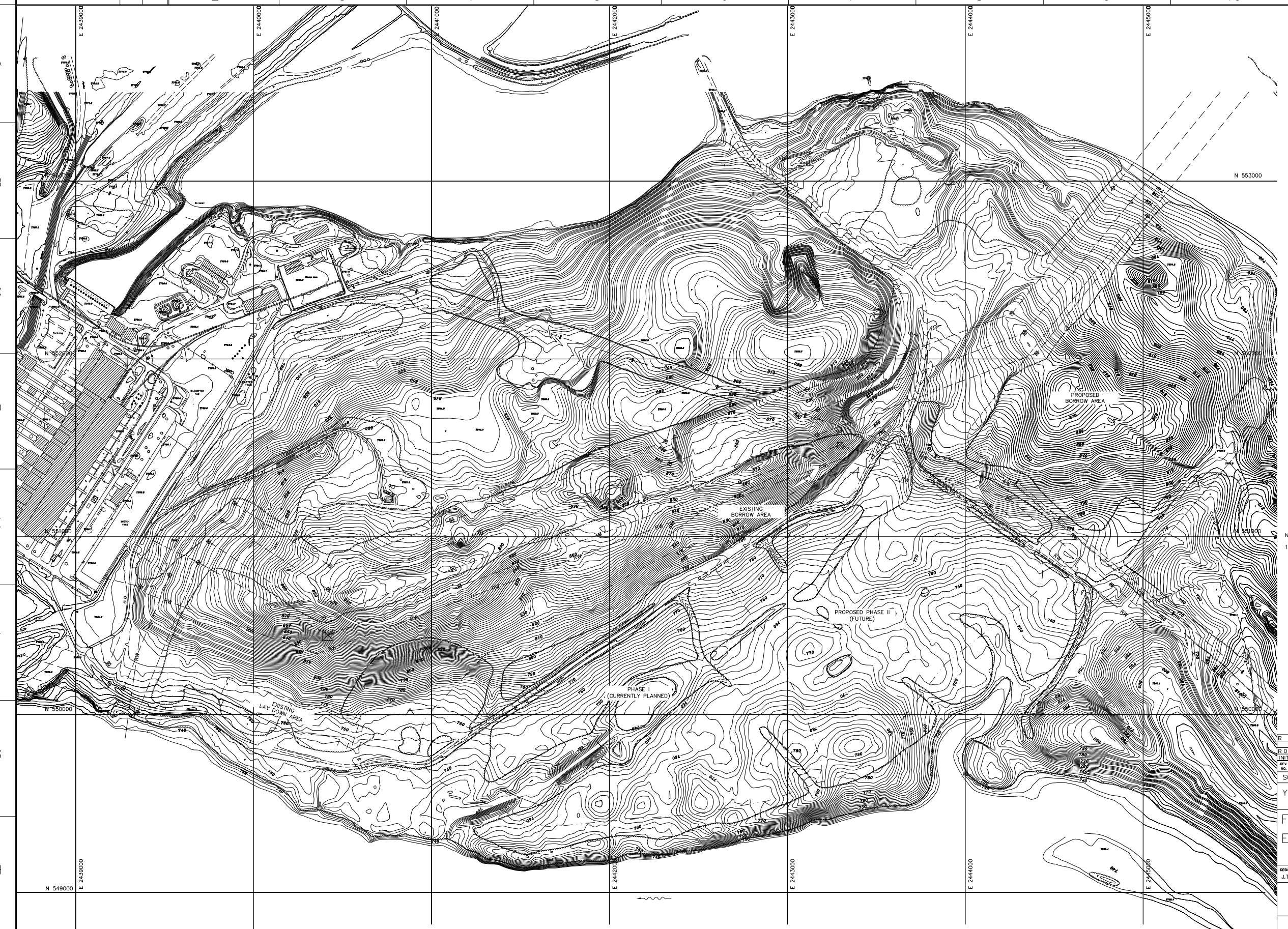
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R00	DATE	36	C	10W428–1	R 0

CONSOLIDATED TECH., INC.
TASK COMPLETED BY: REV NO.

PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY



NOTES:

1. THE TOPOGRAPHIC MAP SHOWN ON THIS DRAWING WAS PROVIDED BY TVA AND BASED ON AERIAL PHOTOGRAPHY TAKEN IN NOVEMBER 1993. GRID COORDINATES CORRESPOND TO TENNESSEE STATE PLANE COORDINATE SYSTEM, NORTH AMERICAN DATUM (NAD) 1927. ELEVATIONS ARE IN FEET ABOVE SEA LEVEL, NATIONAL GEODETIC VERTICAL DATUM (NGVD) 1929.

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ENGR	AS CONST	REV NO.
R											
R 0	02/08	JTS	SAH	JTS	HLP	BAK					0

INITIAL ISSUE. FOR W.O.# KIF 08-500095-000 R0

DISCIPLINE INTERFACE

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
EXISTING SITE CONDITIONS

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

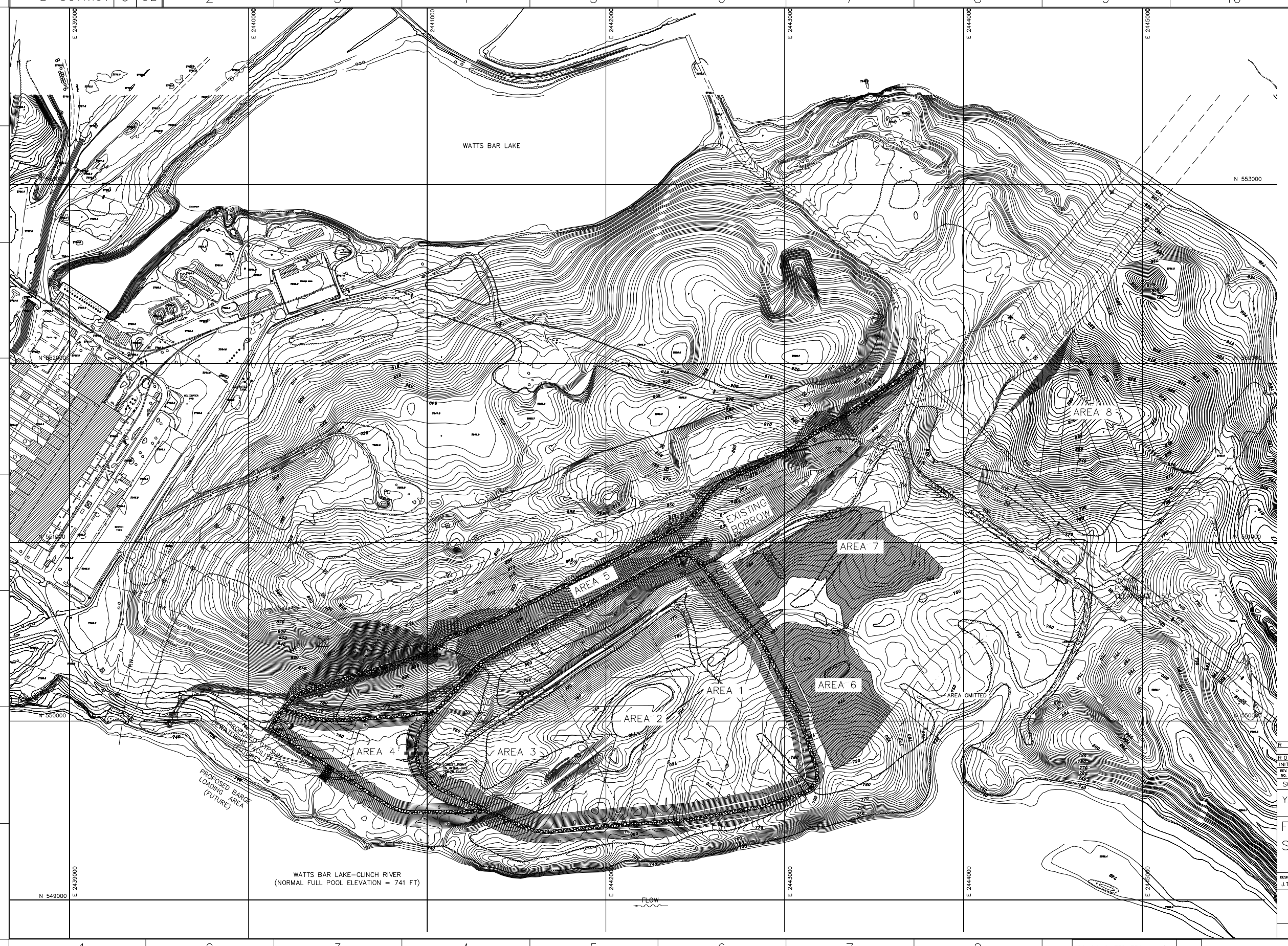
AUTOCAD R00	DATE	36	C	10W428-2	R 0

CONSOLIDATED TECH., INC.	
TASK COMPLETED BY:	REV NO.

PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

36 C 10W428-2



REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST
R 0	02/08	JTS	SAH	JTS	HLP	BAK				

INITIAL ISSUE; FOR W.O.# KIF 08-500095-000 R0

DISCIPLINE INTERFACE

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
SITE DEVELOPMENT PLAN

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

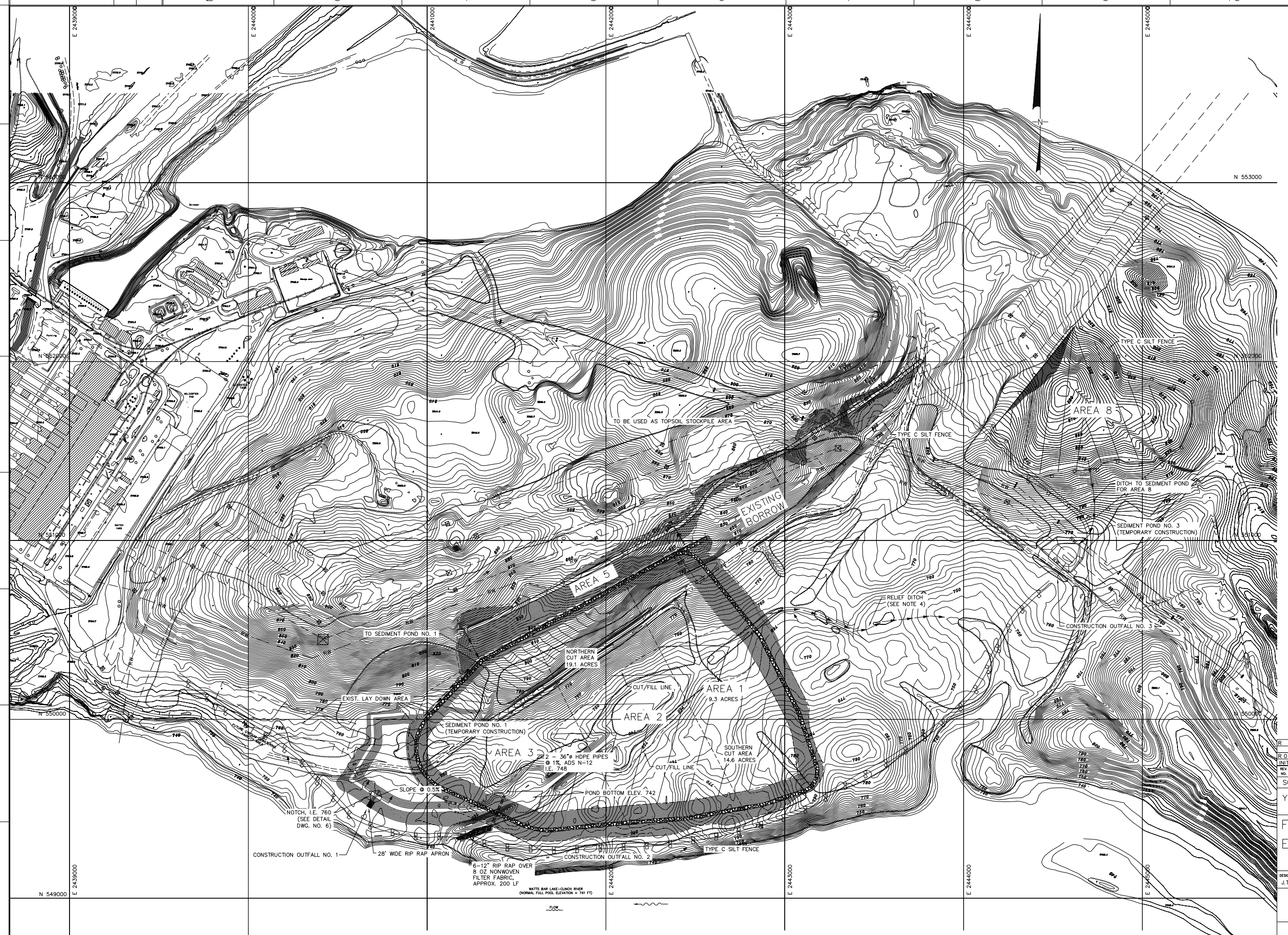
AUTOCAD R00	DATE	36	C	10W428-3	R 0

CONSOLIDATED TECH., INC.

TASK COMPLETED BY: REV NO.

PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY



QUANTITY TABLE

AREA	CUT, CY	FILL, CY	NET, CY	ACREAGE	TOPSOIL, CY
1	114,830	165,047	(50,217)	19.70	31,796
2	88,305	115,686	(27,391)	19.40	31,312
3	78,677	164,369	(85,692)	20.35	32,845
4	14,457	88,750	(74,293)	7.78	12,557
5	199,122	61,055	138,067	13.73	22,160
6	44,770	0	44,770	6.32	10,200
7	93,851	0	93,851	8.80	14,203
8	251,463	0	251,463	9.78	15,785

QUANTITY TABLE NOTES:
1. AREAS 6 AND 7 HAVE BEEN OMITTED FOR USE AS BORROW AREAS.
2. TOPSOIL QUANTITY IS BASED ON AN AVERAGE STRIPPING DEPTH OF 12-INCHES.
3. A SHRINKAGE FACTOR OF 20 PERCENT HAS BEEN ADDED TO FILL QUANTITIES.
4. THE QUANTITY OF TOPSOIL FROM STRIPPING ACTIVITIES IS APPROXIMATELY 150,000 CUBIC YARDS. THE DIMENSIONS FOR A SINGLE STOCKPILE OF THIS MATERIAL WOULD HAVE A SQUARE BASE WITH SIDE DIMENSIONS OF 450 FEET, A HEIGHT OF 25 FEET, AND A SQUARE TOP WITH SIDE DIMENSIONS OF 354 FEET ASSUMING SIDE SLOPES OF 2:1.

TEMPORARY SEDIMENT PONDS

POND NO.	TOTAL REQUIRED CAPACITY (CF)	DRAINAGE AREA (ACRES)	CAPACITY PER ACRE (CF)	TOTAL CAPACITY (CF)
1	146,255	49.9	3,118	560,904
2	243,235	78.0	3,118	846,320
3	45,777	16.0	2,924	47,448

1. TOTAL CAPACITY BASED ON RUNOFF FROM THE 2-YEAR, 24-HOUR STORM

NOTE:
REFER TO DOCUMENT ENTITLED KIF GYPSUM POND LINE ITEMS FOR CONSTRUCTION SEQUENCE.

NOTES:
1. ALL SLOPES ASSOCIATED WITH THE ACCESS ROAD CONSTRUCTION AND THE OUTER SLOPE OF THE PHASE I AND STORM WATER POND DIKES WILL RECEIVE SOD.
2. THE TOP OF THE 3-FOOT GEOLOGIC BUFFER AND INSIDE SLOPES OF THE PHASE I AREA WILL RECEIVE POSI-SHELL.
3. THE INSIDE SLOPES AND BOTTOM SURFACE OF THE STORM WATER POND WILL RECEIVE POSI-SHELL.
4. CONSTRUCT RELIEF DITCH PRIOR TO CONSTRUCTION OF SOIL DIKE CONSTRUCTION IN THIS AREA.

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV ID
R											
R 0	02/08	JTS	SAH	JTS	HLP	BAK					D

INITIAL ISSUE: FOR W.O.# KIF 08-500095-000 R0

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
EROSION CONTROL PLAN

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

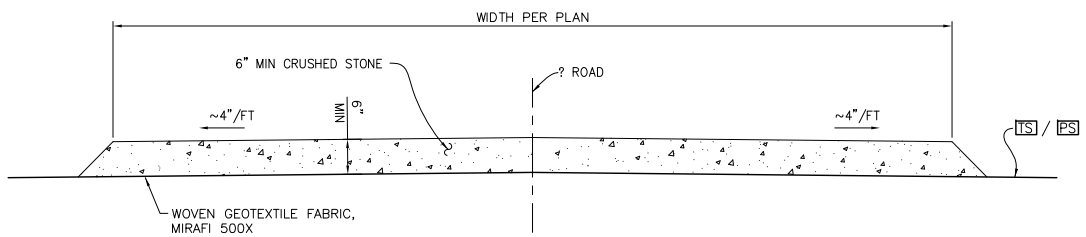
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R00	DATE 02/08	36	C	10W428-4	R 0

CONSOLIDATED TECH., INC.
TASK COMPLETED BY: | REV NO.

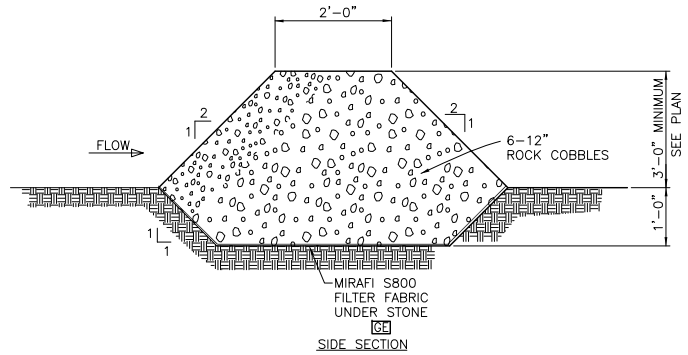
PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

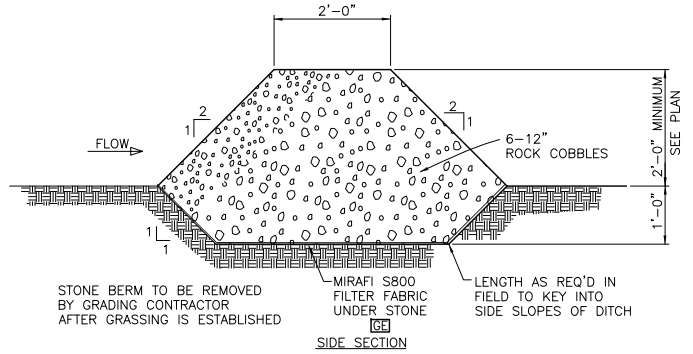


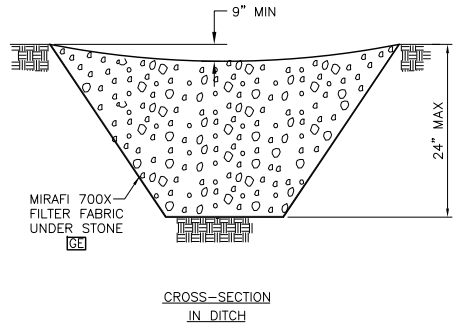
ROADWAY
TYPICAL SECTION
NOT TO SCALE

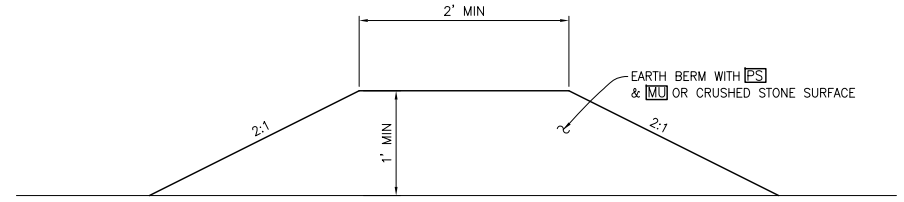
GRAVEL THICKNESS:
1. 6-INCHES PERIMETER ROAD ON STORM WATER AND PHASE 1 DIKES.
2. 12-INCHES ON ACCESS ROAD.
3. GRAVEL SHALL CONFORM TO TYPE "A" GRADING "D" PER T.D.O.T. SPECIFICATIONS SECTION 303.



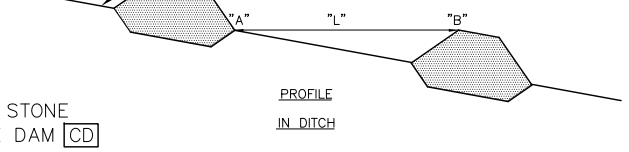
STORM WATER PUMP STATION
INLET FILTER RING DETAIL
NOT TO SCALE



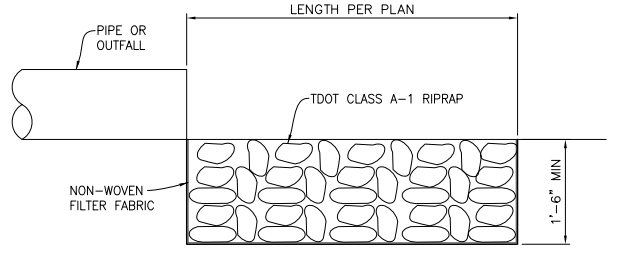




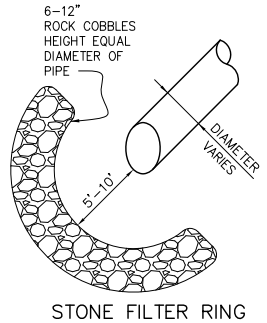
DIVERSION BERM DI
TYPICAL DETAIL 9
NOT TO SCALE



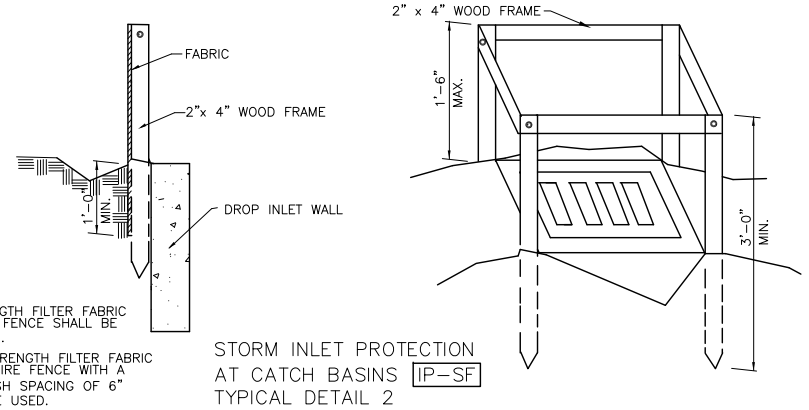
DITCH STONE
CHECK DAM CD
TYPICAL DETAIL 4
NOT TO SCALE



OUTLET PROTECTION OP
TYPICAL DETAIL 8
NOT TO SCALE



STONE FILTER RING
INLET
PROTECTION FR
TYPICAL DETAIL 7
NTS



NOTES
EXTRA STRENGTH FILTER FABRIC
1. NO WIRE FENCE SHALL BE REQUIRED.

STANDARD STRENGTH FILTER FABRIC
1. 14 GA. WIRE FENCE WITH A MAX. MESH SPACING OF 6" SHALL BE USED.
2. EXTEND WIRE FENCE TO BOTTOM OF TRENCH.

STORM INLET PROTECTION
AT CATCH BASINS IP-SF
TYPICAL DETAIL 2
NOT TO SCALE

TS DISTURBED AREA STABILIZATION WITH TEMPORARY VEGETATION
PS DISTURBED AREA STABILIZATION WITH PERMANENT VEGETATION
MU DISTURBED AREA STABILIZATION WITH MULCH
GE GEOTEXTILE

NOTES:
1. CONSTRUCT EROSION PROTECTION SILT FENCE DOWNSTREAM OF ALL DISTURBED AREA PERIMETER.
2. STAKE STRAW BALES AND SILT FENCE AS DESCRIBED IN SWPPP
3. EROSION CONTROL MEASURES SHALL BE INSTALLED AND MAINTAINED AND SHALL INCLUDE, BUT NOT LIMITED TO, SILT FENCES, AND STONE CHECK DAMS. ALL EROSION CONTROL MEASURES SHALL BE ESTABLISHED BEFORE BEGINNING CLEARING, GRUBBING AND CONSTRUCTION OPERATIONS.
4. SEE THE STORMWATER POLLUTION PREVENTION PLAN (SWPPP) FOR ADDITIONAL REQUIREMENTS FOR STORM WATER DISCHARGE AND SEDIMENTATION CONTROL.
5. THE CONTRACTOR SHALL FIELD VERIFY ALL EXISTING UNDERGROUND UTILITIES PRIOR TO BEGINNING THE HAUL ROAD AND FACILITY CONSTRUCTION.
6. STRIP SURFACE OF ALL VEGETATION, DEBRIS, STUMPS, BRUSH AND ORGANIC PERISHABLE MATERIAL PRIOR TO GRADING. DISPOSAL OF MATERIAL SHALL BE IN ACCORDANCE WITH SWPPP.
7. FOR SOIL EROSION PLANS SEE DRAWING NO.10W428-4.
8. REMOVE EXISTING FENCE AS REQUIRED FOR GRADING OPERATIONS.
9. IF WORK IS SUSPENDED AND WILL NOT RESUME WITHIN 15 DAYS, TEMPORARY OR PERMANENT STABILIZATION OF DISTURBED AREAS SHALL BE COMPLETED WITHIN 15 DAYS OF WORK CEASING.
10. TVA CONSTRUCTION MANAGER HAS THE AUTHORITY TO MODIFY DESIGN DOCUMENTS AS NEEDED TO INSURE ADEQUATE MEASURES ARE TAKEN TO CONTROL RUNOFF DURING CONSTRUCTION. ANY MODIFICATIONS SHALL BE INCORPORATED INTO THE SWPPP WITHIN 14 DAYS.
11. PLACE SILT FENCING AROUND STOCKPILE MATERIALS, UNLESS OTHERWISE NOTED ON DRAWINGS.
12. SEDIMENT SHOULD BE REMOVED FROM SEDIMENT TRAPS, SILT FENCES, SEDIMENT PONDS, AND OTHER SEDIMENT CONTROLS AS NECESSARY, AND MUST BE REMOVED WHEN DESIGN CAPACITY HAS BEEN REDUCED BY 50%.
13. THE FIELD MAY UTILIZE DETAIL 6 AS AN ALTERNATIVE FOR STORM INLET PROTECTION, DETAIL 2, IF DESIRED.
14. ALL EROSION CONTROL MATERIALS & DEVICES SHALL COMPLY WITH THE "TENNESSEE EROSION AND SEDIMENT CONTROL HANDBOOK".
15. SILT FENCE AND STRAW BALES MAY BE REMOVED ONCE AREAS ARE STABILIZED.

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	DISCIPLINE INTERFACE
R											
R 0	02/08	JTS	SAH	JTS	HLP	BAK					D

INITIAL ISSUE; FOR W.O.# KIF 08-500095-000 R0

SCALE: NTS — EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
EROSION CONTROL DETAILS
SHEET 1 OF 2

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

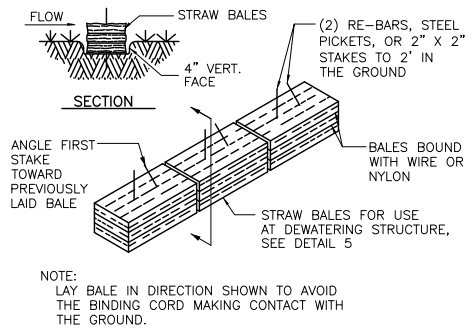
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R00	DATE 02/08	36	C	10W428-5	R 0

CONSOLIDATED TECH., INC.
TASK COMPLETED BY: | REV NO.

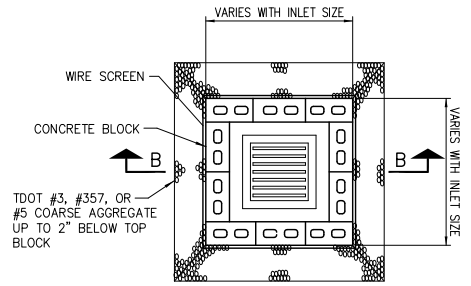
PLOT FACTOR: 1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

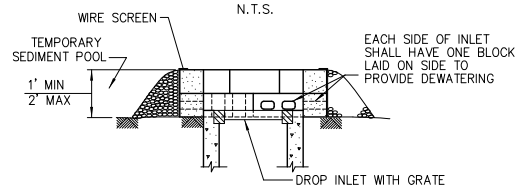


NOTE:
LAY BALE IN DIRECTION SHOWN TO AVOID THE BINDING CORD MAKING CONTACT WITH THE GROUND.

STRAW BALES ANCHORING
TYPICAL DETAIL 3
NOT TO SCALE

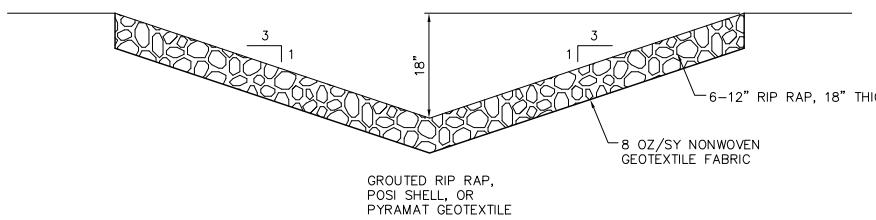


PLAN
N.T.S.

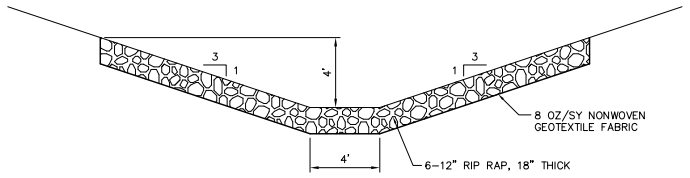


SECTION B–B
NTS

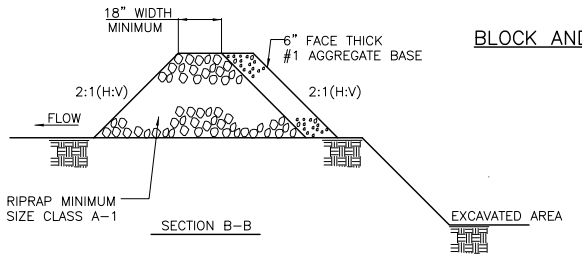
BLOCK AND GRAVEL INLET PROTECTION IP–BG
TYPICAL DETAIL 6
NOT TO SCALE



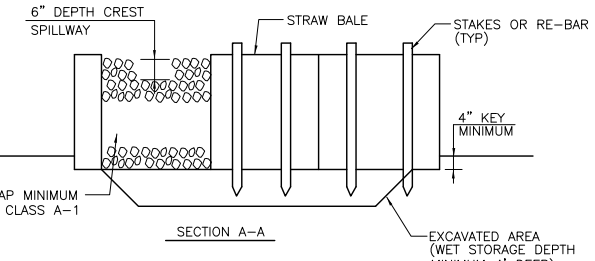
PROPOSED BORROW AREA DITCH TO SEDIMENT POND NO. 3
NOT TO SCALE



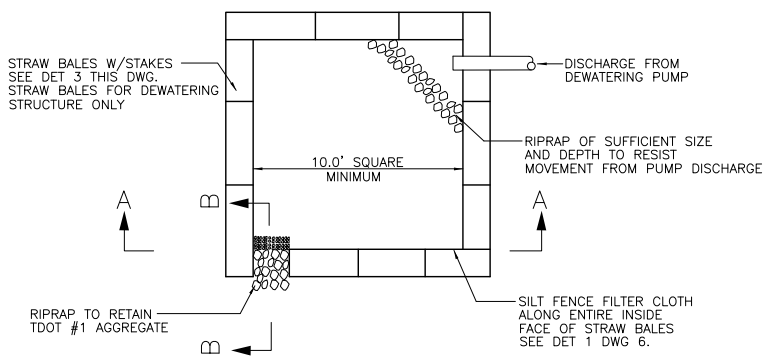
SEDIMENT POND NO. 1 NOTCH SECTION
SCALE: ¼"=1'-0"



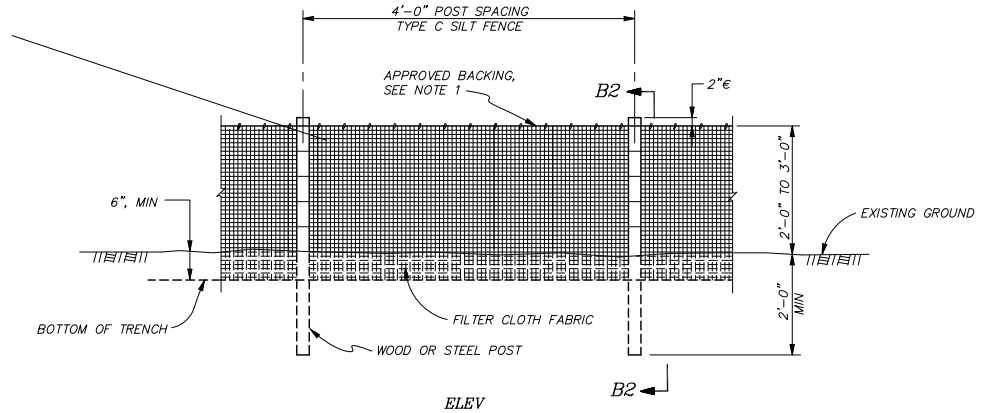
SECTION B–B



SECTION A–A



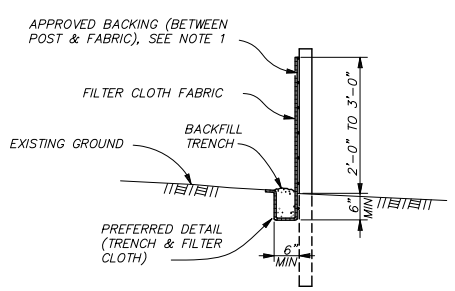
DEWATERING STRUCTURE
STRAW BALE/SILT FENCE DW
TYPICAL DETAIL 5
(AS REQUIRED
AT PUMP LOCATIONS)
NTS



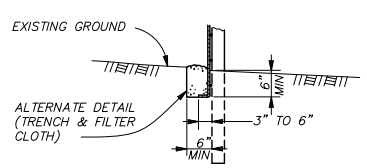
ELEV

SILT FENCE NOTES

1. FILTER CLOTH SHALL HAVE APPROVED BACKING OR A BUILT–IN REINFORCED STRUCTURE, AS RECOMMENDED BY THE MANUFACTURER TO SUPPORT THE FILTER CLOTH.
2. CLASS A FILTER FABRIC SHALL MEET THE REQUIREMENTS OF SECTION 571 OF THE T–1 SPECIFICATIONS.
3. A PREASSEMBLED SILT FENCE MEETING THE REQUIREMENTS OF THIS DRAWING IS ACCEPTABLE IN LIEU OF A FIELD CONSTRUCTED SILT FENCE.



B2–B2
PREFERRED TRENCH



B2–B2
ALTERNATE TRENCH

SILT FENCE TYPE "C"
NOT TO SCALE

R																
R 0	02/08	JTS	SAH	JTS	HLP	BAK										D
INITIAL ISSUE; FOR W.O.# KIF 08–500095–000 R0																
REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV CD

DISCIPLINE INTERFACE

SCALE: NTS — EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
EROSION CONTROL DETAILS
SHEET 2 OF 2

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R00	DATE 02/08	36	C	10W428–6	R 0

CONSOLIDATED TECH., INC.	
TASK COMPLETED BY:	REV NO.

PLOT FACTOR: 1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

36 | C | 10W428-7



REV	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV
R 0	02/08	JTS	SAH	JTS	HLP	BAK					0

INITIAL ISSUE. FOR W.O.# KIF 08-500095-000 R0

DISCIPLINE INTERFACE

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
PHASE I INITIAL GRADING PLAN AND SOIL DIKES

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

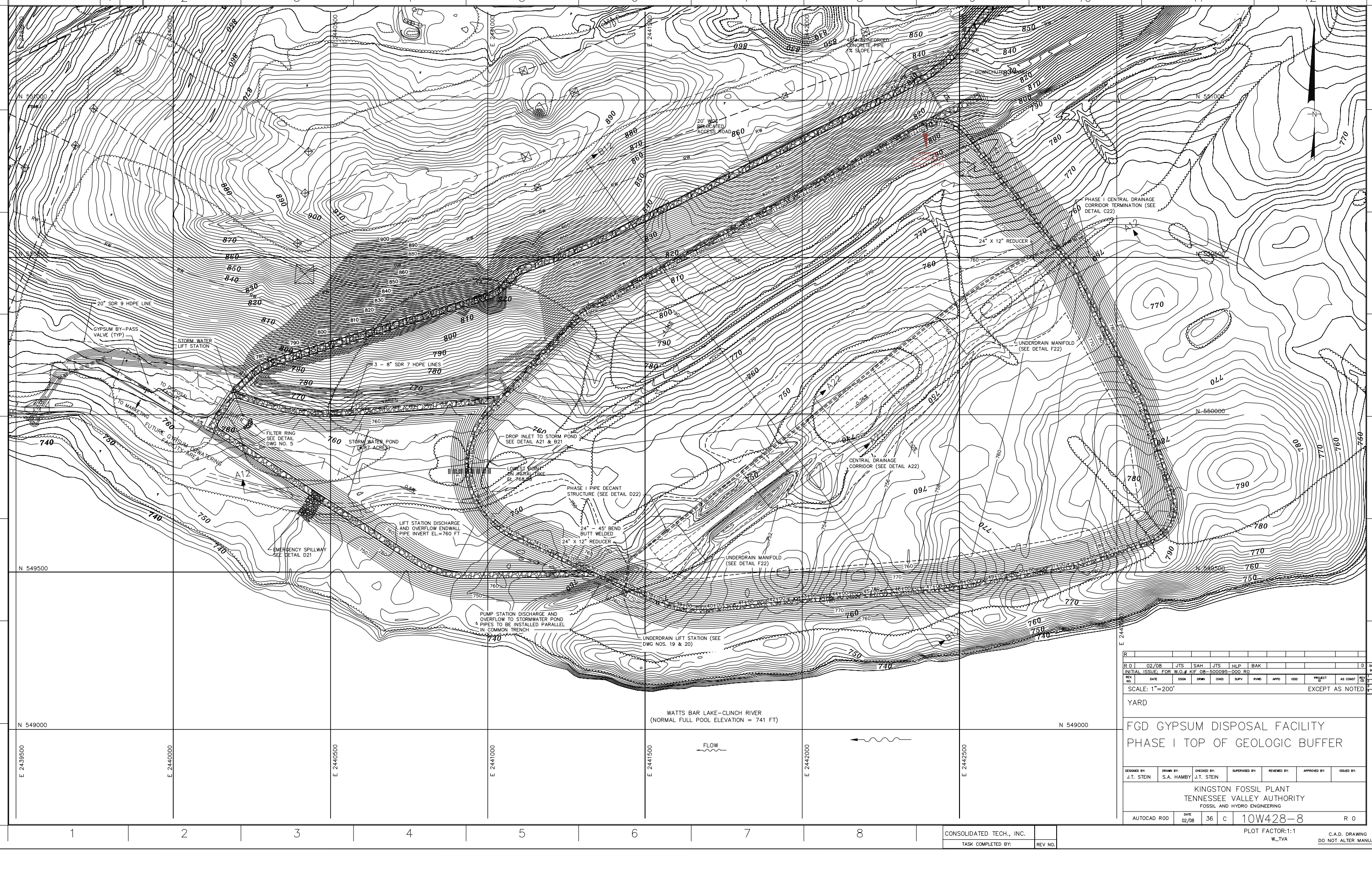
AUTOCAD R00	DATE 02/08	36	C	10W428-7	R 0

CONSOLIDATED TECH., INC.
TASK COMPLETED BY: REV NO.

PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

36 | C | 10W428-8



R															
R 0	02/08	JTS	SAH	JTS	HLP	BAK									D
INITIAL ISSUE; FOR W.O.# KIF 08-500995-000 R0															DISCIPLINE INTERFACE
REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV. ID				

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
PHASE I TOP OF GEOLOGIC BUFFER

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

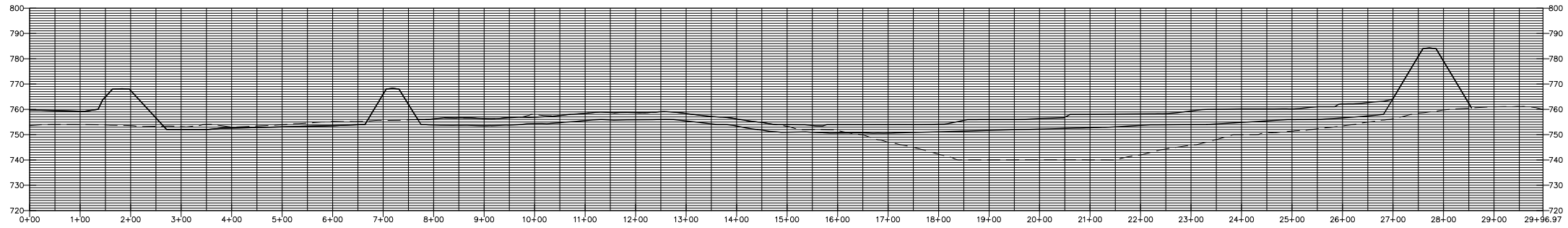
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R00	DATE 02/08	36	C	10W428-8	R 0

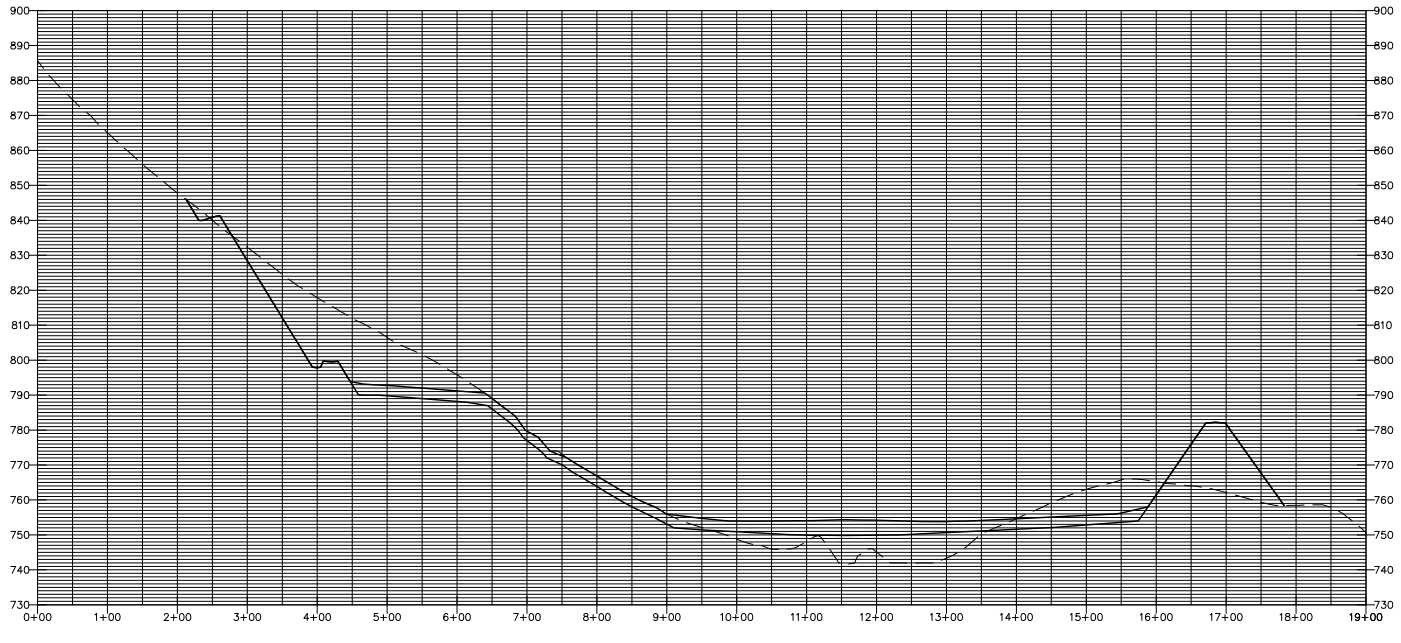
CONSOLIDATED TECH., INC.
TASK COMPLETED BY: REV NO.

PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY



SECTION A12
HORIZ: 1"=100'
VERT: 1"=20'



SECTION B12
HORIZ: 1"=100'
VERT: 1"=20'

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	DISCIPLINE INTERFACE
R											
0	02/08	JTS	SAH	JTS	HLP	BAK					

INITIAL ISSUE; FOR W.O.# KIF 08-500995-000 R0

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
CROSS SECTIONS

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

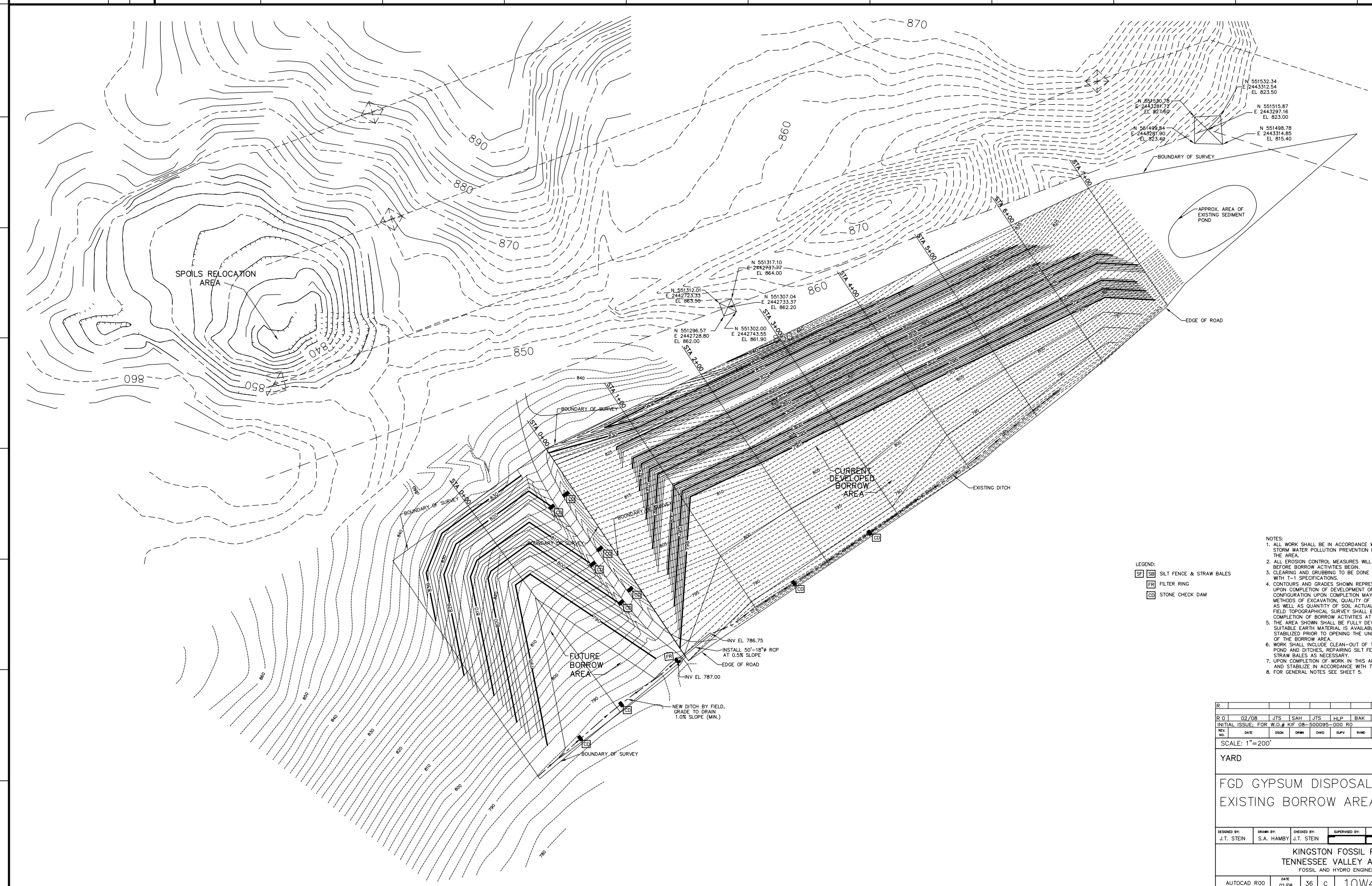
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R00	DATE 02/08	36	C	10W428-9	R 0

CONSOLIDATED TECH., INC.
TASK COMPLETED BY: REV NO.

PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY



LEGEND:

SF SB SILT FENCE & STRAW BALES
FR FILTER RING
CD STONE CHECK DAM

NOTES:

1. ALL WORK SHALL BE IN ACCORDANCE WITH THE CONSTRUCTION STORM WATER POLLUTION PREVENTION PLAN DEVELOPED FOR THE AREA.
2. ALL EROSION CONTROL MEASURES WILL BE IN PLACE BEFORE BORROW ACTIVITIES BEGIN.
3. CLEARING AND GRUBBING TO BE DONE IN ACCORDANCE WITH T-1 SPECIFICATIONS.
4. CONTOURS AND GRADES SHOWN REPRESENT EXPECTED CONFIGURATION UPON COMPLETION OF DEVELOPMENT OF THE AREA. ACTUAL CONFIGURATION UPON COMPLETION MAY VARY DEPENDING UPON METHODS OF EXCAVATION, QUALITY OF SOIL IN CERTAIN LOCATIONS, AS WELL AS QUANTITY OF SOIL ACTUALLY REQUIRED. A FINAL FIELD TOPOGRAPHICAL SURVEY SHALL BE COMPLETED UPON COMPLETION OF BORROW ACTIVITIES AT THE SITES.
5. THE AREA SHOWN SHALL BE FULLY DEVELOPED TO THE EXTENT SUITABLE EARTH MATERIAL IS AVAILABLE AND SHALL BE STABILIZED PRIOR TO OPENING THE UNDEVELOPED PORTION OF THE BORROW AREA.
6. WORK SHALL INCLUDE CLEAN-OUT OF THE EXISTING SEDIMENT POND AND DITCHES, REPAIRING SILT FENCE AND REPLACING STRAW BALES AS NECESSARY.
7. UPON COMPLETION OF WORK IN THIS AREA, SEED, MULCH AND STABILIZE IN ACCORDANCE WITH T-1 SPECIFICATIONS.
8. FOR GENERAL NOTES SEE SHEET 5.

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST
R 0	02/08	JTS	SAH	JTS	HLP	BAK				

INITIAL ISSUE; FOR W.O.# KIF 08-500095-000 R0

DISCIPLINE INTERFACE

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
EXISTING BORROW AREA

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

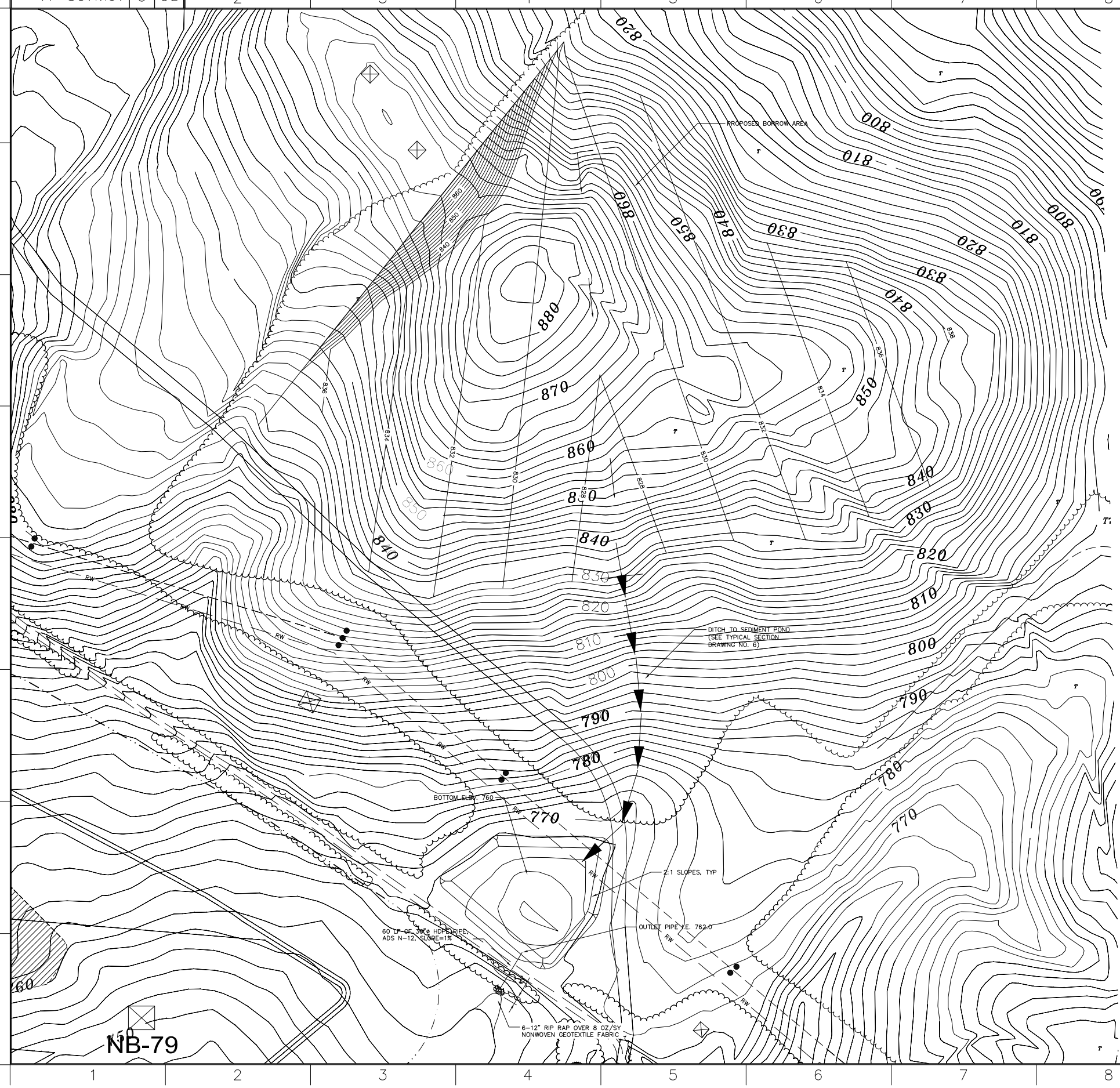
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R00	DATE 02/08	36	C	10W428-10	R 0

CONSOLIDATED TECH., INC.
TASK COMPLETED BY: REV NO.

PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY



NOTE:
FINAL CONTOURS MAY VARY BASED ON THE ACTUAL AMOUNT OF BORROW MATERIAL REQUIRED FOR THE PROJECT.

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV TO
R 0	02/08	JTS	SAH	JTS	HLP	BAK					D

INITIAL ISSUE; FOR W.O.# KIF 08-500095-000 R0

DISCIPLINE INTERFACE

SCALE: 1"=60' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
PROPOSED BORROW AREA
GRADING PLAN

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

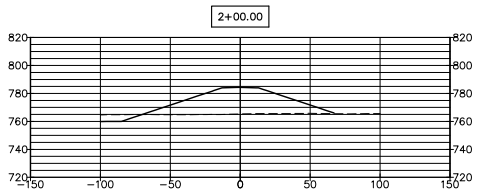
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

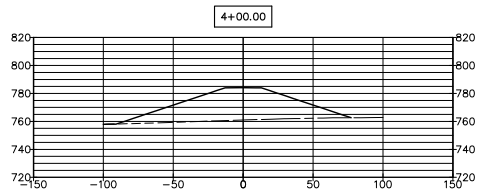
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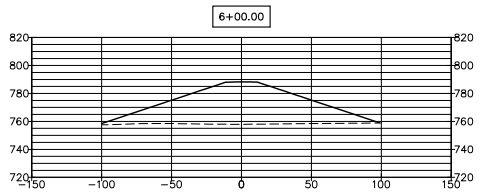
CONSOLIDATED TECH., INC.
TASK COMPLETED BY: REV NO.

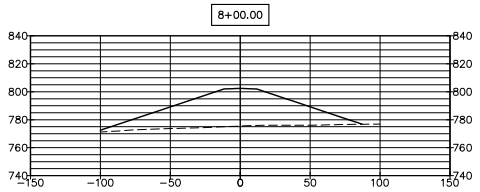
PLOT FACTOR:1:1
W_TVA

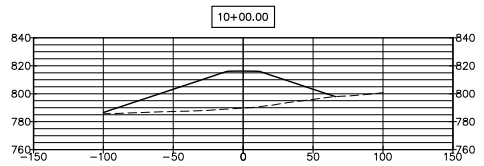
C.A.D. DRAWING
DO NOT ALTER MANUALLY

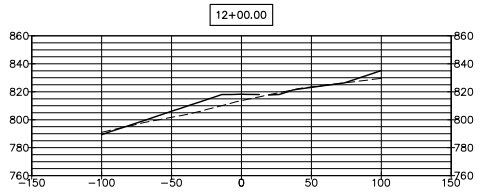


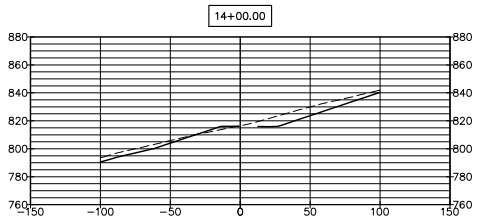


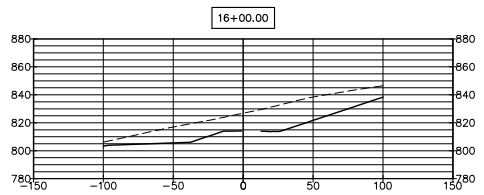


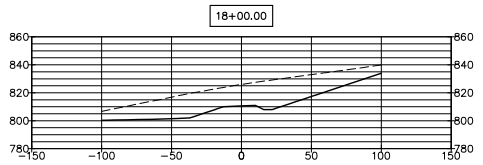


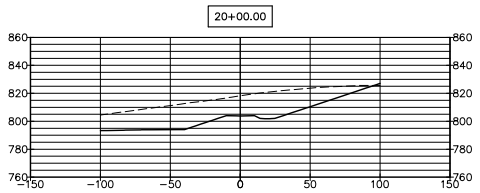


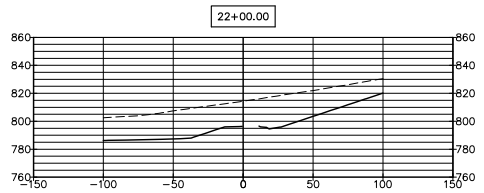


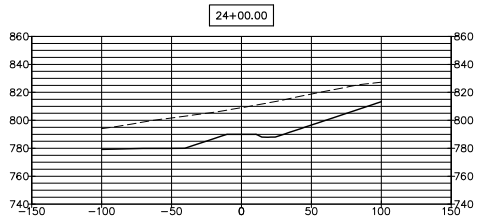


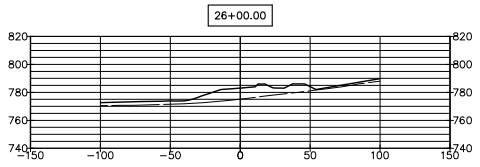


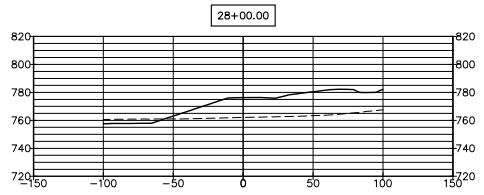


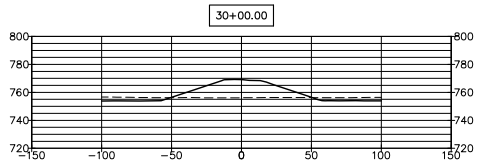












REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	DISCIPLINE INTERFACE
R											
R 0	02/08	JTS	SAH	JTS	HLP	BAK					D

INITIAL ISSUE; FOR W.O.# KIF 08-500995-000 R0

SCALE: 1"=50' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
PERIMETER BERM CROSS SECTIONS
SHEET 1 OF 2

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

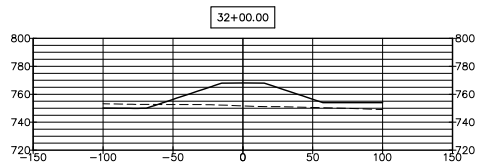
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

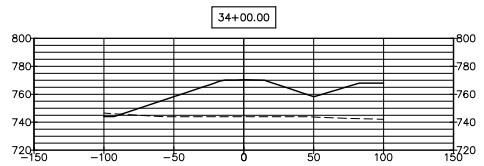
AUTOCAD R00	DATE 02/08	36	C	10W428-12	R 0

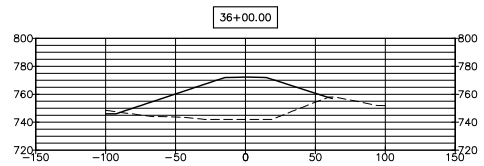
CONSOLIDATED TECH., INC.
TASK COMPLETED BY: REV NO.

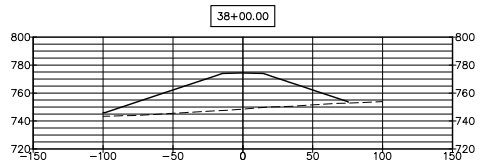
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W_TVA

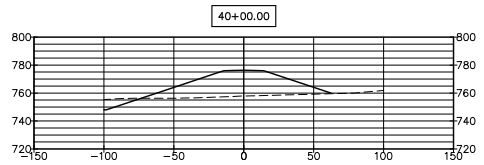
C.A.D. DRAWING
DO NOT ALTER MANUALLY

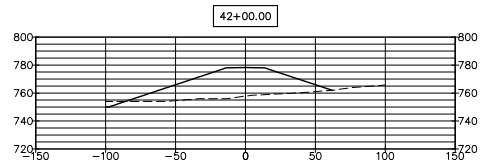


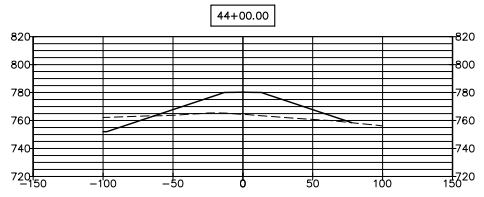


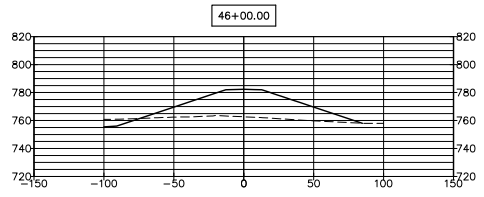


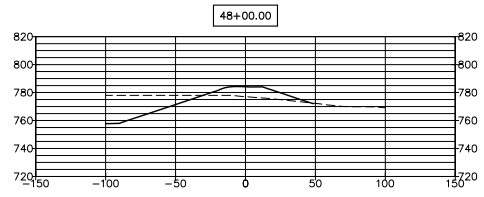


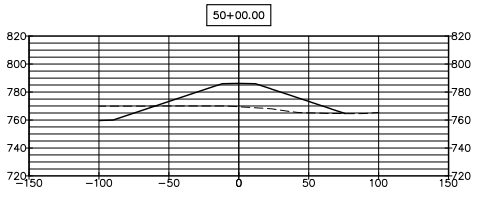


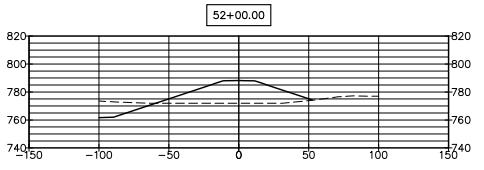


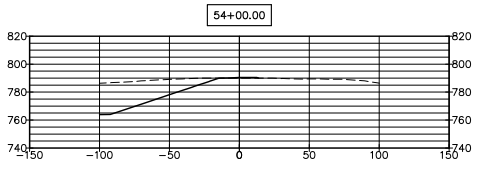


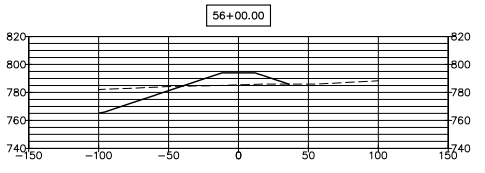


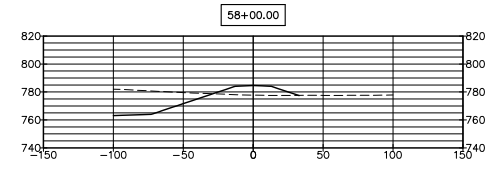












REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ED	AS CONST	DISCIPLINE INTERFACE
R 0	02/08	JTS	SAH	JTS	HLP	BAK					

INITIAL ISSUE; FOR W.O.# KIF 08-500995-000 R0

SCALE: 1"=50' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
PERIMETER BERM CROSS SECTIONS
SHEET 2 OF 2

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

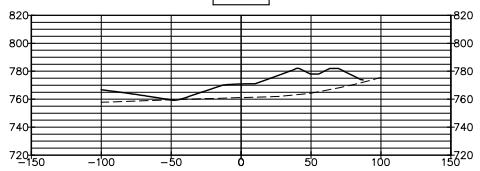
AUTOCAD R00	DATE 02/08	36	C	10W428-13	R 0

CONSOLIDATED TECH., INC.
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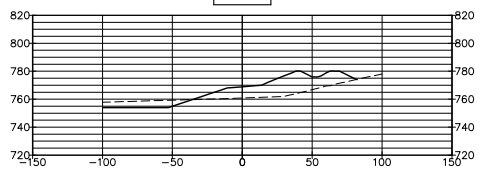
PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

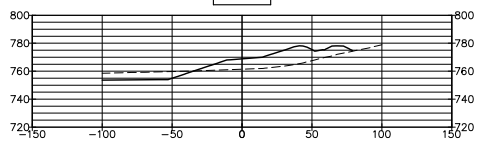
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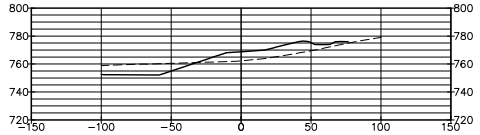
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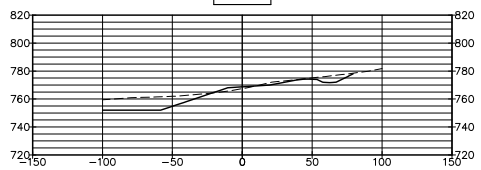
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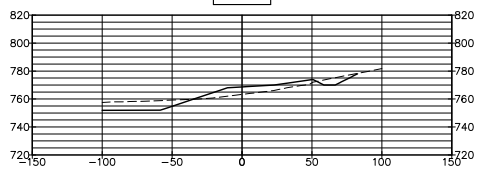
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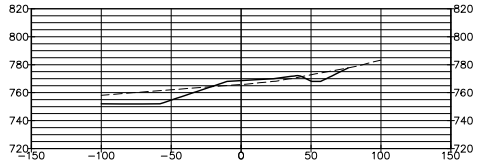
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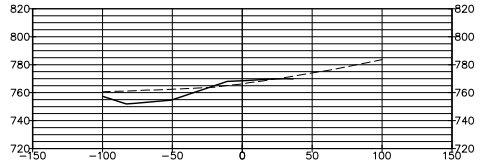
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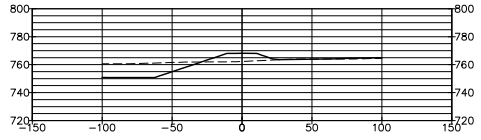
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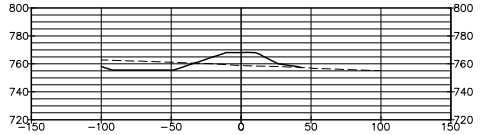
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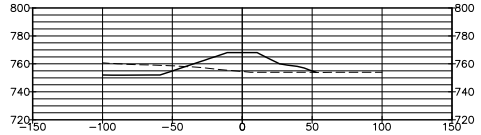
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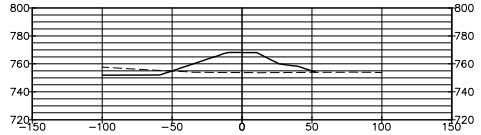
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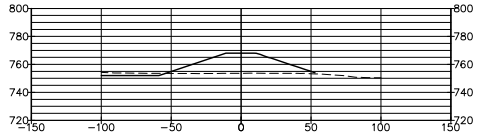
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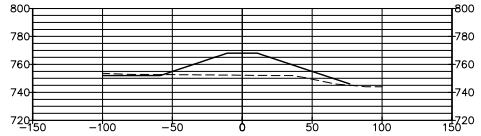
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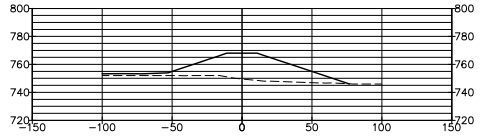
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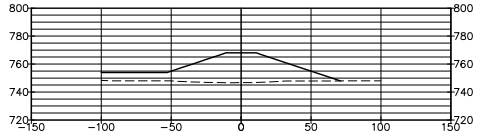
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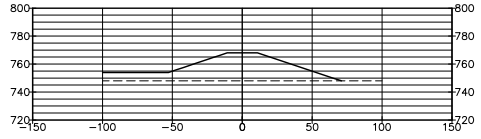
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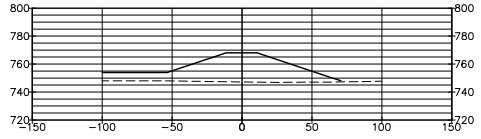
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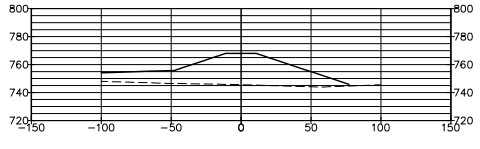
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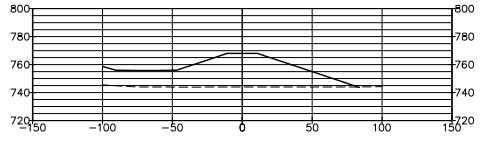
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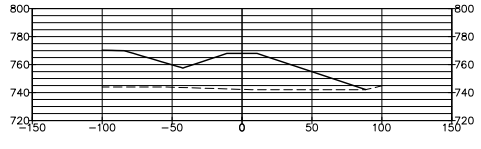
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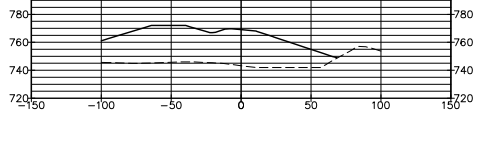
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22+00.00



R

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	DISCIPLINE INTERFACE
R 0	02/08	JTS	SAH	JTS	HLP	BAK					D

INITIAL ISSUE; FOR W.O.# KIF 08-500095-000 R0

SCALE: 1"=50' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
STORM DETENTION POND SECTIONS

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

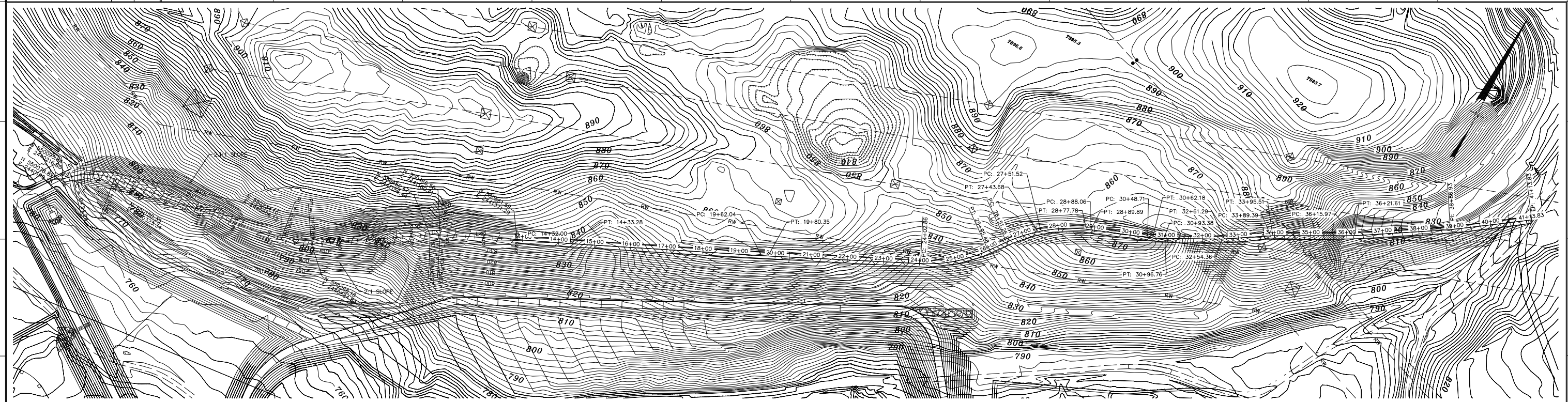
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R00	DATE 02/08	36	C	10W428-14	R 0

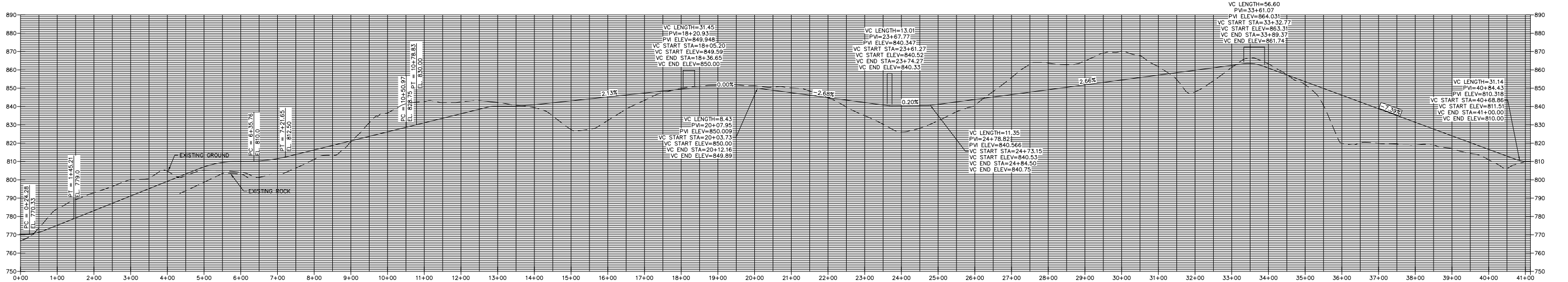
CONSOLIDATED TECH., INC.
TASK COMPLETED BY: REV NO.

PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY



ROAD PLAN
SCALE: 1"=100'



ROAD PROFILE
HORIZ: 1"=100'
VERT: 1"=20'

LINE TABLE		
LINE	LENGTH	BEARING
L1	24.28	N45°30'50"E
L2	490.55	N80°09'32"E
L3	329.32	N55°33'14"E
L4	213.23	N63°32'07"E

CURVE TABLE						
CURVE	LENGTH	RADIUS	DELTA ANGLE	TANGENT	CHORD DIRECTION	CHORD
C1	120.93	200.00	34°38'42"	62.38	S62°50'11"W	119.10
C2	85.89	200.00	24°36'18"	43.62	N67°51'23"E	85.23
C3	27.86	200.00	7°58'53"	13.95	S59°32'40"W	27.84

R											
R 0	02/08	JTS	SAH	JTS	HLP	BAK					D
INITIAL ISSUE; FOR W.O.# KIF 08-500095-000 RO											DISCIPLINE INTERFACE
REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
PROPOSED ROAD PLAN AND PROFILE

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

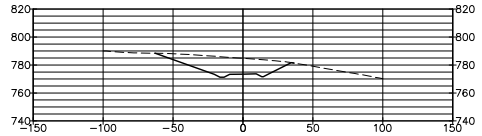
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CONSOLIDATED TECH., INC.
TASK COMPLETED BY: | REV NO.

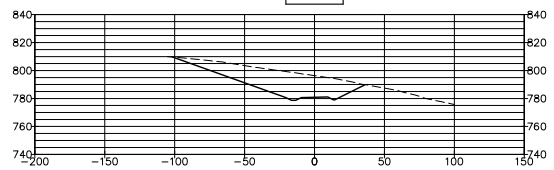
PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

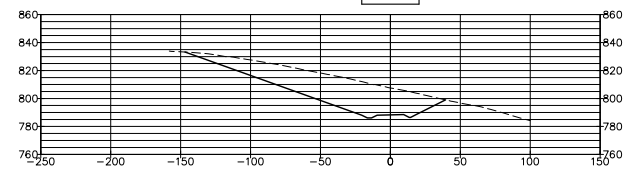
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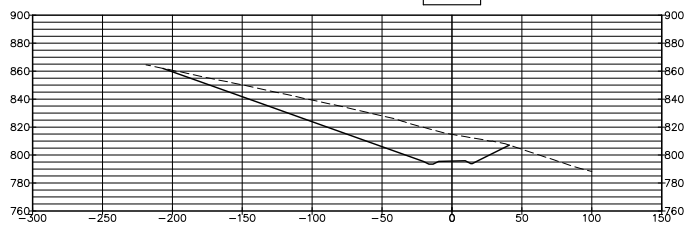
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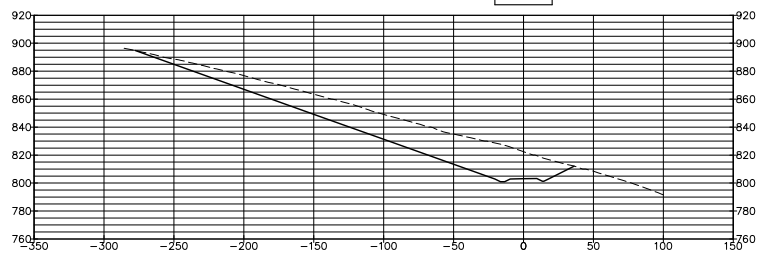
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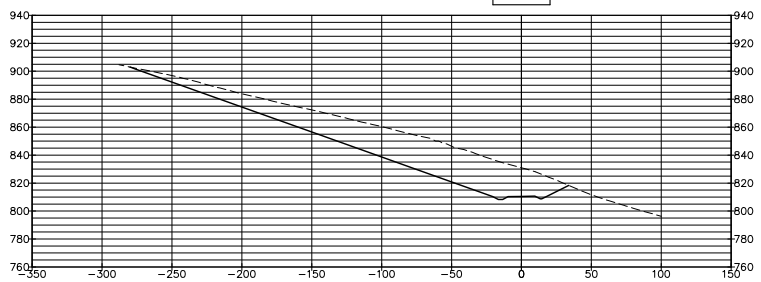
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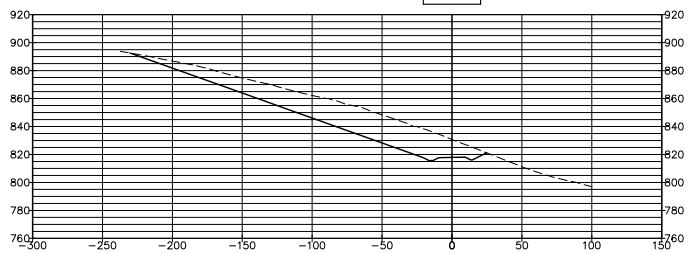
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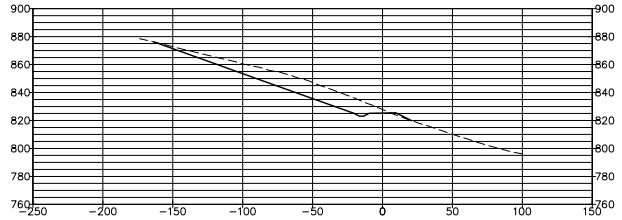
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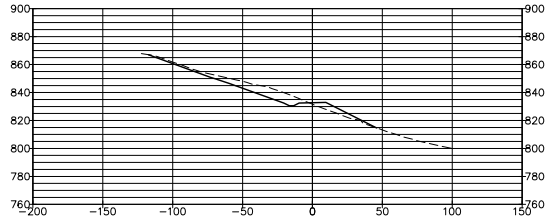
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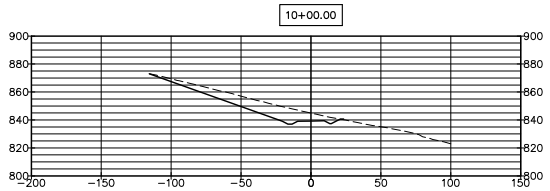
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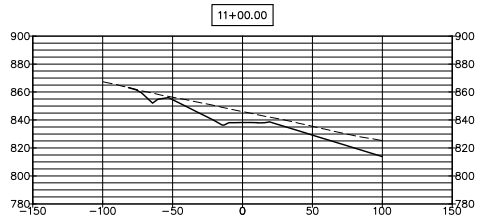
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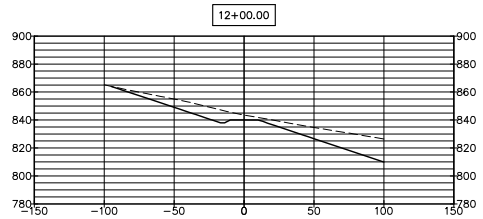
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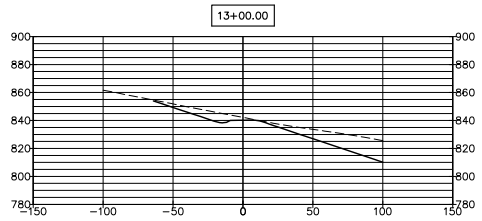
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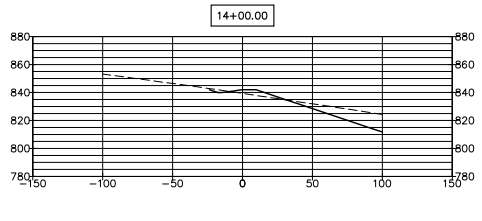
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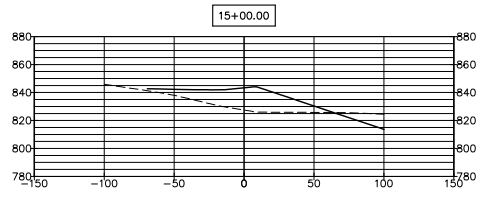
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14+00.00



15+00.00



R

R 0	02/08	JTS	SAH	JTS	HLP	BAK								D

INITIAL ISSUE. FOR W.O.# KIF 08-500095-000 R0

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV. ID

DISCIPLINE INTERFACE

SCALE: 1"=50' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
PROPOSED ROAD CROSS SECTIONS
SHEET 1 OF 3

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

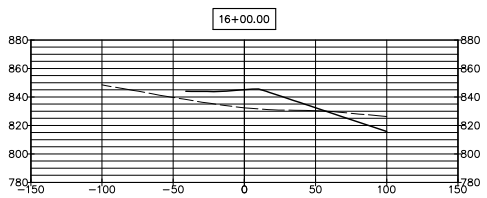
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

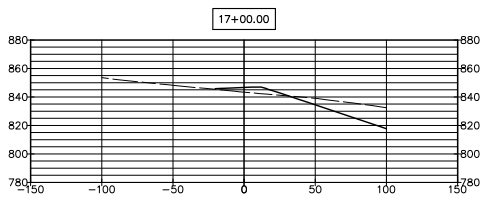
AUTOCAD R00	DATE 02/08	36	C	10W428-16	R 0

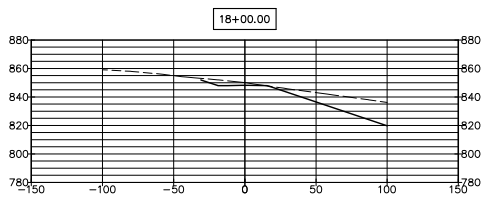
CONSOLIDATED TECH., INC.	
TASK COMPLETED BY:	REV NO.

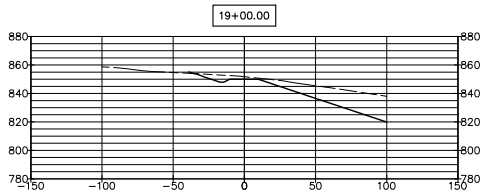
PLOT FACTOR:1:1
W_TVA

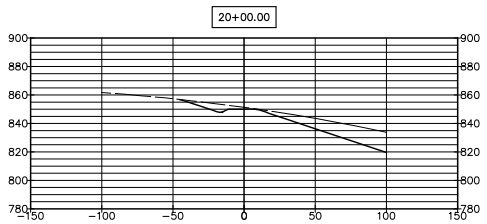
C.A.D. DRAWING
DO NOT ALTER MANUALLY

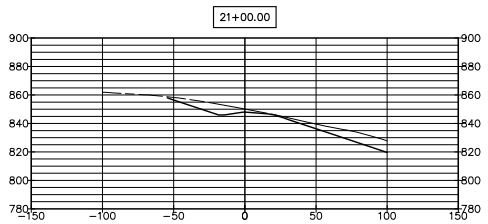


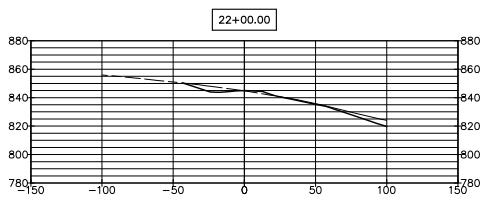


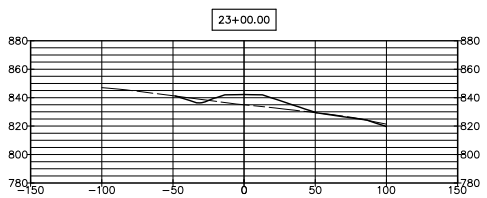


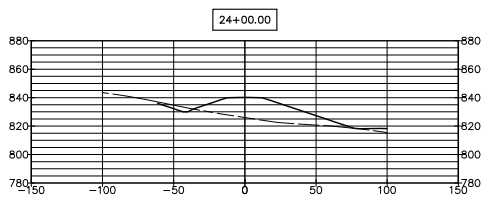


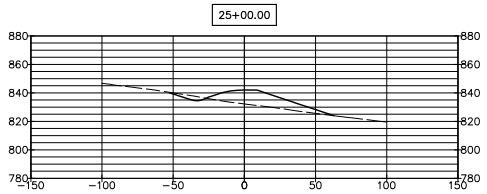


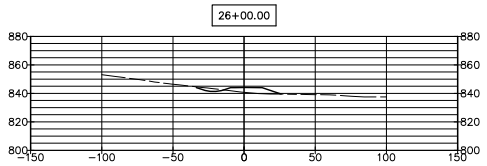


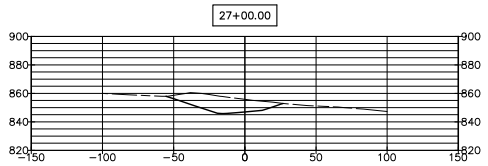


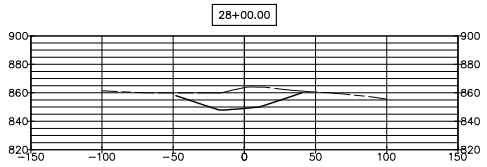


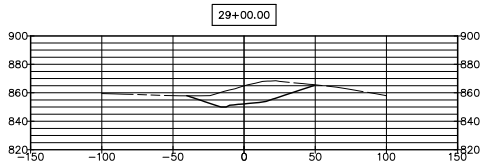


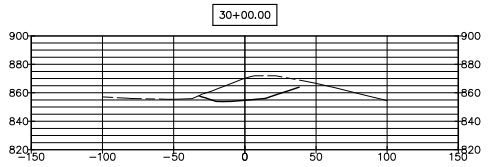












REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV NO.
R 0	02/08	JTS	SAH	JTS	HLP	BAK					D

INITIAL ISSUE; FOR W.O.# KIF 08-500095-000 RO

DISCIPLINE INTERFACE

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
PROPOSED ROAD CROSS SECTIONS
SHEET 2 OF 3

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

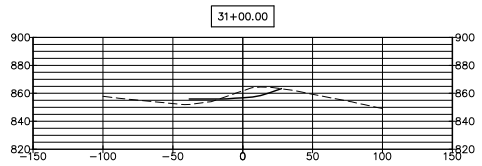
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

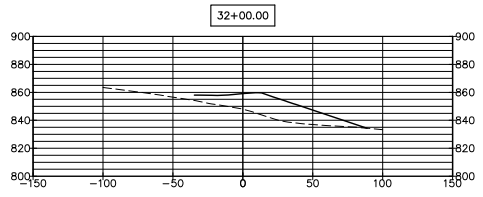
AUTOCAD R00	DATE 02/08	36	C	10W428-17	R 0

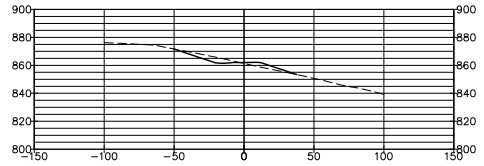
CONSOLIDATED TECH., INC.
TASK COMPLETED BY: | REV NO.

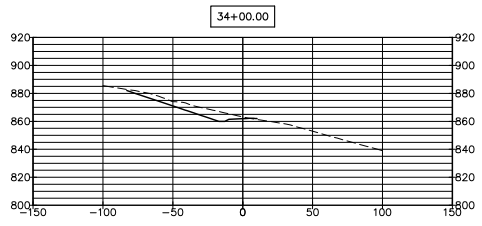
PLOT FACTOR:1:1
W_TVA

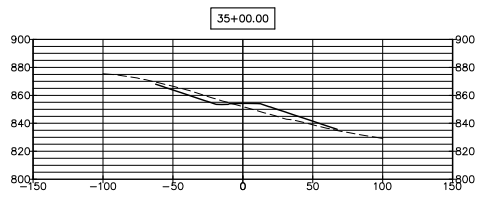
C.A.D. DRAWING
DO NOT ALTER MANUALLY

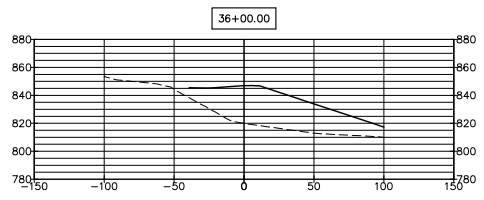


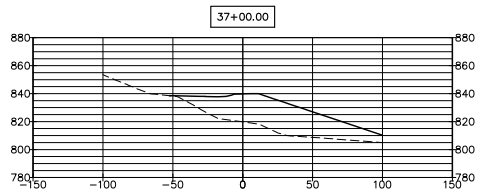


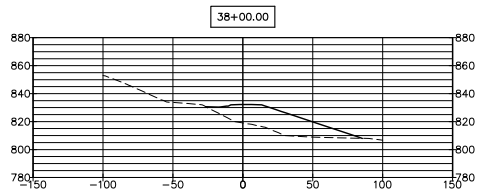


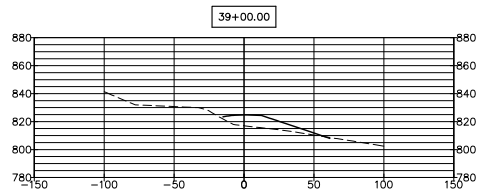


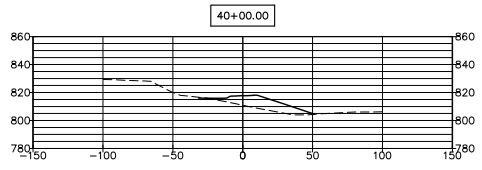


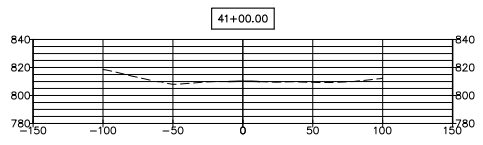












R

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	DISCIPLINE INTERFACE
R 0	02/08	JTS	SAH	JTS	HLP	BAK					D

INITIAL ISSUE; FOR W.O.# KIF 08-500095-000 R0

SCALE: 1"=50' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
PROPOSED ROAD CROSS SECTIONS
SHEET 3 OF 3

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

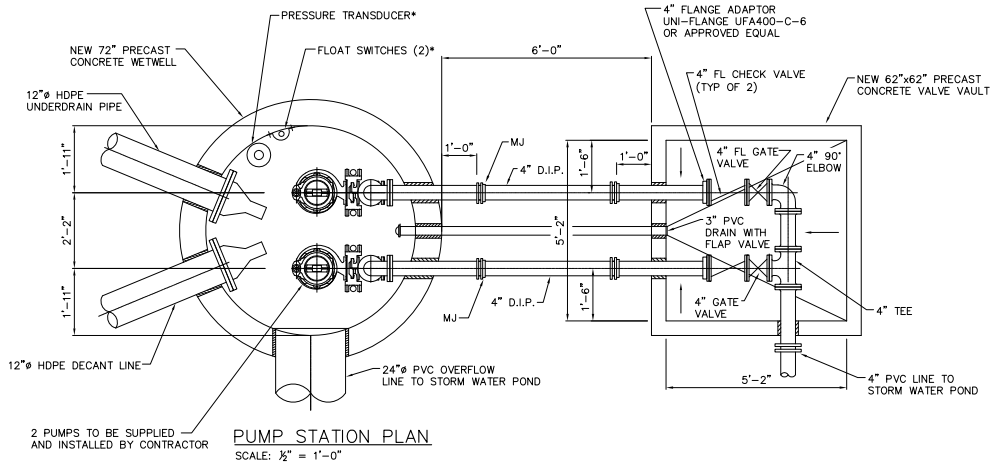
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R00	DATE 02/08	36	C	10W428-18	R 0

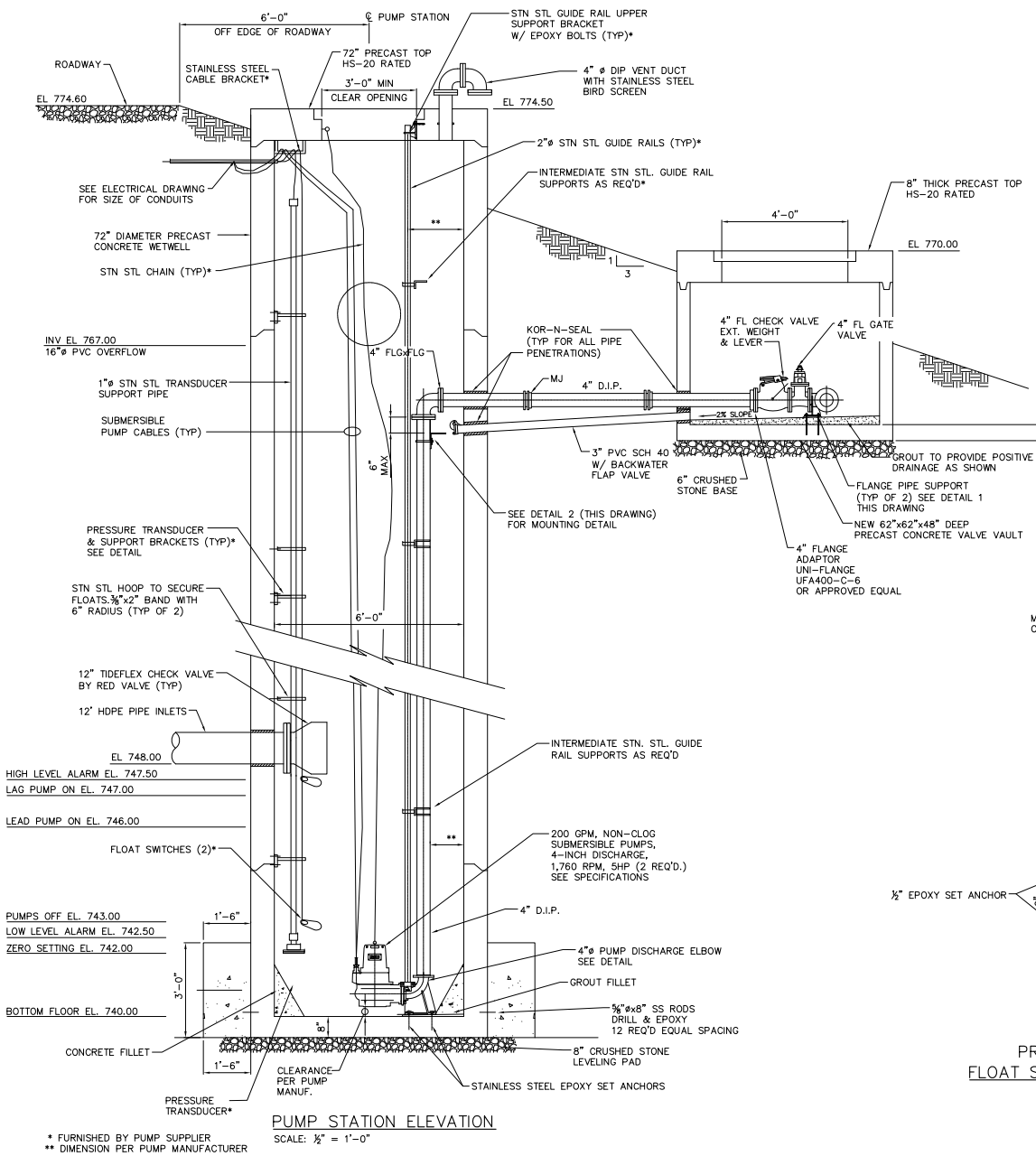
CONSOLIDATED TECH., INC.
TASK COMPLETED BY: REV NO.

PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

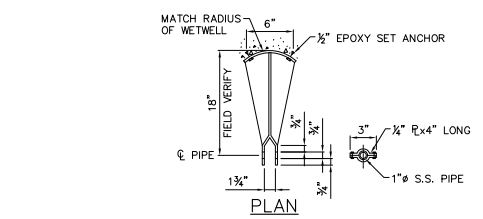


PUMP STATION PLAN
SCALE: ½" = 1'-0"

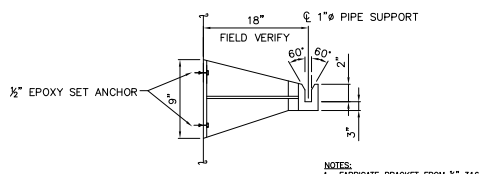


* FURNISHED BY PUMP SUPPLIER
** DIMENSION PER PUMP MANUFACTURER

PUMP STATION ELEVATION
SCALE: ½" = 1'-0"



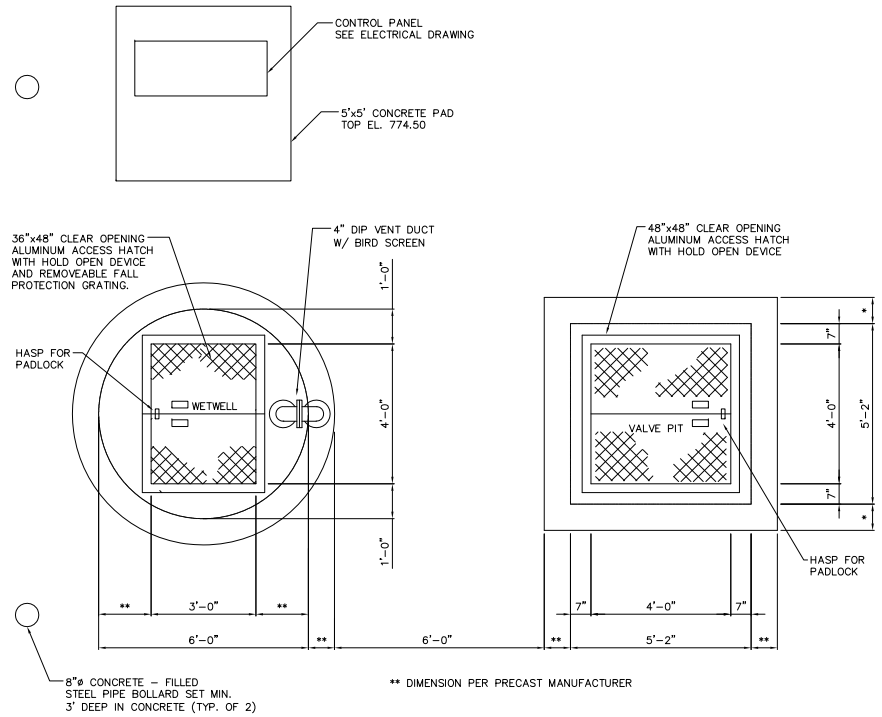
PLAN



NOTES:
1. FABRICATE BRACKET FROM ¾" 316 STAINLESS STEEL.
2. LOCATE BRACKETS SO PIPE CAN BE REMOVED THROUGH HATCH OPENING

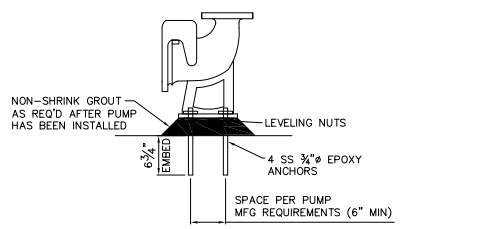
ELEVATION

PRESSURE TRANSDUCER AND FLOAT SUPPORT BRACKET (3 REQUIRED)
N.T.S.

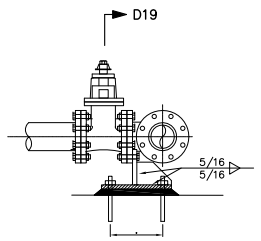


** DIMENSION PER PRECAST MANUFACTURER

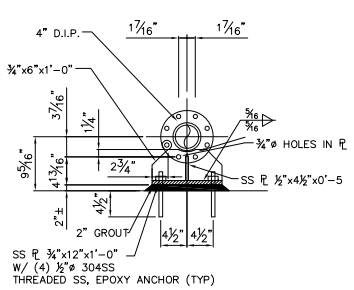
PUMP STATION TOP PLAN
SCALE: ½" = 1'-0"



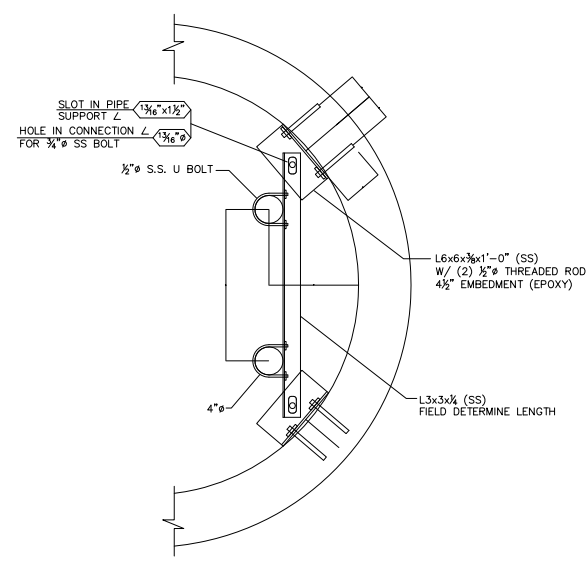
TYPICAL PUMP DISCHARGE
SCALE: 1" = 1'-0"



DETAIL 1
SCALE: 1"=1'-0"



SECTION D19
SCALE: 1"=1'-0"



DETAIL 2
SCALE: 1" = 1'-0"

GENERAL NOTES:
1. PRECAST CONCRETE MANHOLE BASE, RISERS, AND TOP SHALL CONFORM TO ASTM C478. JOINT SEALS SHALL CONSIST OF BUTYL RUBBER ROPE SEAL CONFORMING TO ASTM C990 PLUS EXTERNAL BUTYL RUBBER WRAP CONFORMING TO ASTM C877. COAT INSIDE SURFACES OF WETWELL WITH CEMENT-BASED WATERPROOFING, THOROSEAL, XYPEX, OR EQUAL.
2. PIPE TO MANHOLE CONNECTORS SHALL BE RESILIENT CONNECTORS CONFORMING TO ASTM C923, KOR-N-SEAL, OR EQUAL.
3. DUCTILE IRON PIPE SHALL CONFORM TO AWWA C151, CLASS 350. DUCTILE IRON FITTINGS SHALL CONFORM TO AWWA C110, CLASS 250. FLANGE BOLTS SHALL BE CADMIUM OR ZINC PLATED. PROVIDE STANDARD CEMENT LINING. EXTERIOR SURFACES SHALL BE PRIMED. COAT EXTERIOR SURFACES IN THE FIELD WITH TWO COATS OF COAL TAR EPOXY PAINT.
4. GATE VALVES SHALL BE RESILIENT SEAT GATE VALVES CONFORMING TO AWWA C509. CHECK VALVES SHALL BE OUTSIDE LEVER AND WEIGHT VALVES CONFORMING TO AWWA C508. PROVIDE EPOXY COATING ON INSIDE AND OUTSIDE SURFACES.
5. ALL DRILLED-IN ANCHORS SHALL BE EPOXY-SET, STAINLESS STEEL.
6. EARTH BACKFILL SHALL BE COMPACTED IN 8-INCH LIFTS TO 95 PERCENT OF MAXIMUM DENSITY.

R																
R 0	02/08	JTS	SAH	JTS	HLP	BAK										D
INITIAL ISSUE; FOR W.O.# KIF 08-500095-000 R0																
REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV. NO.					

DISCIPLINE INTERFACE

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
PUMP STATION PLAN
SECTIONS AND DETAILS

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R00	DATE 02/08	36	C	10W428-19	R 0

CONSOLIDATED TECH., INC.	
TASK COMPLETED BY:	REV NO.

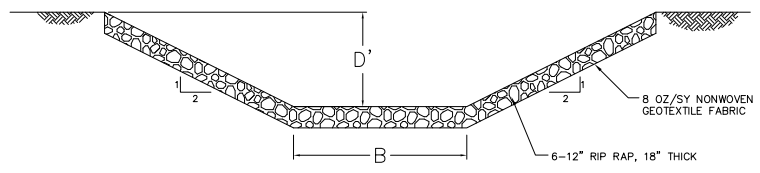
PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY



LEGEND

- 780 — EXISTING GROUND CONTOUR
- T791.2 — SPOT GROUND ELEVATION
- EXISTING UNPAVED ROAD
- EXISTING TREELINE
- EXISTING OVERHEAD ELECTRICAL TOWER
- LIMIT OF GYPSUM DISPOSAL AREA
- <830> — FINAL COVER GRADE CONTOUR (FT)
- 790 — FINISHED GRADE CONTOUR (FT)
- BENCH FLOW
- XXX — SILT FENCE
- RIPRAP
- PERIMETER ROAD
- C2 — CULVERT PIPE AND DESIGNATION
- DROP INLET
- HP — HIGH POINT ON DRAINAGE DITCH
- R1 — PERIMETER DRAINAGE CHANNEL
- RN1 — RUN-ON DRAINAGE CHANNEL



DETAIL G20
RUN-ON DRAINAGE CHANNEL
SCALE: N.T.S.
XREF:37314055
(SEE TABLE - RUN-ON DRAINAGE CHANNELS)

SUMMARY TABLE OF RUNOFF DITCH PARAMETERS - RUN-ON DRAINAGE CHANNELS

	REACH NUMBER	SLOPE, S_o (ft/ft)	REACH DIMENSIONS: LENGTH (ft)	BOTTOM WIDTH, B (ft)	DEPTH OF CHANNEL, D (ft)	SIDE SLOPES: LEFT (H:1V)	SIDE SLOPES: RIGHT (H:1V)	LINING TYPE	STARTING ELEVATION (ft)	ENDING ELEVATION (ft)
	RN1	0.1133	528.7	3.0	2.0	2.0	2.0	RIPRAP	828.00	768.08
*	RN2	0.0231	432.3	3.0	2.0	2.0	2.0	GRASS	828.00	818.00
*	RN3	0.0215	371.4	3.0	2.0	2.0	2.0	RIPRAP	818.00	810.00
	RN4	0.0727	731.1	3.0	2.0	2.0	2.0	RIPRAP	861.90	808.75
	RN5	0.0281	591.7	3.0	3.0	2.0	2.0	RIPRAP	808.12	791.50
	RN6	0.0337	947.6	3.0	2.0	2.0	2.0	RIPRAP	861.90	829.92
	RN7	0.0368	490.9	3.0	2.0	2.0	2.0	RIPRAP	848.00	829.92
	RN8	0.1664	69.0	3.0	2.0	2.0	2.0	RIPRAP	829.12	817.64
	RN9	0.0162	955.7	3.0	2.0	2.0	2.0	GRASS	848.00	834.16
	RN10	0.0730	892.0	3.0	2.0	2.0	2.0	RIPRAP	836.00	770.00
DELETED →	~~RN11~~	~~0.0760~~	~~378.5~~	~~3.0~~	~~2.0~~	~~2.0~~	~~2.0~~	~~RIPRAP~~	~~794.93~~	~~766.18~~
	RN12	0.0229	430.0	4.0	5.0	2.0	2.0	RIPRAP	765.33	755.50
	RN13	0.0179	246.6	4.0	5.0	2.0	2.0	RIPRAP	770.60	766.18
	RN14	0.0155	712.3	4.0	5.0	2.0	2.0	RIPRAP	781.64	770.60
	RN15	0.0282	1158.2	6.0	3.0	2.0	2.0	RIPRAP	814.27	781.64
	RN16	0.0104	311.5	6.0	3.0	2.0	2.0	RIPRAP	817.50	814.27
*	RN17	0.0117	416.6	0.0	3.0	2.0	2.0	GRASS	822.36	817.50
*	RN18	0.0149	392.0	3.0	5.0	2.0	2.0	GRASS	822.36	816.53
*	RN19	0.0639	391.7	3.0	6.0	2.0	2.0	RIPRAP	816.53	791.50
*	RN20	0.0060	229.0	3.0	29.0	2.0	2.0	GRASS	791.50	790.12
*	RN21	0.0231	872.8	3.0	8.0	2.0	2.0	RIPRAP	790.12	770.00
*	RN22	0.0129	465.1	18.0	3.0	2.0	2.0	GRASS	770.00	764.00
*	RN23	0.0289	154.8	1.0	2.0	2.0	2.0	RIPRAP	767.38	764.00
	RN24	0.0224	91.5	4.0	5.0	2.0	2.0	RIPRAP	751.64	749.59

* DITCHES TO BE CONSTRUCTED IN THE FUTURE

	CULVERT ID	CULVERT Type	DIAMETER (in)	LENGTH (ft)	STARTING ELEVATION (ft)	ENDING ELEVATION (ft)	SLOPE (ft/ft)
	C1	Concrete	(3) x 48"	54.2	759.39	758.31	0.020
*	C2	Concrete	(1) x 36"	69.8	768.08	767.38	0.010
*	C3	Concrete	(1) x 36"	69.8	810.00	808.37	0.023
	C4	Concrete	(1) x 36"	63.5	808.75	808.12	0.010
	C5	Concrete	(1) x 48"	80.0	829.92	829.12	0.010
	C6	Concrete	(2) x 48"	80.4	766.18	765.33	0.011
	C7	Concrete	(3) x 48"	160.0	755.50	751.64	0.025
	C8	Concrete	(3) x 48"	60.0	749.59	748.05	0.025

* PIPES TO BE INSTALLED IN THE FUTURE

NOTE:
STORM DRAINAGE PIPE SHALL BE REINFORCED CONCRETE PIPE (RCP) CONFORMING TO ASTM C76, CLASS III, WALL B. JOINTS SHALL BE MADE WITH PORTLAND CEMENT MORTAR SSRBC, SECTION 905, SUBSECTION 905.02 OR RUBBER GASKETS (ASTM C443).

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	DISCIPLINE INTERFACE
R											
R 0	02/08	JTS	SAH	JTS	HLP	BAK				D	
INITIAL ISSUE; FOR W.O.# KIF 08-500095-000 RO											

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY
STORM DRAINAGE PLAN

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

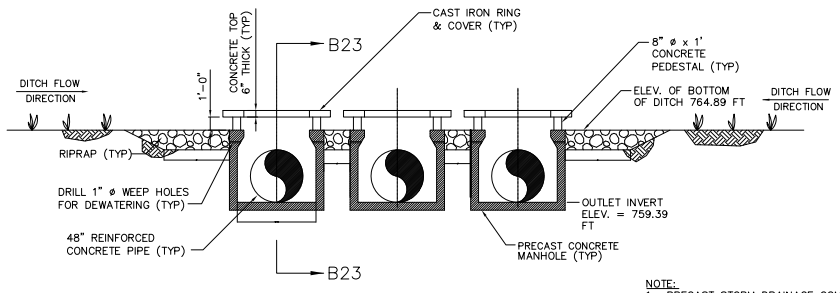
KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R00	DATE 02/08	36	C	10W428-20	R 0

CONSOLIDATED TECH., INC.
TASK COMPLETED BY: | REV NO.

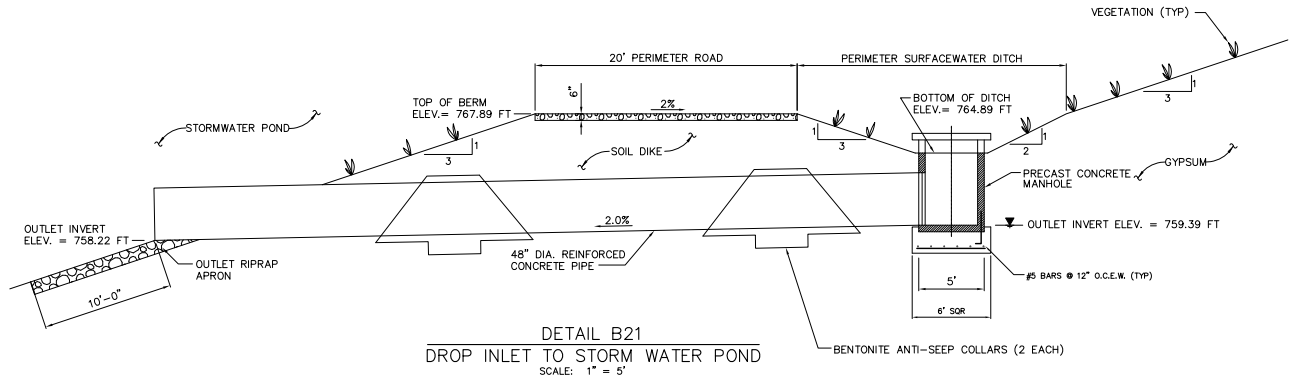
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W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

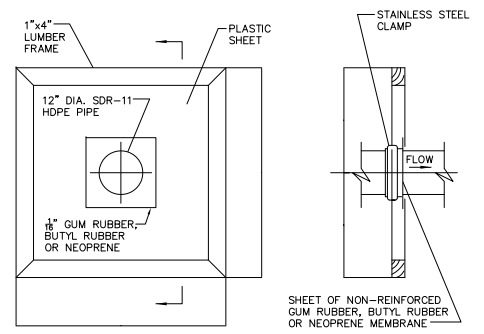


DETAIL A21
STORM WATER POND DROP INLET
SCALE: 1" = 5'

NOTE:
1. PRECAST STORM DRAINAGE COMPONENTS SHALL BE MANUFACTURED IN CONFORMANCE WITH ASTM C478. JOINT SEALS SHALL CONSIST OF BUTYL RUBBER ROPE CONFORMING TO ASTM C990.



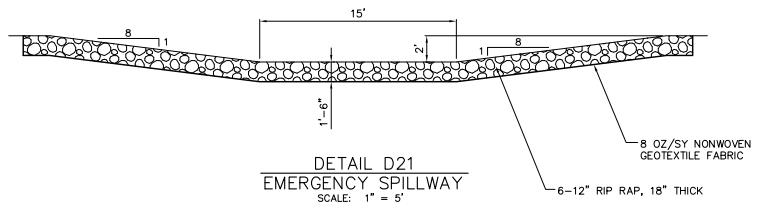
DETAIL B21
DROP INLET TO STORM WATER POND
SCALE: 1" = 5'



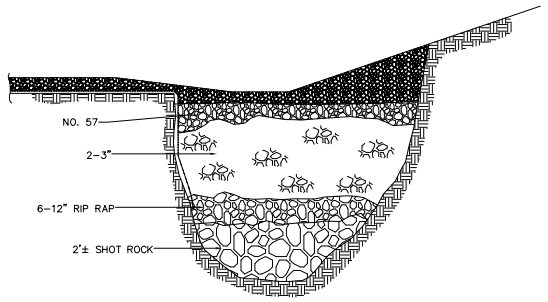
ELEVATION

SECTION

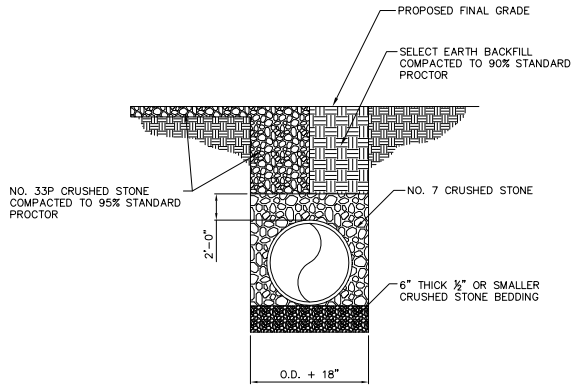
DETAIL C21
FLEXIBLE ANTI-SEEP COLLAR (NOTE 1)



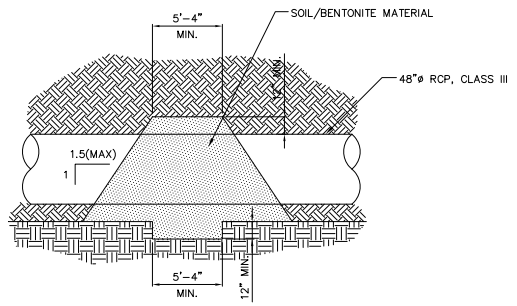
DETAIL D21
EMERGENCY SPILLWAY
SCALE: 1" = 5'



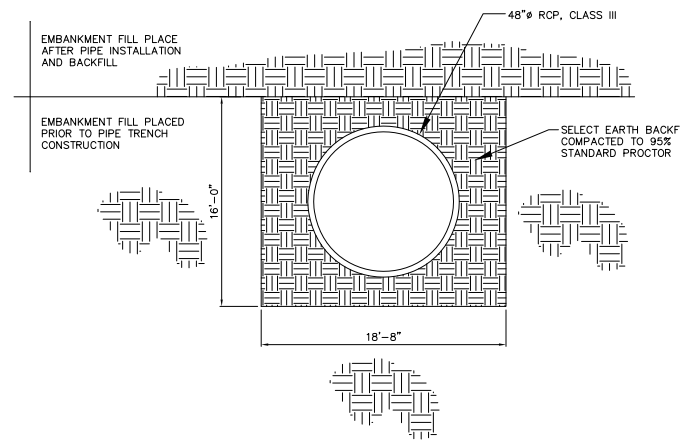
SUBGRADE STABILIZATION(IF REQUIRED)



TYPICAL PIPE INSTALLATION DETAIL FOR RCP AND HDPE
N.T.S.



BENTONITE/EARTH ANTI-SEEP COLLAR
SCALE: ¼"=1'-0"



48" RCP DROP INLET PIPE INSTALLATION DETAIL
N.T.S.

R
R 0 | 02/08 | JTS | SAH | JTS | HLP | BAK | D
INITIAL ISSUE; FOR W.O.# KIF 08-500095-000 RO

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY DRAINAGE SYSTEM DETAILS SHEET 1 OF 2

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

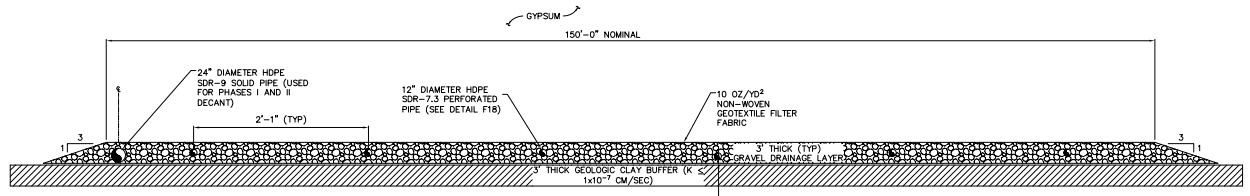
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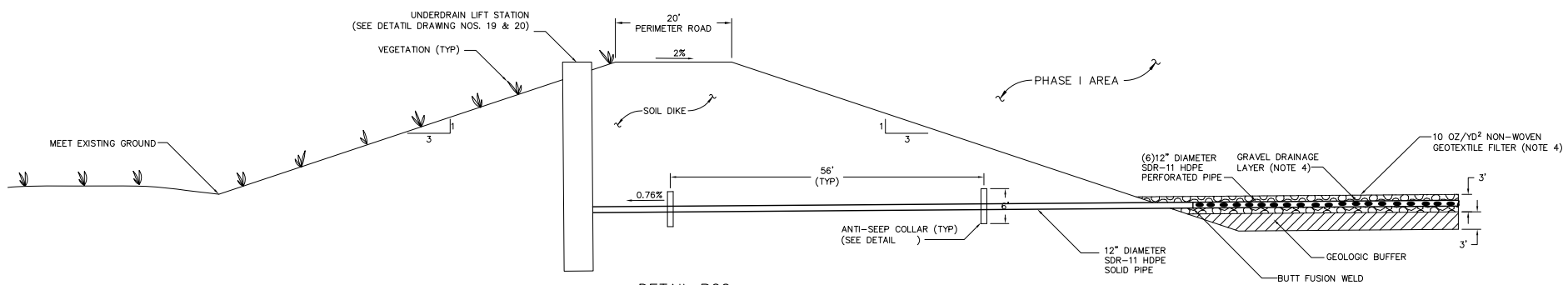
CONSOLIDATED TECH., INC.
TASK COMPLETED BY: | REV NO.

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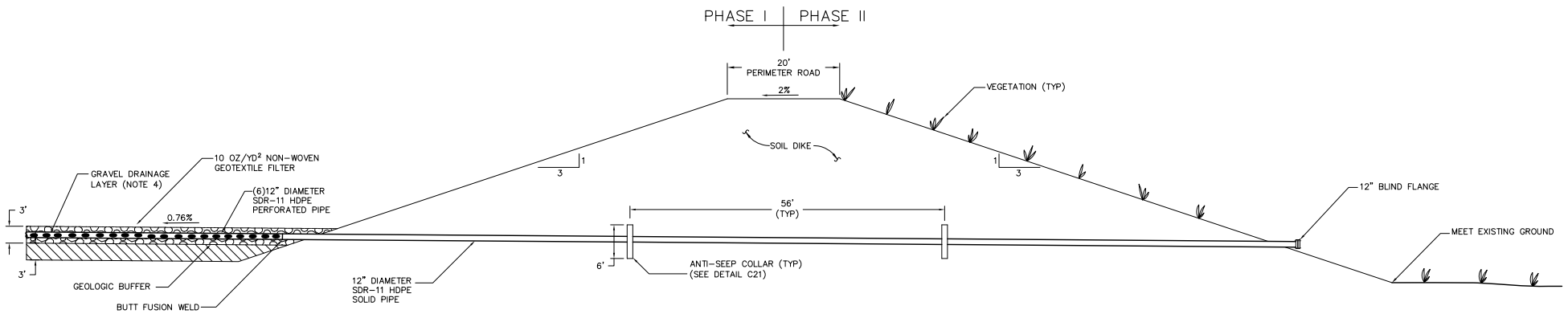
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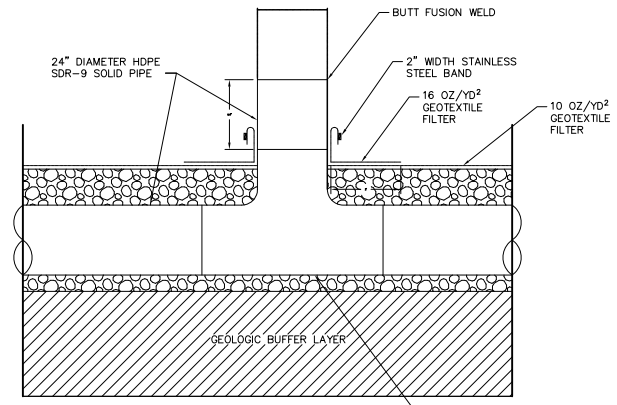
DETAIL A22
GEOLOGIC BUFFER AND CENTRAL DRAINAGE CORRIDOR (NOTE 2)



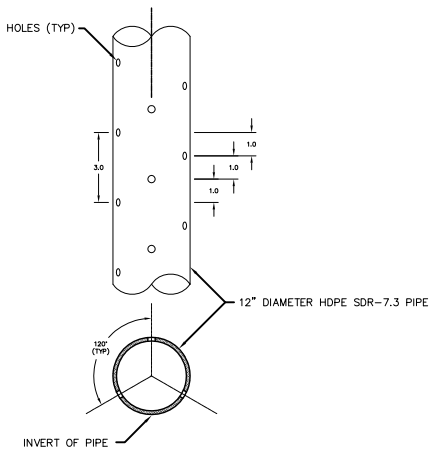
DETAIL B22
DRAINAGE PIPE PENETRATION OF PERIMETER SOIL DIKE
SCALE: 1" = 10'



DETAIL C22
PHASE I CENTRAL DRAINAGE CORRIDOR TERMINATION
SCALE: 1" = 10'



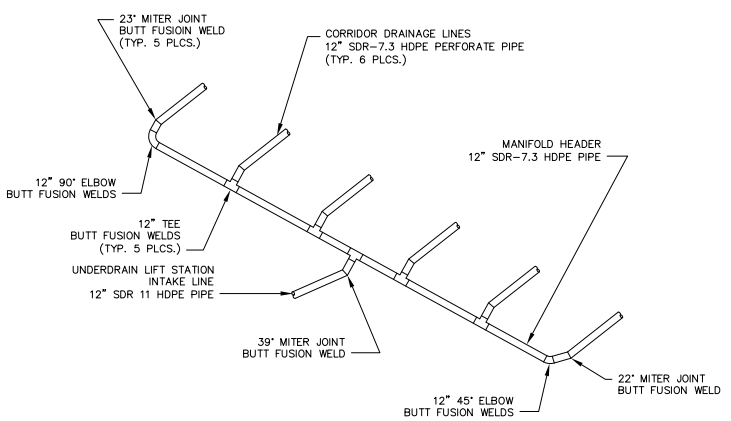
DETAIL D22
PHASE I PIPE DECANT STRUCTURE (NOTE 2)
SCALE: 1" = 2'



DETAIL E22
PERFORATED DRAINAGE PIPE
SCALE: NTS

NOTES:

1. AVOID VEHICLE TRAFFIC ON TOP OF OUTLET PIPE.
2. THE GEOLOGIC CLAY BUFFER AND COMPONENTS OF THE CENTRAL DRAINAGE CORRIDOR ARE CONSTRUCTED IN ACCORDANCE WITH THE MATERIAL SPECIFICATION AND CONSTRUCTION QUALITY ASSURANCE AND QUALITY CONTROL PLAN INCLUDED IN APPENDIX F OF THIS PERMIT APPLICATION.



DETAIL F22
UNDERDRAIN MANIFOLD

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV. NO.
R 0	02/08	JTS	SAH	JTS	HLP	BAK					D

INITIAL ISSUE; FOR W.O.# KIF 08-500095-000 R0

SCALE: 1"=200' EXCEPT AS NOTED

YARD

FGD GYPSUM DISPOSAL FACILITY DRAINAGE SYSTEM DETAILS SHEET 2 OF 2

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
J.T. STEIN	S.A. HAMBY	J.T. STEIN				

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R00	DATE 02/08	36	C	10W428-22	R 0

CONSOLIDATED TECH., INC.
TASK COMPLETED BY: | REV NO.

PLOT FACTOR:1:1
W_TVA

C.A.D. DRAWING
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US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 3

TVA Spreadsheet "CCR Generation and Handling," dated September 27, 2011

Questions:	Kingston
1. Does the utility have drawings showing the CCR generation/handling/storage train for:	
a. Fly Ash	Yes
b. Bottom Ash	Yes
c. Boiler Slag	Yes
d. FGD wastes	Yes
2. What specific equipment is used to collect, handle, and store CCR material? For:	
a. Fly Ash	SCR Hoppers, Economizer Hoppers, Precipitator Hoppers, jet pumps, hydroveyors, air separator tanks, piping, ash pond
b. Bottom Ash	Bottom Ash Hoppers, jet pumps, piping, ash pond
c. Boiler Slag	N/A
d. FGD wastes	Limestone preparation facilities, absorbors, recycle pumps, area sump, piping, FGD pond
3. Is there design information on the handling and transport equipment?	Yes
a. Example:size and length of pipe for sluicing the CCR	Yes
b. Is equipment within a secondary containment or just sitting on the ground?	SCR Hoppers, Economizer Hoppers, Precipitator Hoppers and bottom ash hoppers are inside a building. Jet pumps, hydroveyors, air separator tanks, piping, ash pond
c. Volume of storage silo	5265 Tons
4. What equipment is outside versus enclosed?	SCR Hoppers, Economizer Hoppers, Precipitator Hoppers and bottom ash hoppers are inside a building. Jet pumps, hydroveyors, air separator tanks, piping, ash pond
5. Has there ever been a release of CCR to the environment from the collection/handling/disposal system?	Yes, hydroveyor release to sump, ash lines parted at coupling
6. How much CCR per hour are they handling in each system, actual and design?	

US EPA ARCHIVE DOCUMENT

US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 4

Tennessee Valley Authority (TVA) Routine Handling Operations and Maintenance (RHO&M) Operations Support Document, Kingston Fossil Plant, July 2011

100	**TABLE OF CONTENTS**	.....	**1 to 2**
200	**OVERVIEW**	.....	**1 to 6**
300[1]	**ROLES AND RESPONSIBILITIES**	.....	**1 to 16**
400[1]	**CONTACTS**	.....	**1 to 3**
500	**HISTORIC INFORMATION**	.....	**N/A**
501	Historical Timeline	.....	1 to 1
502	Pipe Inventory	.....	1 to 23
503	Instrumentation Plan and 2010 Factor of Safety Results	.....	1 to 2
504	Seepage Log	.....	1 to 7
505	FEMA Flood Map	.....	1 to 3
600[2]	**CONTRACT**	.....	**1 to 2**
700	**CONSTRUCTION PLAN**	.....	**1 to 72**
701	10W427 Peninsula Site	.....	1 to 31
702	10W429 Ballfield Interim Ash Staging Area	.....	1 to 10
800	**DRAWING INVENTORY LIST**	.....	**1 to 1**
900	**PERMITS**	.....	**1 to 1**
901	Operations Requirement Matrix	.....	1 to 39
902	NPDES Flow Schematic	.....	1 to 2
903	NEPA Commitments	.....	1 to 1
1000[1]	**SURVEY**	.....	**1 to 3**

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**

1100	**PROCEDURES**		**1 to 7**
	1101	Operations Manuals – Ballfield Operations Plan	1 to 24
		Operations Manuals – Peninsula Site Operations Manual	25 to 59
	1102	Technical Specifications	1 of 1
	1103	Peninsula Site QA/QC Plan	1 to 89
	1104	General Repair Guidelines	1 to 6
	1105	RHO&M Daily Field Report	1 to 2
	1106	RHO&M Weekly Material Summary	1 to 1
	1107	Weekly Facility Observation Form	1 to 1
	1108	Monthly/Quarterly/Special Facility Observation Form	1 to 1
	1109	Project Startup Checklist	1 to 1
	1110	RHO&M Additional Work/Change Order Approval Form	1 to 1
	1111	Procedures for Compliance with the National Environmental Policy Act	1 to 15
	1112	RHO&M Work Control	1 to 14
	1113	Clay Dike Restrictions	1 to 1
1200[(1)]	**WORK PACKAGES**		**1 to 25**
1300[(1)]	**MARKETING**		**1 to 1**
1400	**PLANNING**		**N/A**
	1401	Remaining Life Calculations	1 to 1
	1402	Long Range Plan	1 to 1
	1403	Master Strategy Plan	1 to 8

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**

US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 5

Stantec Consulting Services Correspondence Dated October 4, 2011, "Reference: Dam Safety Hazard Classification Review, Ash Disposal Area 2, Kingston Fossil Plant, Roane County, Tennessee"

US EPA ARCHIVE DOCUMENT

Dam Safety Hazard Classification Review

Kingston Main Ash and Stilling Pond
Kingston Fossil Plant
Roane County, Tennessee

Stantec Consulting Services Inc.
One Team. Infinite Solutions
11687 Lebanon Road
Cincinnati, OH 45241-2012
Tel: (513) 842-8200 • Fax: (513) 842-8250
www.stantec.com

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

October, 2011

Stantec Consulting Services Inc.
11687 Lebanon Road
Cincinnati OH 45241-2012
Tel: (513) 842-8200
Fax: (513) 842-8250

October 04, 2011
File: 175630008

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

Reference: **Dam Safety Hazard Classification Review**
Ash Disposal Area 2
Kingston Fossil Plant
Roane County, Tennessee

Dear Mr. Turnbow:

Stantec has a completed a review of the hazard classification assessments for facilities at the Kingston Fossil Plant. The attached report summarizes the likely impacts of potential failure scenarios and provides conclusions relative to hazard classification.

We appreciate the opportunity to assist you with your dam safety program. If you have any questions or comments, please call.

Sincerely,

STANTEC CONSULTING SERVICES INC.

Dan Hoffman
Project Engineer

John Menninger, PE
Senior Project Engineer

/lfb

US EPA ARCHIVE DOCUMENT

Dam Safety Hazard Classification Review

Kingston Main Ash and Stilling Pond
Kingston Fossil Plant
Roane County, Tennessee

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

October, 2011

Dam Safety Hazard Classification Review

Kingston Main Ash and Stilling Pond
Kingston Fossil Plant
Roane County, Tennessee

Table of Contents

Section **Page No.**

1. Study Description 1

2. Potential Failure Scenarios 1

2.1. Main Ash Pond and stilling Pond 1

2.1.1. Facility Description 1

2.1.2. Failure Scenario 1

3. Hazard Classification 1

4. References 1

US EPA ARCHIVE DOCUMENT

US EPA ARCHIVE DOCUMENT

Dam Safety Hazard Classification Review

Kingston Main Ash and Stilling Pond
Kingston Fossil Plant
Roane County, Tennessee

1. Study Description

Kingston Fossil Plant is located on the northern shore of Watts Bar Lake, an impoundment of the Clinch River, at the confluence of the Clinch and Emory Rivers. The Clinch River bounds the plant to the south and the Emory River bounds the plant to the east. A natural hill slopes up from the plant to the northwest.

Kingston Fossil Plant has two active ash impoundment facilities: the Main Ash Pond and the Stilling Pond. In July 2009 TVA reviewed the dam safety hazard classification for each of its active coal combustion product (CCP) impoundments and determined these impoundments to be a Significant Hazard[(1)]. The Main Ash Pond and Stilling Pond are both contained by Dike C and act as impoundment areas for settling sluiced ash from the Kingston Fossil Plant.

Stantec has been requested to review the hazard classification assessment and either confirm or recommend a revised classification. This report summarizes the likely impacts of potential failure scenarios and provides conclusions relative to hazard classification. This review is based on available topographic and aerial mapping.

2. Potential Failure Scenarios

2.1. Main Ash Pond and Stilling Pond

2.1.1. Facility Description

The Main Ash Pond and Stilling Pond facilities are located on the northeastern side of Kingston Fossil Plant. The facilities are ringed by Dike C and have a combined surface area of approximately 110 acres. The crest elevation of the impoundment is 765 feet which is 24 feet above the normal pool elevation of Watts Bar Lake (El. 741 ft). The internal dike dividing the Main Ash Pond and Stilling Pond crests at approximate elevation 765 feet. To the northwest, the facility is bounded by the Ash Dredge Cell. The layout is presented as Figure 2.1.

2.1.2. Failure Scenario

One failure scenario for the Main Ash Pond and Stilling Pond facilities was reviewed with regards to the impact to the surrounding structures and infrastructure. The failure is through the dike which separates the facilities from Watts Bar Lake. A breach at this location would discharge directly into Watts Bar Lake or the plant intake channel. The minimum width of the lake at the location of a potential breach is approximately 800 feet. The banks of the river in the vicinity of the plant had multiple private residential structures prior to the failure of the Kingston Dredge Pond in 2008. In response to the failure in 2008, TVA purchased many of the residential structures as shown in Figure 2.1. The three remaining structures in the vicinity of the impoundments, are located an estimated 17 feet or more above normal pool of the lake based on available USGS topographic mapping[(2)] and August 2011 aerial survey data provided

by TVA. Stantec experience with modeling similar impoundment failures into large reservoirs or rivers suggest the likely flood wave will not significantly raise the water surface elevation and thus will not impact the remaining private residential structures on the opposite bank making modeling of the breach unnecessary. Figure 2.1 illustrates the potential breach scenarios.

A breach to the northwest of the Main Ash Pond and Stilling Pond facilities is unlikely because of the presence of the Dredge Cell, and is in the process of being closed.

3. Hazard Classification

No infrastructure, except those owned by TVA, was identified within the direct paths of the identified potential breach scenarios for the Main Ash Pond and Stilling Pond at the Kingston Fossil Plant. Buildings and publicly owned roadways within the vicinity of the Kingston Fossil Plant facilities are either located above the elevation of the impoundment or are separated by distances which would permit the dissipation of a potential breach wave within existing waterways.

A breach of Main Ash Pond and Stilling Pond represents a minor risk to external infrastructure and does not, in Stantec's opinion, represent a probable threat to human life. However, a breach would likely result in the off-site release of CCP's into waters of the United States. Based on the Federal Guidelines for Dam Safety - Hazard Potential Classification System for Dams[3], it is recommended that the dam safety hazard classification of the Main Ash Pond and Stilling Pond remain at Significant Hazard.

This study addresses the current facility conditions. It is recommended that the hazard classification be revisited once the ash pond is closed in association with the ongoing Dredge Cell closure followed by the planned Stilling Pond closure.

4. References

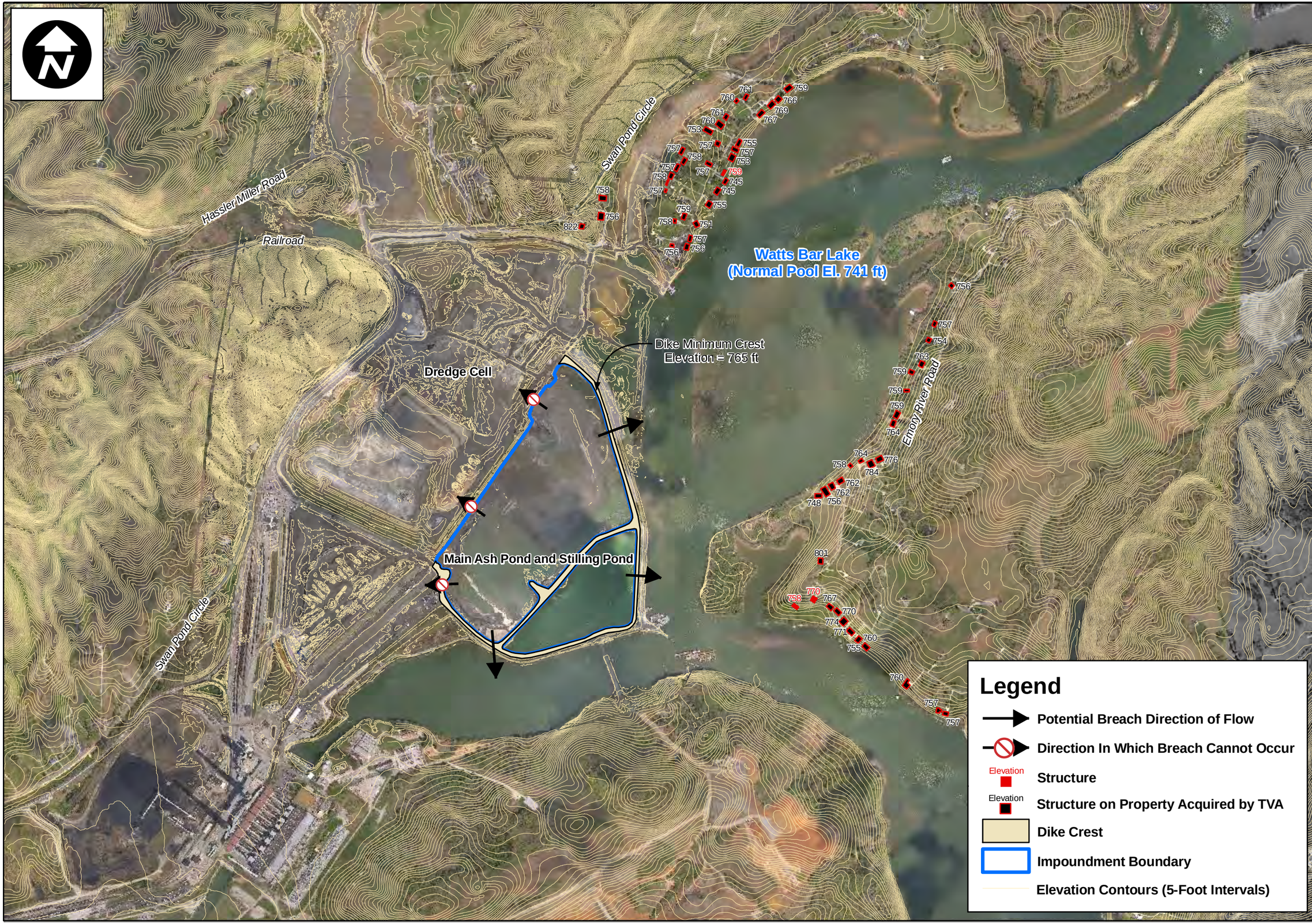
1. Tennessee Valley Authority (TVA). (2009). *Evaluation of Fossil Coal Combustion Products (CCP) Facilities for Dam Safety Hazard Classification*.

2. United States Geological Survey (USGS). (2009). *1/3-Arc Second National Elevation Dataset*. USGS, Sioux Falls, South Dakota.

3. Federal Emergency Management Association (FEMA). (2004). *Federal Guidelines for Dam Safety: Hazard Potential Classification System for Dams (FEMA 333)*. Interagency Committee on Dam Safety, FEMA.

US EPA ARCHIVE DOCUMENT

US EPA ARCHIVE DOCUMENT



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Stantec

Main Ash Pond and Stilling Pond Breach Scenarios

TVA Hazard Classification
Kingston Fossil Plant
Tennessee Valley Authority
Roane County, Tennessee

PROJECT NO.	175630008
DATE	September 21, 2011
DRAWN BY	DEH
CHECKED BY	X
CHECKED BY	X
SCALE	1" = 1000'
REVISED	
1.	
2.	
3.	
4.	
5.	
6.	
7.	
8.	

SHEET

Figure 2.1

US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 6

"Plans for Construction, Diffuser Replacement Project, Stilling Pond Work Plan 6 (KIF-110503-WP-6) Kingston Fossil Plant" Drawing Numbers 81W403-01 through 81W403-07

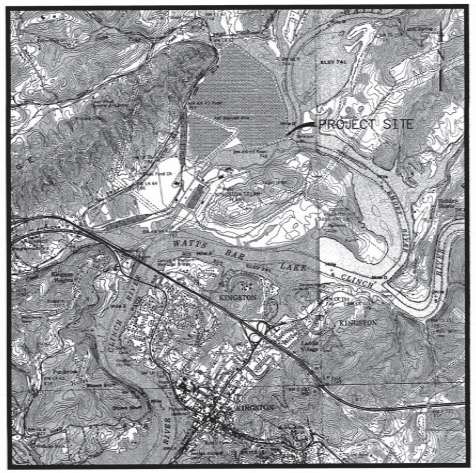
PLANS FOR CONSTRUCTION

DIFFUSER REPLACEMENT PROJECT
STILLING POND
WORK PLAN 6 (KIF-110503-WP-6)
KINGSTON FOSSIL PLANT

HARRIMAN, ROANE COUNTY, TENNESSEE

PREPARED FOR

TENNESSEE VALLEY AUTHORITY



VICINITY MAP

1000 0 2000 4000 FEET
GRAPHIC SCALE

PREPARED BY

Stantec Consulting Services Inc.
1859 Bowles Ave., Ste 250
St. Louis, Missouri
63026-1944
Tel. 636.343.3880
Fax 636.343.3554
www.stantec.com

INDEX OF SHEETS

81W403-01	COVER SHEET
81W403-02	GENERAL NOTES
81W403-03	EXISTING CONDITIONS & BASELINES
81W403-04	PLAN – DIFFUSER REPLACEMENT
81W403-05	DETAILS – SIPHON SPILLWAYS
81W403-06	DETAILS – SIPHON SPILLWAYS
81W403-07	DETAILS – DIFFUSER REPLACEMENT

ISSUED FOR CONSTRUCTION

FOR SUPPORTING DESIGN CALCULATIONS SEE FPGK1FFESCDX00030020110002

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	PVWD	APPD	ISSD	PROJECT ID	AS CONST
R 0	05/03/11	LJP	TMM	MAH	HMW	VJD	MST	JCK	–	–
–	ISSUED FOR CONSTRUCTION									

SCALE: AS SHOWN EXCEPT AS NOTED

YARD
STILLING POND

DIFFUSER REPLACEMENT PROJECT
COVER SHEET
WORK PLAN 6 (KIF–110503–WP–6)

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
L.J. PERKINS	T.M. MYERS	M.A. HOY	H.M. WILLIS	V.J. DOTSON	M.S. TURNBOW	J.C. KAMMEYER

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000 | DATE 05/03/11 | 36 | C | 81W403–01 | R 0

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Fax 636.343.3554
www.stantec.com

STANTEC	0
TASK COMPLETED BY:	REV NO.

PLOT FACTOR:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

36 | C | 81W403–01

GENERAL NOTES

SUMMARY OF WORK

THE WORK DESCRIBED IN THIS DRAWING SET WILL BE PERFORMED IN THE STILLING POND AND PLANT INTAKE CHANNEL AT THE TENNESSEE VALLEY AUTHORITY (TVA) KINGSTON FOSSIL PLANT (KIF). WORK INCLUDES INSTALLATION OF SIPHON SPILLWAYS TO MAINTAIN THE TYPICAL DAILY PROCESS FLOW FROM THE PLANT WHILE THE EXISTING SPILLWAYS ARE DEACTIVATED TO ALLOW FOR INSTALLATION OF BUTTRESS MATERIAL (BY OTHERS) AND REPLACEMENT OF THE EXISTING SPILLWAY DIFFUSERS.

1.0 GENERAL PROVISIONS

1.1 THE ATTACHED PLANS, TECHNICAL SPECIFICATIONS AND QUALITY CONTROL PLAN WERE PREPARED BY STANTEC CONSULTING SERVICES INC. (STANTEC). FOR THE CONSTRUCTION ASPECTS SHOWN HEREIN THE TENNESSEE VALLEY AUTHORITY (TVA) IS THE OWNER AND STANTEC IS THE ENGINEER. THE CONTRACTOR SHALL BE RETAINED UNDER CONTRACT BY THE OWNER.

1.2 THE CONTRACTOR SHALL SUBMIT DETAILED WORKPLANS OUTLINING ALL PROVISIONS TO OBTAIN REGULATORY APPROVAL PRIOR TO CONSTRUCTION. THE CONTRACTOR SHALL SUBMIT SAID WORKPLANS TO THE OWNER FOR APPROVAL AND TO THE ENGINEER.

1.3 THE CONTRACTOR SHALL CONFINE ALL SPECIFIC WORK ACTIVITIES TO THE AREA DEFINED BY THE PLANS OR APPROVED BY THE OWNER. ACCESS INTO THE WORK AREA FOR DELIVERY OF EQUIPMENT, MATERIALS AND WORKFORCE SHALL BE REVIEWED DAILY BY THE CONTRACTOR, AND CONTROLLED AS NEEDED TO PREVENT ANY DAMAGE TO THE CREST AND SLOPES OF THE DIKES SURROUNDING THE ASH POND.

1.4 THE CONTRACTOR SHALL COORDINATE WITH THE OWNER TO DETERMINE THE LOCATION AND AREA FOR EQUIPMENT OR MATERIAL STORAGE AND FOR OTHER CONSTRUCTION LAY DOWN ACTIVITY.

1.5 WHENEVER REFERENCE IS MADE TO TENNESSEE DEPARTMENT OF TRANSPORTATION (TDOT) STANDARD SPECIFICATIONS, AMERICAN CONCRETE INSTITUTE (ACI), AMERICAN SOCIETY OF TESTING AND MATERIALS (ASTM) AND OTHER PUBLISHED STANDARDS OR SPECIFICATIONS, IT SHALL MEAN THE LATEST VERSION IN ITS ENTIRETY.

1.6 ACCEPTABLE CONSTRUCTION TOLERANCES FROM PLAN DIMENSIONS, ELEVATIONS, AND GRADES SHALL BE AS FOLLOWS:
–EXCAVATIONS AND FINAL GRADES FOR EARTH SURFACES AND SLOPES: ± 0.25 FEET (FT)
–RIPRAP: FINAL GRADE ±0.5 FT, THICKNESS ZERO TO +0.5 FT
–FINAL GRADE TOP OF DAM: ZERO TO +0.25 FT
–MEASUREMENTS WITHIN 0.5 INCHES OF PLAN DIMENSIONS
–SIPHON PIPES AND APPURTENANCES: ±0.10 FT OF PLAN ELEVATION
–SIPHON AND DIFFUSER PIPES: ±2 FT OF DESIGNATED HORIZONTAL ALIGNMENT

1.7 NO DEVIATIONS FROM THE PLANS OR APPROVED SHOP DRAWINGS SHALL BE MADE WITHOUT PRIOR APPROVAL FROM THE ENGINEER. THE CONTRACTOR SHALL MAINTAIN A RECORD OF ALL DEVIATIONS IN LOCATION OR ELEVATION OF ANY INSTALLATION FROM THAT SHOWN ON THE PLANS, AND ANY DEVIATIONS IN INSTALLATIONS FROM APPROVED SHOP DRAWINGS. AT COMPLETION OF THE PROJECT A SET OF RECORD DRAWINGS WILL BE PREPARED BY THE CONSTRUCTION QUALITY ASSURANCE (CQA) REPRESENTATIVE BASED ON THE AS–BUILT RECORD PROVIDED BY THE CONTRACTOR. THE CONTRACTOR SHALL COOPERATE FULLY AND ASSIST WITH PREPARATION OF THE FINAL RECORD DRAWINGS.

2.0 SITE CONDITIONS

2.1 TOPOGRAPHIC INFORMATION:
UNLESS OTHERWISE INDICATED, THESE DRAWINGS WERE COMPILED USING SURVEY INFORMATION PROVIDED BY TVA. AUTOCAD DRAWING "1–24–11 POINTS.DWG" AND LIDAR DATA FROM A SURVEY PERFORMED IN APRIL OF 2009 BY TUCK MAPPING SOLUTIONS, INC WERE UTILIZED. HORIZONTAL COORDINATES ARE REFERENCED TO TENNESSEE STATE PLANE COORDINATE SYSTEM, NAD 27. ELEVATIONS ARE BASED ON NGVD 29. TOPOGRAPHIC INFORMATION SHOWN IN THE DRAWINGS IS DATED AND DOES NOT REFLECT CHANGES SINCE THE TIME OF AERIAL PHOTOGRAPHY. AREAS OF HYDROGRAPHIC SURVEYING MAY NOT REFLECT CURRENT CONDITIONS AS THE UNDERWATER SURFACE CONSTANTLY CHANGES.

2.2 THE CONTRACTOR SHALL NOTIFY KIF PLANT PERSONNEL 48 HOURS IN ADVANCE OF RAISED DIKE ACCESS ROAD CUTTING TO ALLOW FOR COORDINATION OF ALTERNATIVE TRAFFIC ROUTES.

2.3 VEHICULAR ACCESS TO THE SITE SHALL BE DETERMINED BY TVA. NO CONSTRUCTION ACTIVITIES SHALL BE PERFORMED THAT CAUSE AN INTERFERENCE WITH THROUGH TRAFFIC DURING CONSTRUCTION. CONTRACTOR SHALL PROVIDE APPROVED SIGNAGE, BARRIERS AND TRAFFIC SAFETY PERSONNEL RESPONSIBLE FOR DIRECTING CONSTRUCTION AND PLANT TRAFFIC AT ALL POINTS OF INGRESS AND EGRESS.

2.4 CONTRACTOR SHALL BE RESPONSIBLE FOR LOCATING UTILITIES PRIOR TO COMMENCEMENT OF WORK. UTILITIES AND INSTRUMENTATION SHOWN ON THE ATTACHED PLANS SHALL BE CONSIDERED APPROXIMATE AND DO NOT COVER ALL EXISTING UTILITIES IN THE AREA. CONTRACTOR SHALL EXERCISE EXTREME CAUTION DURING EXCAVATION. SHOULD PIPING OR OTHER UTILITIES THAT ARE NOT SHOWN ON THE DRAWINGS BE ENCOUNTERED, CONTRACTOR SHALL NOTIFY THE ENGINEER IMMEDIATELY, WHO WILL IN TURN NOTIFY TVA PLANT PERSONNEL IMMEDIATELY. WHERE APPLICABLE, THE CONTRACTOR SHALL PROVIDE NECESSARY MEASURES TO PROTECT UTILITIES FROM DAMAGE DUE TO CONTRACTORS WORK.

2.5 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 OCCUPATIONAL SAFETY AND HEALTH ADMINISTRATION (OSHA) STANDARDS AND TVA SAFETY REQUIREMENTS.

2.6 THE CONTRACTOR IS RESPONSIBLE FOR SITE DRAINAGE THROUGHOUT CONSTRUCTION AND SHALL INSTALL TEMPORARY DRAINAGE STRUCTURES OR PUMP WATER AS NECESSARY TO PREVENT INTERFERENCE WITH THE WORK.

3.0 SURVEY

3.1 STAKING:
THE CONTRACTOR SHALL BE RESPONSIBLE FOR ALL SURVEYING AND STAKING NECESSARY FOR LAYOUT AND CONSTRUCTION OF THE PROJECT. STAKING SHALL BE PERFORMED BY OR UNDER THE DIRECTION OF A LICENSED LAND SURVEYOR.

3.2 CONTROL:
A GLOBAL POSITIONING SYSTEM (GPS) BASE STATION HAS BEEN ESTABLISHED AND TRANSFORMATION PARAMETERS DETERMINED BY TVA USING SELECTED SURVEY CONTROL MONUMENTS. CONTACT WITH TVA SURVEYING DEPARTMENT (423)751–8416 OR (423)751–2571 SHALL BE MADE BEFORE ANY SURVEY OR CONSTRUCTION WORK IS COMMENCED. BASE STATION FREQUENCIES AND TRANSFORMATION PARAMETERS WILL BE PROVIDED TO THE CONTRACTOR FOR USE IN CONSTRUCTION ACTIVITIES AT THE SITE. PREVIOUSLY USED OR ESTABLISHED CONTROL POINTS AND MONUMENTS SHALL NOT BE USED BY THE CONTRACTOR WITHOUT PRIOR APPROVAL BY TVA SURVEYING DEPARTMENT.

3.3 CONTRACTOR SHALL EMPLOY A COMPETENT SURVEYOR LICENSED IN THE STATE OF TENNESSEE TO ESTABLISH OR CONFIRM ALL LINES, ELEVATIONS, REFERENCE MARKS, ETC. NEEDED BY THE CONTRACTOR DURING CONSTRUCTION. THE QUANTITIES OF MATERIALS PRESENTED HERE AND IN THE BID SCHEDULE SHALL BE CONSIDERED APPROXIMATE AND ARE SUBJECT TO CHANGE BASED ON FIELD CONDITIONS.

4.0 EROSION PREVENTION AND SEDIMENT CONTROL (EPSC)

4.1 THE CONTRACTOR SHALL EXERCISE EVERY REASONABLE PRECAUTION AT ALL TIMES TO MINIMIZE SOIL EROSION AND PREVENT WATER POLLUTION BY DEPOSITION OF SEDIMENT INTO THE EMORY RIVER. EROSION PREVENTION AND SEDIMENT CONTROL MEASURES SHALL BE IMPLEMENTED AND MAINTAINED IN ACCORDANCE WITH THE TECHNICAL SPECIFICATIONS.

4.2 PRIOR TO BEGINNING CLEARING, GRUBBING, AND TOPSOIL STRIPPING ACTIVITIES, EROSION PREVENTION AND SEDIMENT CONTROL (EPSC) BEST MANAGEMENT PRACTICES (BMP'S) SHALL BE INSTALLED IN ACCORDANCE WITH THE PLANS AND SPECIFICATIONS. ANY MODIFICATIONS OF EPSC MEASUREMENTS MUST BE REVIEWED AND APPROVED BY THE ENGINEER. IF DEEMED NECESSARY BY THE ENGINEER, ADDITIONAL EPSC BMP'S SHALL BE INSTALLED.

4.3 EPSC BMP'S SHALL BE MAINTAINED AT ALL TIMES IN ACCORDANCE WITH THE PLANS AND SPECIFICATIONS. ANY DEFICIENCIES IDENTIFIED BY THE CQA REPRESENTATIVE DURING INSPECTIONS SHALL BE REMEDIED IMMEDIATELY.

4.4 ACTIONS SHALL BE TAKEN TO MINIMIZE THE TRACKING OF MUD AND SOIL FROM THE PROJECT AREA ONTO THE EXISTING PLANT ACCESS ROAD. SOIL TRACKED ONTO THE ROADWAY SHALL BE REMOVED DAILY.

4.5 WHERE CONSTRUCTION OR LAND–DISTURBING ACTIVITIES HAVE TEMPORARILY CEASED ON ANY PORTION OF THE PROJECT SITE, TEMPORARY SITE STABILIZATION MEASURES SHALL BE IMPLEMENTED AS SOON AS PRACTICABLE, BUT NO LATER THAN 15 CALENDAR DAYS AFTER THE ACTIVITY HAS CEASED.

4.6 SEEDING, MULCHING, AND FERTILIZATION OF DISTURBED AREAS SHALL BE IMPLEMENTED IN ACCORDANCE WITH THE PLANTS EXISTING STORMWATER AND EROSION CONTROL PLAN.

5.0 EARTHWORK

5.1 MATERIAL SPECIFICATIONS FOR STRUCTURAL FILL, AGGREGATE, PIPE AND OTHER CONSTRUCTION MATERIALS ARE DEFINED IN THE TECHNICAL SPECIFICATIONS UNLESS NOTED OTHERWISE.

5.2 TEMPORARY EXCAVATIONS SHALL BE PREPARED IN ACCORDANCE WITH TVA SAFETY STANDARDS (SECTION 804) AND OSHA STANDARDS. STABILITY OF EXCAVATION SLOPES IS THE RESPONSIBILITY OF THE CONTRACTOR.

5.3 NO MATERIAL SHALL BE PLACED IN ANY SECTION OF THE EMBANKMENT UNTIL THE FOUNDATION FOR THAT SECTION HAS BEEN APPROVED BY THE ENGINEER OR CQA REPRESENTATIVE. THE CONTRACTOR SHALL KEEP THE FOUNDATION AND SUBGRADE FREE FROM WATER OR UNACCEPTABLE MATERIALS AFTER FILL OPERATIONS HAVE STARTED.

5.4 MATERIAL THAT IS PLACED ADJACENT TO AND LESS THAN 2 FT ABOVE THE TOP OF SIPHON PIPES SHALL BE COMPACTED USING HAND–DIRECTED COMPACTORS (JUMPING JACK TYPE COMPACTOR/RAMMER) WITH A MAXIMUM LIFT THICKNESS OF 4–INCHES.

6.0 SIPHON SPILLWAY OPERATION

6.1 THE SIPHON SPILLWAYS SHALL BE INSTALLED AND OPERATED TO TEMPORARILY DRAWDOWN THE POOL IN ADDITION TO MAINTAINING THE TYPICAL DAILY PROCESS FLOW DURING EXISTING SPILLWAY DIFFUSER REPLACEMENT.

6.2 THE CONTRACTOR SHALL INSTALL MIRAFI 500X (OR ENGINEER APPROVED EQUIVALENT) GEOTEXTILE FABRIC BENEATH THE TORPEDO STRAINERS TO MINIMIZE MIGRATION OF ASH.

6.3 THE CONTRACTOR SHALL INSTALL A CURTAIN BOOM AROUND THE SIPHON SPILLWAYS TO PREVENT CENOSPHERES AND DEBRIS FROM ENTERING THE TORPEDO STRAINERS.

6.4 THE CONTRACTOR SHALL INSTALL A STAFF GAGE WITH ALTERNATING MARKS AT 0.1 FT INCREMENTS AND WHOLE FOOT CALL–OUTS. THE CONTRACTOR SHALL MONITOR THE GAGE AND REGULATE THE SIPHON FLOWS TO MAINTAIN THE POND ELEVATION DURING DIFFUSER REPLACEMENT.

6.5 THE CONTRACTOR SHALL BE RESPONSIBLE FOR INSTALLING, PERIODICALLY PRIMING (INCLUDING RENTAL OR PURCHASE OF A GODWIN DRI–PRIME PUMP, OR APPROVED EQUAL), MAINTAINING AND REGULATING THE FLOW OF THE SIPHONS IN A MANNER THAT ENSURES SAFETY AND STABILITY OF THE ASH POND DIKES AT ALL TIMES DURING CONSTRUCTION. THE CONTRACTOR SHALL IMMEDIATELY HALT POOL LOWERING IF ANY SIGN OF INTERIOR PERIMETER SLOPE INSTABILITY IS NOTED.

6.6 DRAWDOWN (USING SIPHONS) WILL RESULT IN A DECREASED POND SURFACE AREA DURING CONSTRUCTION. THIS DECREASED SURFACE AREA MAY AFFECT SETTLING TIMES. THE OWNER SHALL BE RESPONSIBLE FOR MITIGATING ANY ADVERSE IMPACTS TO TOTAL SUSPENDED SOLID (TSS) LEVELS.

6.7 THE OUTLETS OF THE EXISTING SPILLWAYS ARE A NPDES PERMITTED DISCHARGE. DURING CONSTRUCTION, WORK IN THE ASH POND MAY CAUSE CHANGES IN PERMITTED EFFLUENT CHARACTERISTICS. THE OWNER SHALL BE RESPONSIBLE FOR MITIGATING EFFECTS OF POOL DRAWDOWN ON PH, TSS, AND DIKE STABILITY.

6.8 THE ROUTINE HANDLING AND OPERATIONS (RHO&M) PROGRAM MANAGER MUST BE NOTIFIED BEFORE CHANGES IN POOL LEVEL OCCUR.

6.9 THE CONTRACTOR SHALL VERIFY NO CENOSPHERES ARE PRESENT NEAR SIPHONS AND THAT CURTAIN BOOM IS IN PLACE PRIOR TO ACTIVATION.

7.0 DIFFUSER REPLACEMENT

7.1 THE CONTRACTOR SHALL PROVIDE A DETAILED WORK PLAN FOR THE REMOVAL AND REPLACEMENT OF THE DIFFUSER PIPES TO THE ENGINEER AT LEAST 15 DAYS PRIOR TO BEGINNING WORK. THE DETAILED WORKPLAN SHALL INCLUDE THE METHOD USED TO REMOVE THE EXISTING DIFFUSER PIPES AND INSTALL THE NEW HDPE DIFFUSER PIPES. THE FLOAT AND SINK METHOD SHALL BE USED TO INSTALL THE DIFFUSER PIPES. BALLAST WEIGHTS SHALL BE SECURED TO PIPE ON LAND. PIPE ENDS SHALL BE PLUGGED AND PIPE SHALL BE FLOATED INTO PLACE ALONG PROPOSED ALIGNMENT. PIPE SHALL BE SLOWLY FILLED WITH WATER SUCH THAT A CONTROLLED SINKING OCCURS.

7.2 ANY HARDWARE USED FOR ATTACHMENT OF BALLAST WEIGHTS TO DIFFUSER PIPE SHALL BE STAINLESS STEEL OR GALVANIZED.

7.3 DIVERS SHALL VERIFY INSTALLED HDPE PIPE IS NOT RESTING ON ANY ROCKS, DEBRIS OR MATERIAL THAT COULD CAUSE DAMAGE TO THE PIPE.

7.4 DURING INSTALLATION, HDPE PIPE BENDS SHALL HAVE A MINIMUM RADIUS OF 30 FT.

8.0 SEQUENCE OF CONSTRUCTION

8.1 PRIOR TO THE COMMENCEMENT OF WORK, CONTRACTOR SHALL VERIFY BUTTRESS CONSTRUCTION HAS BEEN COMPLETED FROM APPROXIMATE STATION 124+00 TO 126+00 (BY OTHERS) TO ALLOW FOR SIPHON INSTALLATION. CONTRACTOR SHALL ALSO VERIFY THAT THE CAUSEWAY HAS BEEN MODIFIED (BY OTHERS) AS SHOWN ON THE PLANS.

8.2 INSTALL SIPHON SPILLWAYS IN ACCORDANCE WITH THE CONSTRUCTION PLANS AND TECHNICAL SPECIFICATIONS.

8.3 ACTIVATE THE SIPHONS. CONTRACTOR SHALL MONITOR AND ADJUST SIPHON VALVES TO TEMPORARILY DRAWDOWN THE POOL TO A MINIMUM ELEVATION OF 754.0 FT AND TO MAINTAIN THE TYPICAL DAILY PROCESS FLOW FROM THE PLANT.

8.4 FOLLOWING TEMPORARY DRAWDOWN TO MINIMUM ELEVATION OF 754.0 FT, CLOSE BUTTERFLY VALVES AT THE EXISTING SPILLWAY OUTLET HEADWALL IN CASE OF LOSS OF PRIME DURING DIFFUSER REMOVAL OR REPLACEMENT.

8.5 REMOVE EXISTING SPILLWAY DIFFUSERS (CONTRACTOR SHALL SUBMIT A DETAILED WORKPLAN AT LEAST 15 DAYS PRIOR TO DIFFUSER REMOVAL FOR ENGINEER APPROVAL).

8.6 REMAINING PORTION OF SEGMENT B OF DIKE C BUTTRESS TO BE CONSTRUCTED BY OTHERS.

8.7 INSTALL NEW 24–INCH HDPE DIFFUSER PIPES AS SHOWN ON THE PLANS AND IN ACCORDANCE WITH THE TECHNICAL SPECIFICATIONS (CONTRACTOR SHALL SUBMIT A DETAILED WORKPLAN AT LEAST 15 DAYS PRIOR TO DIFFUSER REPLACEMENT FOR ENGINEER APPROVAL).

8.8 FOLLOWING INSTALLATION, CONTRACTOR SHALL VERIFY THE FOLLOWING:
–ALIGNMENT OF INSTALLED DIFFUSER PIPES;
–BALLASTS SECURING THE DIFFUSER PIPE ARE PROPERLY SITTING ON THE CHANNEL BED AND THE PIPE IS NOT FORCED TO BRIDGE ANY CHANGES IN ELEVATION;
–PIPE IS NOT RESTING ON ANY ROCKS, DEBRIS OR MATERIAL THAT COULD CAUSE DAMAGE; AND
–ALL TEMPORARY MATERIALS AND EQUIPMENT USED DURING INSTALLATION HAVE BEEN REMOVED.

8.9 OPEN BUTTERFLY VALVES AND PARTIALLY CLOSE SIPHON VALVES TO RAISE THE POOL LEVEL AND ALLOW FLOW THROUGH THE NEW DIFFUSER PIPES. VERIFY THAT THE EXISTING SPILLWAYS ARE OPERATING CORRECTLY AND THERE ARE NO LEAKS IN THE NEW DIFFUSER PIPES.

8.10 UPON APPROVAL FROM THE OWNER, CLOSE SIPHON SPILLWAY VALVES.

8.11 CLEANUP SITE.

9.0 CONSTRUCTION DOCUMENTATION

9.1 CONSTRUCTION OBSERVATION AND TESTING:
THE OWNER OR DESIGNATED CQA REPRESENTATIVE SHALL PROVIDE QUALITY ASSURANCE CONTROL TESTING AND SHALL OBSERVE AND DOCUMENT CONSTRUCTION ACTIVITIES.

9.2 DAILY REPORT:
THE CONTRACTOR SITE FOREMAN OR CQA REPRESENTATIVE SHALL COMPLETE A DAILY ACTIVITIES REPORT FOR EACH DAY IN WHICH ANY CONSTRUCTION ACTIVITY OCCURS. THIS REPORT SHALL DOCUMENT THE EQUIPMENT AND PERSONNEL ACTIVE ON SITE (WITH RELEVANT TIMES NOTED) AND IT SHALL PROVIDE A GENERAL DESCRIPTION OF THE CONSTRUCTION ACTIVITIES PERFORMED DURING THE DAY. IT SHALL ALSO INCLUDE A DESCRIPTION OF ONGOING CONSTRUCTION PROGRESS. DISCUSSIONS OF INCLEMENT WEATHER, EQUIPMENT BREAKDOWNS AND ANY DAMAGE, SAFETY, ENVIRONMENTAL, ETC. INCIDENTS WHICH OCCUR SHALL BE PROVIDED. THE CONTRACTOR SHALL PROVIDE HIS OWN FORMAT FOR THIS REPORT, SUBJECT TO ENGINEER APPROVAL. IT IS EXPECTED THAT THIS REPORT SHALL BE SUBMITTED BY THE CONTRACTOR TO THE ENGINEER OR CQA REPRESENTATIVE BY NOON ON THE NEXT BUSINESS DAY FOLLOWING THE DAY WITHIN WHICH THE REPORT COVERS.

9.3 RECORD DRAWINGS
THE CONTRACTOR SHALL MAINTAIN A CLEAN SET OF CONSTRUCTION PLANS AT THE PROJECT SITE. THIS SET OF PLANS SHALL BE USED ONLY FOR RECORDING CONSTRUCTION ACTIVITY AS IT OCCURS. ALL ITEMS INSTALLED INTO THE WORK SHALL BE NOTED FOR ANY VARIATION IN MATERIAL SPECIFICATIONS, DETAILS, ELEVATIONS, OR HORIZONTAL LOCATIONS SHALL BE NOTED ON THE PLAN SET WITH RED MARKINGS. TO THE EXTENT NECESSARY, THE CONTRACTOR SHALL ENGAGE A QUALIFIED SURVEYOR TO ESTABLISH THE "AS–BUILT" LOCATION OF ALL PIPES, DRAINS, STRUCTURES, EMBANKMENT ZONE LIMITS, ETC. EACH ITEM'S POSITION SHALL BE DULY NOTED ON THIS "RED–LINE" PLAN SET. THE INFORMATION SHALL BE RECORDED WITH SUFFICIENT ACCURACY AND CLARITY TO BE UNDERSTOOD AND RELIED UPON. AT THE COMPLETION OF THE CONSTRUCTION PROCESS, THIS "RED–LINE" PLAN SET SHALL BE PROVIDED TO THE ENGINEER FOR USE IN DEVELOPING THE RECORD DRAWINGS FOR THE PROJECT. THE INFORMATION ON THE "RED–LINE" DRAWING SET SHALL BE KEPT CURRENT AT ALL TIMES AND SHALL BE AVAILABLE BY THE OWNER OR ENGINEER AT ANY TIME. THE CONTRACTOR SHALL SUBMIT THE "RED–LINE" AS–BUILT SET OF DRAWINGS TO THE CQA REPRESENTATIVE WITHIN FIVE WORKING DAYS OF COMPLETION AND ACCEPTANCE OF THE WORK DESCRIBED IN THIS WORK PACKAGE.

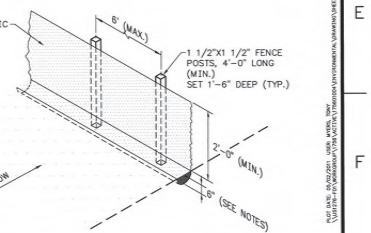
9.4 CONSTRUCTION CERTIFICATION REPORT:
THE CQA REPRESENTATIVE SHALL PREPARE AND SUBMIT A CONSTRUCTION CERTIFICATION REPORT IN ACCORDANCE WITH TVA'S MASTER PROGRAMMATIC DOCUMENTS WITHIN TWO WEEKS FOLLOWING COMPLETION OF CONSTRUCTION.

ISSUED FOR CONSTRUCTION

CONSTRUCTION QUALITY CONTROL PRODUCT SUBMITTALS

MATERIAL	SPECIFICATION	REQUIREMENTS	(FIO)/(EA)[1]	MINIMUM FREQUENCY	SUBMITTAL DATE	APPROVAL DATE
GENERAL		DETAILED WORKPLANS OUTLINING ALL PROJECT PROVISIONS TO BE SUBMITTED FOR APPROVAL BY THE OWNER.	FIO	15 DAYS PRIOR TO BEGINNING WORK		
CONCRETE						
BALLAST WEIGHTS	03 40 00	SHOP DRAWINGS OF THE PROPOSED PRE–CAST BALLAST WEIGHTS	EA	15 DAYS PRIOR TO FABRICATION		
		CERTIFIED TEST RESULTS FROM MANUFACTURER SHOWING THAT CONCRETE MEETS OR EXCEEDS ALL SPECIFICATIONS	FIO	1/SUPPLIER		
NON–SHRINK (EXPANSIVE) GROUT	03 60 00	DESIGN MIX SUBMITTAL AND SUPPLIER CERTIFICATION VERIFYING MIX MEETS SPECIFICATIONS (IF NON–COMMERCIAL GROUT MIX IS USED)	EA	15 DAYS PRIOR TO BEGINNING WORK		
		PASSING RESULTS OF STRENGTH AND EXPANSION TESTS SUBMITTED BY CQA REPRESENTATIVE (IF NON–COMMERCIAL GROUT MIX IS USED)	FIO	1/BATCH		
		DELIVERY SLIPS CERTIFYING CONTENTS OF EACH BATCH NOTING TIME OF DELIVERY	FIO	1/BATCH		
METALS						
WELDING	05 50 00	CERTIFICATION SHOWING THAT WELDERS HAVE SATISFACTORILY PASSED AWS QUALIFICATION TEST WITHIN THE PREVIOUS 12 MONTHS	FIO	PRIOR TO BEGINNING WORK		
EARTHWORK						
SITE PREPARATION	31 10 00	SITE SAFETY PLAN FOR APPROVAL BY TVA SAFETY PROFESSIONAL	FIO	PRIOR TO BEGINNING WORK		
BACKFILL AND EMBANKMENT	31 23 00	MATERIAL CLASSIFICATION OR SAMPLES FOR TESTING FROM EACH REPRESENTATIVE BORROW SOURCE	EA	1/SUPPLIER/SOURCE		
		PASSING RESULTS OF DENSITY AND MOISTURE TESTING DURING PLACEMENT AND COMPACTION.	FIO	1/4–INCH OR 8–INCH LIFT		
GEOTEXTILE FABRIC	31 23 13	CERTIFICATION STATING THAT THE PRODUCT MEETS OR EXCEEDS ALL REQUIREMENTS OF THE TECHNICAL SPECIFICATION	FIO	1/SUPPLIER/SOURCE		
STONE AND RIPRAP	31 37 00	CERTIFICATION INDICATING THE SOURCE OF STONE SURFACING IS AN "APPROVED SOURCE" (APL) PER TDOT SECTION 709.	FIO	1/SUPPLIER/SOURCE		
		GRADATION CURVE FOR THE MATERIAL AND CERTIFICATION THAT THE MATERIAL MEETS THE SPECIFICATIONS IN THE TECHNICAL SPECIFICATIONS.	FIO	1/SUPPLIER/SOURCE		
UTILITIES						
SIPHON SPILLWAYS	33 10 00	SUPPLIER SPECIFICATIONS FOR THE VALVES AND FLOAT BALLS.	FIO	1/SUPPLIER		
		RESULTS OF PASSING PRESSURE TEST FOR EACH PIPE RUN	FIO	1/PIPE RUN		
DIFFUSER REPLACEMENT	33 20 00	DETAILED WORKPLAN FOR THE REMOVAL OF THE EXISTING SPILLWAY DIFFUSERS AND INSTALLATION OF NEW SPILLWAY DIFFUSERS.	EA	15 DAYS PRIOR TO BEGINNING WORK		
		RESULTS OF PASSING PRESSURE TEST FOR EACH PIPE RUN	FIO	1/PIPE RUN		
HDPE PIPE	33 40 00	MANUFACTURER CERTIFICATION THAT THE PIPE AND EACH FITTING COMPLIES WITH THE TECHNICAL SPECIFICATIONS	FIO	1/SUPPLIER		

(1) FIO = FOR INFORMATION ONLY; EA = ENGINEER APPROVAL



NOTES:
1. INSTALL SILT FENCE IN ACCORDANCE WITH TECHNICAL SPECIFICATIONS.
2. THE BOTTOM 12 INCHES OF THE FABRIC SHALL BE BURIED IN A 6–INCH TRENCH CUT INTO THE GROUND TO PREVENT SEDIMENT FROM ESCAPING UNDER THE FENCE. ALL EARTHWORK SHALL BE ON THE UPSTREAM SIDE OF THE FENCE.
3. GEOTEXTILE FABRIC SHALL MEET THE FOLLOWING SPECIFICATIONS: TENSILE STRENGTH (ASTM D 4632) – 100 LBS. MIN., WIDTH – 36" MIN., APPARENT OPENING SIZE (ASTM D4751) NO. 30. ULTRAVIOLET DEGRADATION (ASTM D 4355) 80% STRENGTH RETAINED. ELONGATION OF 40% MAX. (ASTM D 4632).

1/2 DETAIL – SILT FENCE
NOT TO SCALE

FOR SUPPORTING DESIGN CALCULATIONS SEE FPGKIFFESCDX0003020110002

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	DISCIPLINE INTERFACE
R 0	05/03/11	LJP	TMM	MAH	HMW	VJD	MST	JCK	–	–	–
ISSUED FOR CONSTRUCTION											

SCALE: NONE EXCEPT AS NOTED

YARD
STILLING POND

DIFFUSER REPLACEMENT PROJECT
GENERAL NOTES
WORK PLAN 6 (KIF–110503–WP–6)

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
L.J. PERKINS	T.M. MYERS	M.A. HOY	H.M. WILLIS	V.J. DOTSON	M.S. TURNBOW	J.C. KAMMEYER

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000 | DATE: 05/03/11 | 36 | C | 81W403–02 | R 0

PLOT FACTOR:1
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

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STANTEC	0
TASK COMPLETED BY:	REV NO.

LAST DATE: 05/03/2011 - USER: MYERS, TMM
L:\STNTEC\FOSSIL\PROJECTS\KINGSTON\DIVISION\DRAWINGS\81W403-02-R0.DWG

36 C 81W403–02

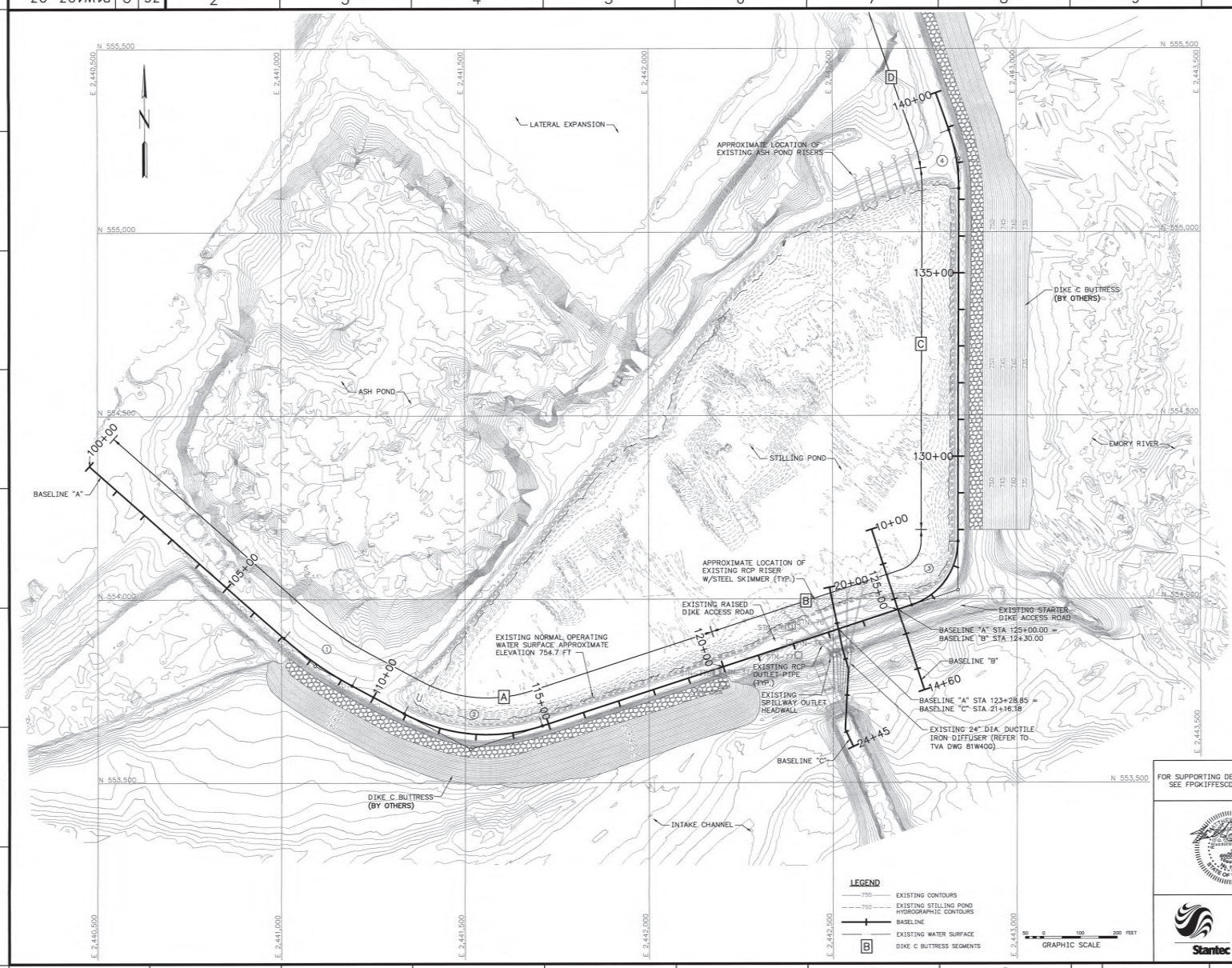


TABLE OF BASELINE COORDINATES			
STATION	BASELINE	NORTHING	EASTING
100+00	A	554,363.86	2,440,447.77
140+00	A	555,379.31	2,442,781.47
10+00	B	554,094.87	2,442,636.76
14+60	B	553,752.64	2,442,748.46
20+00	C	554,031.98	2,442,490.98
21+72.07	C	553,863.44	2,442,525.66
22+28.24	C	553,809.73	2,442,542.12
23+93.08	C	553,645.15	2,442,533.00
24+50	C	553,598.18	2,442,555.13

BASELINE CURVE DATA

①
P.I. STA. = 108+20.74
NORTHING = 553,819.18
EASTING = 2,441,091.71
Δ = 13°20'06"
R = 974.68'
T = 113.94'
L = 226.85'
E = 6.64'
P.C. STA. = 107+06.80
P.T. STA. = 109+33.65

②
P.I. STA. = 112+99.93
NORTHING = 553,591.92
EASTING = 2,441,514.77
Δ = 46°19'10"
R = 479.84'
T = 205.26'
L = 387.92'
E = 42.06'
P.C. STA. = 110+94.67
P.T. STA. = 114+82.59

③
P.I. STA. = 126+71.85
NORTHING = 554,024.61
EASTING = 2,442,840.47
Δ = 71°45'50"
R = 182.94'
T = 132.33'
L = 229.13'
E = 42.85'
P.C. STA. = 125+39.52
P.T. STA. = 127+68.65

④
P.I. STA. = 138+12.12
NORTHING = 555,200.41
EASTING = 2,442,843.77
Δ = 19°21'32"
R = 479.84'
T = 81.84'
L = 162.13'
E = 6.93'
P.C. STA. = 137+30.28
P.T. STA. = 138+92.41

INSTRUMENTATION LEGEND

STN-1 ⬡ SLOPE INCLINOMETER (SI)
STN-2 ⬡ PIEZOMETER (PZ)
STN-3 ⬡ SETTLEMENT PLATFORM (SP)

MAPPING NOTE:

1. THESE DRAWINGS WERE COMPILED USING SURVEY INFORMATION PROVIDED BY TVA.
2. AN AUTOCAD DRAWING INCLUDING TOPOGRAPHIC AND HYDROGRAPHIC SURVEY POINTS ENTITLED '1-24-11 POINTS.DWG' WAS USED FOR ELEVATIONS BELOW AND INCLUDING ELEVATION 760 FT. TOPOGRAPHIC DATA FROM A LIDAR SURVEY PERFORMED IN APRIL 2009 BY TUCK MAPPING SOLUTIONS, INC WAS USED IN AREAS OUTSIDE OF THE STILLING POND ABOVE ELEVATION 760 FT.
3. DIKE C BUTTRESS AS DEPICTED REPRESENTS THE PROPOSED DESIGN (BY OTHERS), NOT AN AS-BUILT CONFIGURATION.
4. HORIZONTAL COORDINATES ARE REFERENCED TO TENNESSEE STATE PLANE COORDINATE SYSTEM, NAD 27. ELEVATIONS ARE BASED ON NGVD 29.

SURVEY CONTROL NOTE:

A GLOBAL POSITIONING SYSTEM (GPS) BASE STATION HAS BEEN ESTABLISHED AND TRANSFORMATION PARAMETERS DETERMINED BY TVA USING SELECTED SURVEY CONTROL MONUMENTS. CONTACT WITH TVA SURVEYING DEPARTMENT (423)751-8416 OR (423)751-2571 SHALL BE MADE BEFORE ANY SURVEY OR CONSTRUCTION WORK IS COMMENCED. BASE STATION FREQUENCIES AND TRANSFORMATION PARAMETERS WILL BE PROVIDED TO THE CONTRACTOR FOR USE IN CONSTRUCTION ACTIVITIES AT THE SITE. PREVIOUSLY USED OR ESTABLISHED CONTROL POINTS AND MONUMENTS SHALL NOT BE USED BY THE CONTRACTOR WITHOUT PRIOR APPROVAL BY TVA SURVEYING DEPARTMENT.

ISSUED FOR CONSTRUCTION

FOR SUPPORTING DESIGN CALCULATIONS SEE FPGK1FFESCDX00030020110002

REV NO.	DATE	ORIGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD
R 0	05/03/11	LJP	TMM	MAH	HMW	VJD	MST	JCK
ISSUED FOR CONSTRUCTION								

SCALE: 1"=100' EXCEPT AS NOTED

YARD
STILLING POND

DIFFUSER REPLACEMENT PROJECT
EXISTING CONDITIONS & BASELINES
WORK PLAN 6 (KIF-110503-WP-6)

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
L.J. PERKINS	T.M. MYERS	M.A. HOY	H.M. WILLIS	V.J. DOTSON	M.S. TURNBOW	J.C. KAMMEYER

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

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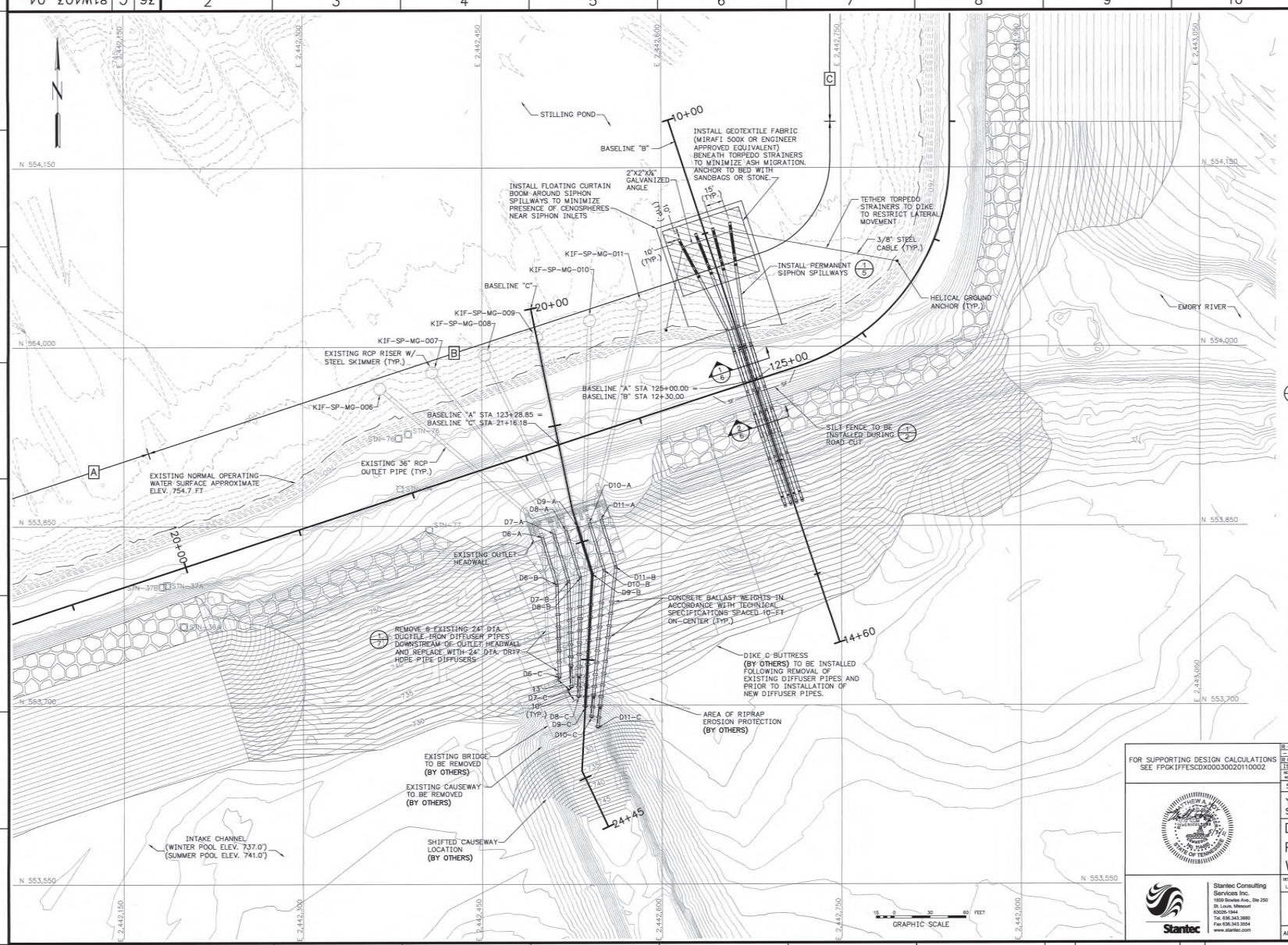
AUTOCAD R 2000 | DATE 05/03/11 | 36 | C | 81W403-03 | R 0

PLOT FACTOR:100 W_TVA

C.A.D. DRAWING DO NOT ALTER MANUALLY

STANTEC 0
TASK COMPLETED BY: REV NO.

36 C 81W403-03



DIFFUSER & SIPHON STRUCTURE SCHEDULE			
STRUCTURE REFERENCE	BASELINE	STATION	OFFSET (FT)
D6-A	C	21+87	30.8'-R
D6-B	C	22+39	28.9'-R
D6-C	C	23+16	22.8'-R
D7-A	C	21+87	20.6'-R
D7-B	C	22+35	19.3'-R
D7-C	C	23+24	12.3'-R
D8-A	C	21+87	10.3'-R
D8-B	C	22+32	9.7'-R
D8-C	C	23+32	6.0'-R
D9-A	C	21+87	0'
D9-B	C	22+28	0'
D9-C	C	23+40	0'
D10-A	C	21+86	10.5'-L
D10-B	C	22+28	10.5'-L
D10-C	C	23+48	5.7'-L
D11-A	C	21+86	20.6'-L
D11-B	C	22+28	20.6'-L
D11-C	C	23+57	11.2'-L

INSTRUMENTATION LEGEND

STN-1 SLOPE INCLINOMETER (SI)
STN-2 PIEZOMETER (PZ)
STN-3 SETTLEMENT PLATFORM (SP)

LEGEND
750 EXISTING CONTOURS
750 EXISTING STILLING POND HYDROGRAPHIC CONTOURS
750 PROPOSED CONTOURS
EXISTING WATER SURFACE
SF SILT FENCE
LIMITS OF DISTURBANCE
BASELINE
A DIKE C BUTTRESS SEGMENT

Section or Detail No.
Sheet Where Shown
REFERENCE KEY

MAPPING NOTE:

1. THESE DRAWINGS WERE COMPILED USING SURVEY INFORMATION PROVIDED BY TVA.
2. AN AUTOCAD DRAWING INCLUDING TOPOGRAPHIC AND HYDROGRAPHIC SURVEY POINTS ENTITLED '1-24-11 POINTS.DWG' WAS USED FOR ELEVATIONS BELOW AND INCLUDING ELEVATION 760 FT. TOPOGRAPHIC DATA FROM A LIDAR SURVEY PERFORMED IN APRIL 2009 BY TUCK MAPPING SOLUTIONS, INC WAS USED IN AREAS OUTSIDE OF THE STILLING POND ABOVE ELEVATION 760 FT.
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A GLOBAL POSITIONING SYSTEM (GPS) BASE STATION HAS BEEN ESTABLISHED AND TRANSFORMATION PARAMETERS DETERMINED BY TVA USING SELECTED SURVEY CONTROL MONUMENTS. CONTACT WITH TVA SURVEYING DEPARTMENT (423)751-8416 OR (423)751-2571 SHALL BE MADE BEFORE ANY SURVEY OR CONSTRUCTION WORK IS COMMENCED. BASE STATION FREQUENCIES AND TRANSFORMATION PARAMETERS WILL BE PROVIDED TO THE CONTRACTOR FOR USE IN CONSTRUCTION ACTIVITIES AT THE SITE. PREVIOUSLY USED OR ESTABLISHED CONTROL POINTS AND MONUMENTS SHALL NOT BE USED BY THE CONTRACTOR WITHOUT PRIOR APPROVAL BY TVA SURVEYING DEPARTMENT.

ISSUED FOR CONSTRUCTION

FOR SUPPORTING DESIGN CALCULATIONS SEE FPGK1FFESCDX0003002O110002

REV NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST
R 0	05/03/11	LJP	TMM	MAH	HMW	VJD	MST	JCK	–	–
ISSUED FOR CONSTRUCTION										

SCALE: 1"=30' EXCEPT AS NOTED

YARD
STILLING POND

DIFFUSER REPLACEMENT PROJECT
PLAN – DIFFUSER REPLACEMENT WORK PLAN 6 (KIF-110503-WP-6)

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
L.J. PERKINS	T.M. MYERS	M.A. HOY	H.M. WILLIS	V.J. DOTSON	M.S. TURNBOW	J.C. KAMMERER

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

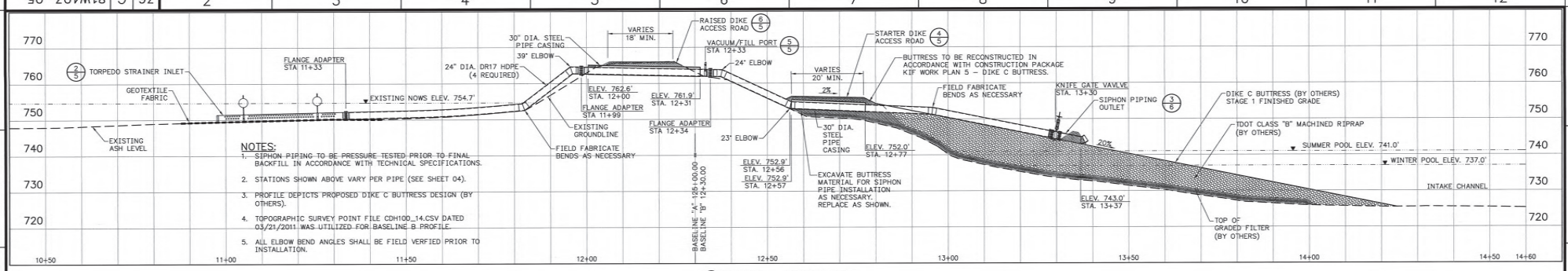
AUTOCAD R 2000 | DATE 05/03/11 | 36 | C | 81W403-04 | R 0

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STANTEC | 0
TASK COMPLETED BY: | REV NO.

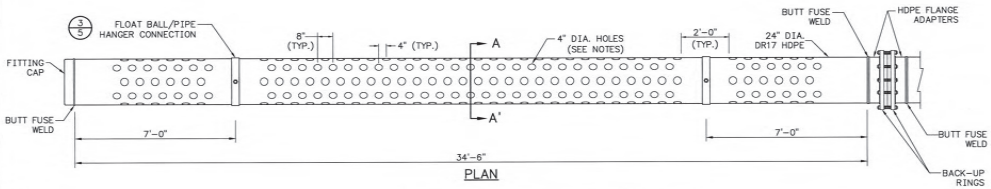
PLOT FACTOR:30
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY



NOTES:
1. SIPHON PIPING TO BE PRESSURE TESTED PRIOR TO FINAL BACKFILL IN ACCORDANCE WITH TECHNICAL SPECIFICATIONS.
2. STATIONS SHOWN ABOVE VARY PER PIPE (SEE SHEET 04).
3. PROFILE DEPICTS PROPOSED DIKE C BUTTRESS DESIGN (BY OTHERS).
4. TOPOGRAPHIC SURVEY POINT FILE CDH100_14.CSV DATED 03/21/2011 WAS UTILIZED FOR BASELINE B PROFILE.
5. ALL ELBOW BEND ANGLES SHALL BE FIELD VERFIED PRIOR TO INSTALLATION.

(1/5) PROFILE – BASELINE B
SCALE: 1"=10'



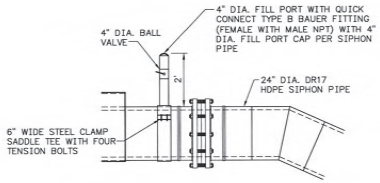
PLAN

(2/5) DETAIL – TORPEDO STRAINER INLET
SCALE: 1/2"=1'-0"

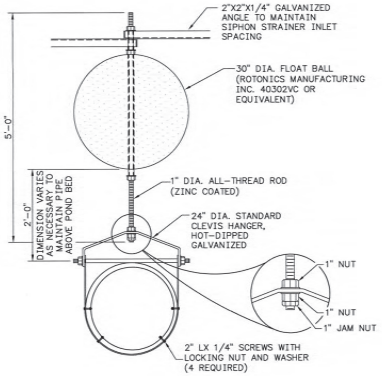


SECTION A–A'
NOT TO SCALE

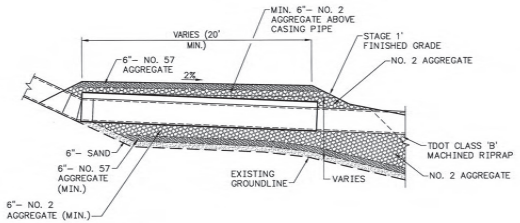
NOTES:
1. HOLES WILL BE ON TOP HALF OF PIPE ONLY.
2. 4" DIA. HOLES AT 8" CENTER TO CENTER.
3. 200 HOLES PER STRAINER (5 ROWS WITH 40 HOLES EACH).
4. ROWS ARE TO BE 8" APART CENTER TO CENTER, AND THE HOLES WILL BE OFFSET BY 4".
5. PROVIDE 2 FOOT LENGTH WITHOUT HOLES AT 7 FEET FROM EACH END FOR FLOAT BALL HANGERS.



(5/5) DETAIL – VACUUM/FILL PORT
NOT TO SCALE

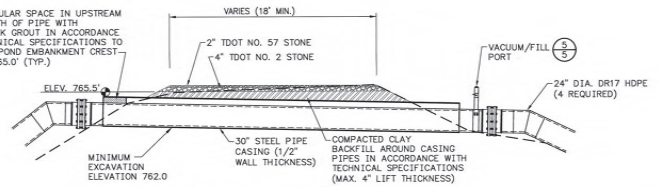


(3/5) DETAIL – FLOAT BALL/PIPE HANGER CONNECTION
SCALE: 1"=1'-0"



NOTES:
1. BUTTRESS MATERIAL AND ACCESS ROAD TO BE CONSTRUCTED BY OTHERS
2. THE MINIMUM COVER THICKNESS FROM ROAD SURFACE TO TOP OF SAND FILTER IS 36–INCHES.
3. PLACE NO. 2 AGGREGATE AND NO. 57 AGGREGATE ON RIPRAP OR NO. 2 AGGREGATE AS APPLICABLE TO FORM A REVERSE FILTER FOR ROAD SURFACE.
4. REMOVE AGGREGATE CONTAMINATED WITH ASH AS DIRECTED BY THE QC MANAGER.

(4/5) DETAIL – STARTER DIKE ACCESS ROAD
SCALE: 1/4"=1'-0"



(6/5) DETAIL – RAISED DIKE ACCESS ROAD
SCALE: 1/4"=1'-0"

ISSUED FOR CONSTRUCTION

FOR SUPPORTING DESIGN CALCULATIONS SEE FPGKIFFESCDX0003002110002

SECTION OR DETAIL NO.
SHEET WHERE SHOWN
REFERENCE KEY

LEGEND
NOWS = NORMAL OPERATING WATER SURFACE

R –	–	–	–	–	–	–	–	–	–	–	–	–	–	–
R 0	05/03/11	LJP	TMM	MAH	HMW	VJD	MST	JCK	–	–	–	–	–	–
ISSUED FOR CONSTRUCTION														
REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST				

SCALE: AS SHOWN EXCEPT AS NOTED

YARD
STILLING POND

DIFFUSER REPLACEMENT PROJECT
DETAILS – SIPHON SPILLWAYS
WORK PLAN 6 (KIF–110503–WP–6)

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
L.J. PERKINS	T.M. MYERS	M.A. HOY	H.M. WILLIS	V.J. DOTSON	M.S. TURNBOW	J.C. KAMMEYER

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000 | DATE: 05/03/11 | 36 | C | 81W403–05 | R 0

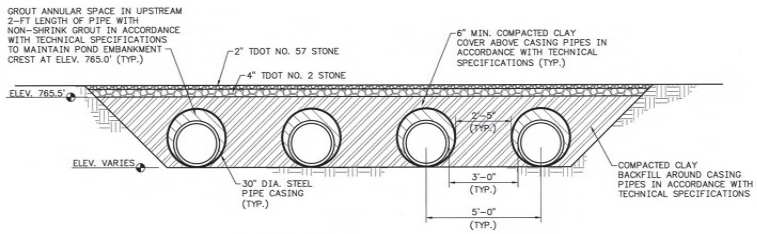
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TASK COMPLETED BY:	REV NO.

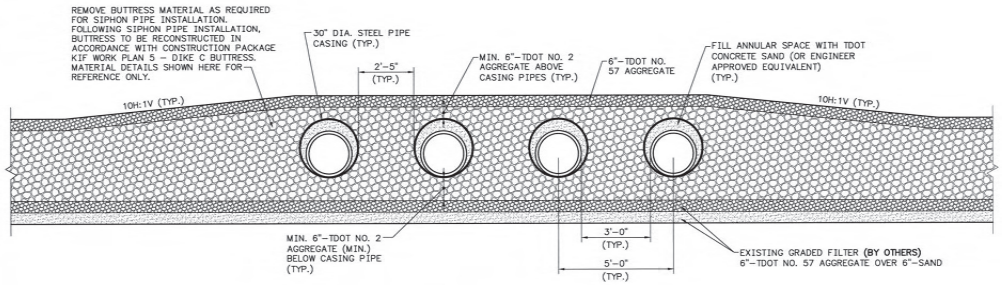
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W_TVA

C.A.D. DRAWING DO NOT ALTER MANUALLY

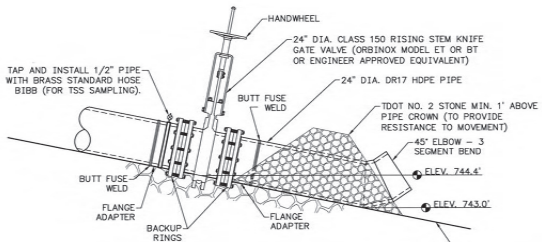
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1/6 SECTION – RAISED DIKE ACCESS ROAD
SCALE: 1/2"=1'



2/6 SECTION – STARTER DIKE ACCESS ROAD
SCALE: 1/2"=1'



3/6 DETAIL – SIPHON PIPING OUTLET
SCALE: 1/2"=1'

ISSUED FOR CONSTRUCTION

FOR SUPPORTING DESIGN CALCULATIONS SEE FPGKIFFESCDX0003002011000 2

REV	DATE	DESN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD
R 0	05/03/11	LJP	TNM	MAH	HMW	VJD	MST	JCK
	ISSUED FOR CONSTRUCTION							

SCALE: AS SHOWN — EXCEPT AS NOTED

YARD
STILLING POND

DIFFUSER REPLACEMENT PROJECT
DETAILS – SIPHON SPILLWAYS
WORK PLAN 6 (KIF-110503-WP-6)

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
L.J. PERKINS	T.M. MYERS	M.A. HOY	H.M. WILLIS	V.J. DOTSON	M.S. TURNBOW	J.C. KAMMEYER

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

Stantec Consulting Services Inc.
1859 Bowles Ave., Ste 250
St. Louis, Missouri
63205-1944
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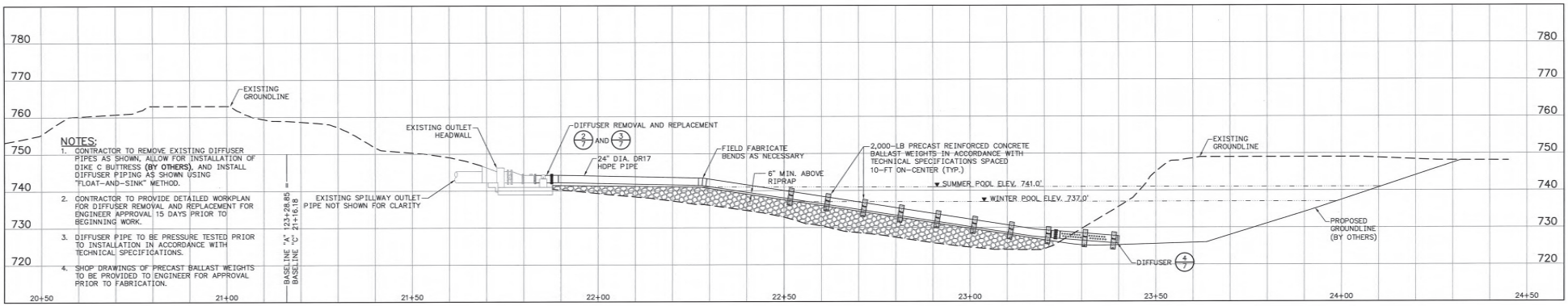
AUTOCAD R 2000 | DATE 05/03/11 | 36 | C | 81W403-06 | R 0

STANTEC	0
TASK COMPLETED BY:	REV NO.

PLOT FACTOR:1
W_TVA

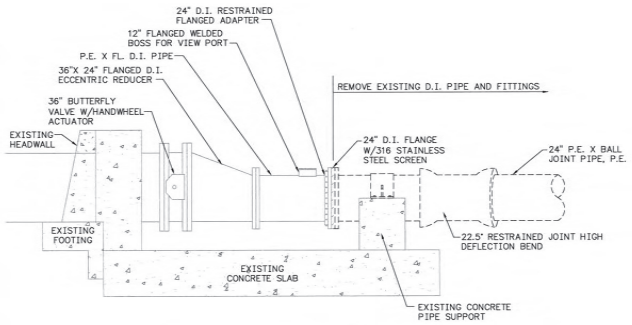
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PLOT DATE: 05/03/2011 - USER: MYERS, TNM
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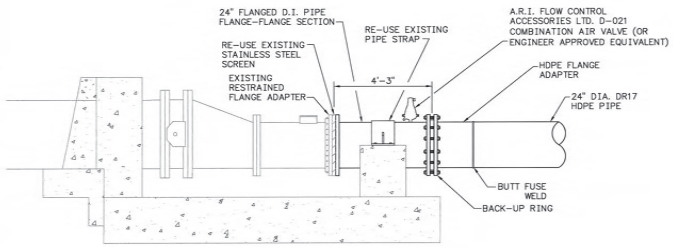


NOTES:
1. CONTRACTOR TO REMOVE EXISTING DIFFUSER PIPES AS SHOWN, ALLOW FOR INSTALLATION OF DIKE C BUTTRESS **(BY OTHERS)**, AND INSTALL DIFFUSER PIPING AS SHOWN USING "FLOAT-AND-SINK" METHOD.
2. CONTRACTOR TO PROVIDE DETAILED WORKPLAN FOR DIFFUSER REMOVAL AND REPLACEMENT FOR ENGINEER APPROVAL 15 DAYS PRIOR TO BEGINNING WORK.
3. DIFFUSER PIPE TO BE PRESSURE TESTED PRIOR TO INSTALLATION IN ACCORDANCE WITH TECHNICAL SPECIFICATIONS.
4. SHOP DRAWINGS OF PRECAST BALLAST WEIGHTS TO BE PROVIDED TO ENGINEER FOR APPROVAL PRIOR TO FABRICATION.

1/7 PROFILE – BASELINE C
SCALE: 1"=10'



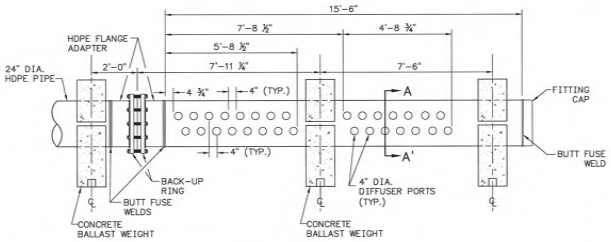
2/7 DETAIL – EXISTING DIFFUSER PIPE REMOVAL
SCALE: 1/2"=1'



3/7 DETAIL – DIFFUSER REPLACEMENT
SCALE: 1/2"=1'



SECTION A–A' – DIFFUSER HOLE ORIENTATION



ELEVATION VIEW

4/7 DETAIL – DIFFUSER
SCALE: 1/2"=1'

NOTE:
1. CONTRACTOR SHALL ASSESS CONDITION OF EXISTING DIFFUSER UPON REMOVAL. IF EXISTING DIFFUSER IS IN ADEQUATE CONDITION, IT MAY BE RE-USED UPON APPROVAL FROM THE ENGINEER.
2. 24" HDPE BLIND FLANGE REQUIRED FOR INSTALLATION OF DIFFUSER PIPING.

SECTION OR DETAIL NO.
SHEET WHERE SHOWN
REFERENCE KEY

ISSUED FOR CONSTRUCTION

FOR SUPPORTING DESIGN CALCULATIONS SEE FPGKIFFESCDX0003002010002

R-	–	–	–	–	–	–	–	–	–	–	–	–
R 0	05/03/11	LJP	TNM	MAH	HMW	VJD	MST	JCK	–	–	CONSTRUCTION ENTRANCE
ISSUED FOR CONSTRUCTION											
REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	

SCALE: AS SHOWN EXCEPT AS NOTED

YARD
STILLING POND

DIFFUSER REPLACEMENT PROJECT
DETAILS – DIFFUSER REPLACEMENT
WORK PLAN 6 (KIF-110503-WP-6)

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
L.J. PERKINS	T.M. MYERS	M.A. HOY	H.M. WILLIS	V.J. DOTSON	M.S. TURNBOW	J.C. KAMMEYER

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000 | DATE: 05/03/11 | 36 | C | 81W403-07 | R 0

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STANTEC	0
TASK COMPLETED BY:	REV NO.

PLOT FACTOR:10
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

PLOT DATE: 05/03/2011 - 10:09, M:\TVA\KIF\ESC\DWG\DIFFUSER\DIFFUSER REPLACEMENT\DRAWINGS\SHEET_FILES\DIFFUSER_DETAILS-07-R0.DWG

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APPENDIX A

Document 7

Doc 2010 Annual Inspection of CCP Facilities and Ponds, Kingston Fossil Plant, Harriman, Tennessee, Stantec Consulting Services, June 1, 2010

US EPA ARCHIVE DOCUMENT

2010 Annual Inspection of CCP Facilities and Ponds

Kingston Fossil Plant
Harriman, Tennessee

Stantec Consulting Services Inc.
One Team. Infinite Solutions.
1901 Nelson Miller Parkway
Louisville KY 40223-2177
Tel: (502) 212-5000 • Fax: 502) 212-5055
www.stantec.com

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

June 1, 2010

Stantec Consulting Services, Inc.
1901 Nelson Miller Parkway
Louisville, KY 40223-2177
Tel: (502) 212-5000
Fax: (502) 212-5055

June 1, 2010 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
Kingston Fossil Plant
Harriman, Tennessee

Dear Mr. Turnbow:

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

- Ash Pond
- Coal Yard Drainage Pond
- Dike 2 Sediment Pond
- Dredge Cell
- Engineered Redwater Wetlands
- Peninsula Gypsum Pond
- Sluice Trench
- Stilling Pond

The field work was performed from April 21 through 30, 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 are 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 continued difficulties establishing new vegetation persist, then TVA should consider refining existing procedures or developing site specific specifications which address topsoil, fertilizing, seed mixtures, etc.

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- It is recommended that TVA catalog, 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) or Emergency Action Plan for KIF. Stantec understands that TVA plans to update the current O&M manual to include dam safety-related items in association with the transition from the initial December 2008 Dredge Cell incident recovery to re-initiation of plant operations. Current recovery operations include a site-wide Emergency Action Plan. In addition, Stantec is in the process of reviewing plant operations and developing an O&M Manual for Kingston ash operations.

- 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.

Mark L. Jones, EIT
Project Engineer

Thomas W. Ward, EIT
Project Engineer

Don W. Fuller II, PE
Principal

/cdm

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Enclosure A

Ash Pond

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TVA 2010 Annual Inspection Program
Kingston Fossil Plant (KIF)
Ash Pond

1. General Facility Information

Facility Status:	Active	NID Identification:	TN14504
Surface Area (inside dikes):	79.92 Acres	TVA Hazard Classification:	Dike C: Significant
Maximum Height (toe to top of dike):	28 ft	Dike Length:	Dike D: 2,641 ft Dike C: 2,813 ft Divider Dike: 2,163 ft
Plant Discharge to Facility:	Unknown	Current Pool Elevation:	Approx. 761 ft

2. Site Visit Information

Stantec Inspection Team:	Thomas Ward EIT, Mark Jones EIT, and Leo Booth
TVA Staff Present:	None present.
Field Inspection Date:	04/30/2010
Weather/Site Conditions:	Sunny, 70s

3. History/Current and Future Operations

History:	This structure is used as the main Ash Pond and receives discharge from the fossil plant, Dredge Cell, Engineered Redwater Wetlands, and river dredging operations through the Sluice Trench. Construction of the Lateral Expansion in the northern extent of the Ash Pond is ongoing.
Current Operations:	Fly ash and bottom ash is sluiced into the Ash Pond. Sluice water flows from the active Ash Pond into the Stilling Pond through the five spillways constructed in 2005.
Future Planned Operational Changes:	The ash pond will be closed as part of the overall Dredge Cell closure.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

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4.1. Interior Slopes

Vegetation:	Grass-covered dike slopes with generally good coverage. Occasional sparse areas of vegetation and/or settled ash with sparse cattails.
Trees:	Occasional small trees were observed along the toe of the Divider Dike and Dike C.
Wave Wash Protection:	Riprap was observed along north slope in from of riser weirs. Area of concrete observed at the Dike C/Divider Dike junction.
Erosion:	None observed.
Instabilities:	None observed.
Animal Burrows:	None observed, although several rabbits were noted during inspection.
Freeboard:	Approximately 5 feet.
Encroachments:	None observed.
Slope:	Varies from approximately 3.66H:1V to 3.12H:1V along the Ash Pond side of the Divider Dike. The interior slopes of Dike C along the Ash Pond are approximately 1.85H:1V and flatten out to 4.85H:1V near the current pool elevation.

4.2. Crest

Crest Cover and Slope:	Gravel (crushed stone) access road, adequate coverage. Relatively flat.
Erosion:	Some rutting on access road.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	Some rutting on access road.
Width:	Approximately 15 to 25 feet.

4.3. Exterior Slopes

Vegetation:	Grass-covered dike slopes with generally good coverage.

Trees:	Numerous trees were observed along the shoreline of Dike C with the Intake Channel (Photo 5).
Erosion:	Some minor erosion on lower bench observed.
Instabilities:	None observed.
Uniform Appearance:	Yes.
Seepage:	Potential seepage was observed along the portion of Dike C where the Intake Channel is directly downstream of the Ash Pond (Photo 3).
Benches:	One bench with a width of approximately 15 feet.
Foundation Drains, and Seepage Collection Systems:	Five 12-inch pipes installed at various locations below lower bench to intersect seeps from Ash Pond and Stilling Pond.
Instrumentation:	Eight piezometers and seven slope inclinometers have been installed along the sections of Dike C that dam the Ash Pond.
Animal Burrows:	None Observed.
Slope:	Varies from approximately 1.57H:1V to 2.89H:1V on the Stilling Pond side of the Divider Dike. Dike C varies from approximately 1.85H:1V near the crest to 3.02H:1V near the river waterline. Buttressing for Dike C being constructed at 6H:1V.
Height:	Approximately 23 ft.

4.4. Spillway Weirs/Riser Inlets

Number:	5
Size, Type and Material:	48-inch diameter RCP push-together risers with standard TVA steel skimmers.
Height of Riser Inlets:	Estimated at 8.5 feet.
Access:	Spillways -001 and -003 accessible via fixed metal catwalks from Dike C.
Joints:	No joints were observable. Reportedly sealed annually by divers.

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Mis-Alignment:	None reported or observed.
Closed/Abandoned Conduits:	There is reportedly one abandoned riser (-035) and one inactive riser (-036) in the Ash Pond.

4.5. Outlet Pipes

Number:	5
Size, Type and Material:	36-inch diameter steel pipes.
Headwall:	N/A.
Joint Separations:	Unknown, unable to observe (outlet pipes submerged).
Mis-Alignment:	None reported or observed (outlet pipes submerged).
Closed/Abandoned Conduits:	Kennedy Weir reportedly abandoned and removed.

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

The following activities and/or improvements have been performed within the Ash Pond since the last annual inspection:

- Ongoing construction of the Lateral Expansion.

6. Recommendations

The following maintenance recommendations are offered for the Ash Pond. Priority codes are included in parentheses and are described in Enclosure K:

- Trees and brush are located on the interior and exterior slopes. It is recommended that trees and brush be removed from these slopes in accordance with the general guidelines shown in Enclosure I. (Priority 4)
- It is recommended that TVA continue to monitor the potential seepage area (Photo 3). The frequency of monitoring should be in accordance with TVA's current inspection program.
- It is recommended that dike rutting (Photo 6) and Dike C toe undercutting (Photo 1) be repaired by placing and compacting material to restore the areas. (Priority 4)

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TVA 2010 Annual Inspection Program
Kingston Fossil Plant (KIF)
Ash Pond
Photos



Photo 1

Typical view of lower bench on downstream side of Dike C looking east. Some undercutting at toe of riser dike observed.



Photo 2

Outlet pipes from Engineered Redwater Wetlands to Ash Pond through Dike C with overgrown vegetation.



Photo 3

Possible Seepage observed on exterior side of Dike C toward Intake Channel amidst vegetation looking southeast.



Photo 4

Trees and vegetation on Dike C alongside Intake Channel looking southeast.



Photo 5

Spillways in Ash Pond looking west.



Photo 6

Typical rutting on crest of Dike C looking northeast.

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Photo 7

Ongoing buttressing of Dike C looking northeast.



Photo 8

Cenospheres and ash deposits in south end of Ash Pond looking west from Divider Dike.



Photo 9

View looking south of settled ash with sparse vegetation in foreground near intersection with Dike D. Ongoing work for Lateral Expansion in background.

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Sheet 1 of 1

PROJECT NO.	175550002
DATE	MAY 2010
DRAWN BY	TMM
CHECKED BY	TWW
CHECKED BY	MJJ
SCALE	1"=500'
REVISED	

Plan View - KIF 2010 Annual Inspection
Ash Pond

Tennessee Valley Authority
Kingston Fossil Plant
Kingston, Roane County, Tennessee

Stantec

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Enclosure B

Coal Yard Drainage Pond

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TVA 2010 Annual Inspection Program
Kingston Fossil Plant (KIF)
Coal Yard Drainage Pond

1. General Facility Information

Facility Status:	Active	NID Identification:	Unknown
Surface Area (inside dikes):	0.408 acres	TVA Hazard Classification:	Not Classified
Maximum Height (toe to top of dike):	8 feet deep	Dike Length:	N/A
Plant Discharge to Facility:	None	Current Pool Elevation:	750 feet

2. Site Visit Information

Stantec Inspection Team:	Thomas Ward EIT, Mark Jones EIT, and Don Fuller PE
TVA Staff Present:	N/A
Field Inspection Date:	April 21, 2010
Weather/Site Conditions:	Sunny, 67 degrees Fahrenheit

3. History/Current and Future Operations

History:	Basin excavated below grade to capture local stormwater runoff from the coal yard and areas immediately south.
Current Operations:	Local stormwater and drainage basin for runoff from the coal yard and areas immediately south. Water is discharged from the pond by being pumped into the fly ash discharge ditch and flows to the active ash disposal area.
Future Planned Operational Changes:	None reported.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Mixture of grasses and shrubs, generally good coverage.
Trees:	Numerous small trees observed on south and west

	slopes (Photos 10 and 11).
Wave Wash Protection:	N/A
Erosion:	Severe localized erosion at inlet pipe on north embankment of pond. Soil eroded and undercut area approximately 10 feet by 3 feet (Photo 14).
Instabilities:	Slough several feet across observed immediately west of inlet erosion on pond embankment (Photo 15).
Animal Burrows:	None observed.
Freeboard:	Approximately 6 feet (estimated).
Encroachments:	None observed.
Slope:	Estimated at approximately 2H:1V.

4.2. Crest

Crest Cover and Slope:	N/A
Erosion:	N/A
Alignment:	N/A
Settlement/Cracking:	N/A
Bare Spots/Rutting:	N/A
Width:	N/A

4.3. Exterior Slopes

Vegetation:	N/A
Trees:	N/A
Erosion:	N/A
Instabilities:	N/A
Uniform Appearance:	N/A
Seepage:	N/A
Benches:	N/A
Foundation Drains, and	N/A

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Seepage Collection Systems:	
Instrumentation:	N/A
Animal Burrows:	N/A
Slope:	N/A
Height:	N/A

4.4. Spillway Weirs/Riser Inlets

Number:	N/A
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:	N/A

4.5. Outlet Pipes

Number:	3
Size, Type and Material:	Two 14-inch corrugated steel stormwater pipes. One 12-inch steel drainage pipe.
Headwall:	N/A
Joint Separations:	Unable to observe.
Mis-Alignment:	Unable to observe.
Closed/Abandoned Conduits:	Unknown.

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

The following activities and/or improvements have been performed within the Coal Yard Drainage Pond since the last annual inspection:

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- The pond was cleaned out by TVA Civil Projects in the fall of 2009.

6. Recommendations

The following maintenance recommendations are offered for the Coal Yard Drainage Pond. Priority codes are included in parentheses and are described in Enclosure K:

- It is recommended that the large undermined area with adjacent slough be repaired (Photos 14 and 15). These and other eroded areas should be cleaned of loose/sloughed material, re-filled, and re-seeded. Filling procedures should include cutting foundation benches (as necessary), placing clay in maximum 6-inch lifts, and compacting with small compaction equipment such as a jumping jack or a hand-held mechanical tamper. An erosion control blanket should be placed after seeding. (Priority 4)
- It is recommended that the 2 rock check dams located in the drainage basin be cleaned and repaired as needed.
- It is recommended that TVA personnel monitor the storage capacity and pool level of the basin and perform dredging and pumping as needed to maintain sufficient capacity. (Priority 5)
- Trees and brush are located on the interior and exterior slopes. It is recommended that trees and brush be removed from these slopes in accordance with the general guidelines shown in Enclosure I. (Priority 4)

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Photo 10

View of rock check dams from floating pump structure looking east. Trees and storm water outlet visible on south slope in upper right of photo.



Photo 11

Pump structure with attached walkway, looking west. South slope visible on left of photo showing overgrown vegetation.



Photo 12

Settled material immediately east/northeast of coal yard drainage basin, looking southeast.

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Photo 13

Settled material immediately east of coal yard drainage basin, looking southwest.



Photo 14

Erosion undercutting inlet pipe northward from drainage pond.

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Photo 15

Sloughing on the north slope near eroded inlet as shown in Photo 5.

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N

Coal Yard
Drainage Pond

10
11
14
15
13
12

1 Photo Number and Direction

0 200 400 Feet

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Plan View - KIF 2010 Annual Inspection
Coal Yard Drainage Pond

Tennessee Valley Authority
Kingston Fossil Plant
Kingston, Roane County, Tennessee

PROJECT NO.	175550002
DATE	MAY 2010
DRAWN BY	TMM
CHECKED BY	TWW
CHECKED BY	MLJ
SCALE	1"=200'
REVISED	
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Sheet

1 of 1

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Enclosure C

Dike 2 Settlement Pond

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TVA 2010 Annual Inspection Program
Kingston Fossil Plant (KIF)
Dike 2 Sediment Pond

1. General Facility Information

Facility Status:	Active	NID Identification:	Unknown
Surface Area (inside dikes):	3.86 acres	TVA Hazard Classification:	Not Classified
Maximum Height (toe to top of dike):	12 feet	Dike Length:	2,090 feet
Plant Discharge to Facility:	Unknown	Current Pool Elevation:	742 feet

2. Site Visit Information

Stantec Inspection Team:	Thomas Ward EIT, Mark Jones EIT, and Leo Booth
TVA Staff Present:	N/A
Field Inspection Date:	April 30, 2010
Weather/Site Conditions:	Partly cloudy with a high of 84 degrees Fahrenheit

3. History/Current and Future Operations

History:	Temporary structure built in 2009 as part of the incident recovery.
Current Operations:	Final series of sediment control ponds for water discharging from the Dredge Cell and embayment areas to the Emory River. Structure capacity maintained as part of recovery operations.
Future Planned Operational Changes:	Dike 2 structure will be removed in association with embayment restoration following completion of the incident ash removal operations.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	None observed.

Trees:	None observed.
Wave Wash Protection:	Shot rock observed on outslopes of temporary dike.
Erosion:	Some minor erosion features of co-mingled ash on exterior side of dike.
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	Approximately 4 feet on the west interior slope and 8 feet on the east interior slope.
Encroachments:	None Observed.
Slope:	Varies from approximately 2.24H:1V to 2.96H:1V.

4.2. Crest

Crest Cover and Slope:	Crushed stone co-mingled with ash, sloped towards pond.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	No issues observed.
Bare Spots/Rutting:	Some minimal rutting observed on roadway surface.
Width:	Approximately 30 feet.

4.3. Exterior Slopes

Vegetation:	None observed.
Trees:	None observed.
Erosion:	Minor surficial erosion features in exposed ash.
Instabilities:	None observed.
Uniform Appearance:	Yes.
Seepage:	None observed.
Benches:	N/A
Foundation Drains, and	None reported or observed.

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Seepage Collection Systems:	
Instrumentation:	N/A
Animal Burrows:	None observed.
Slope:	Approximately 2:1 on the river side.
Height:	5 to 12 feet

4.4. Spillway Weirs/Riser Inlets

Number:	5 between three holding ponds.
Size, Type and Material:	First to second pond: Two 36-inch corrugated metal pipe (CMP) inlets. Second to third pond: Two 48-inch CMP inlets with 36-inch CMP pipes. One additional 12-inch corrugated plastic pipe convey discharge from pump station.
Height of Riser Inlets:	Unknown.
Access:	Access by boat or amphibious craft only.
Joints:	None observable.
Mis-Alignment:	None observable.
Closed/Abandoned Conduits:	None reported or observed.

4.5. Outlet Pipes

Number:	5
Size, Type and Material:	36-inch diameter reinforced concrete pipes from third pond to river.
Headwall:	N/A
Joint Separations:	None observable.
Mis-Alignment:	None observable.
Closed/Abandoned Conduits:	None reported or observed.

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

Dike 2 is a temporary structure that was not in place during previous inspections.

6. Recommendations

The following maintenance recommendations are offered for the Dike 2 Sediment Pond. Priority codes are included in parentheses and are described in Enclosure K:

- It is recommended that TVA continue to periodically clean out holding ponds by mechanical dredging. Perform as needed.
- It is recommended that TVA monitor crests and slopes of the temporary basin and regrade rutting and erosion as may be needed. Stantec observed some minor rutting/erosion (Photo 19) beginning to form. (Priority 3)

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Photo 16

View looking north/northeast at Dike 2 sediment basin.



Photo 17

View of south end of sediment basin looking west, with floating pump attached to guy wire located in pond.



Photo 18

Shot rock on downstream (river side) slope. Emory River in background.

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Photo 19

Minor surficial erosion located on river side of Dike 2 in exposed ash.



Photo 20

View looking north at extent of exterior Dike 2 slope.

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Photo 21

View looking south across sediment basin with 2 spillway inlets visible in center of photo.

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Sheet 1 of 1

PROJECT NO.	175550002
DATE	MAY 2010
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SCALE	1"=300'
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Plan View - KIF 2010 Annual Inspection
Dike 2 Sediment Pond

Tennessee Valley Authority
Kingston Fossil Plant
Kingston, Roane County, Tennessee

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Enclosure D

Dredge Cell

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TVA 2010 Annual Inspection Program
Kingston Fossil Plant (KIF)
Dredge Cell

1. General Facility Information

Facility Status:	Inactive		
Surface Area:	17.93 acres	Maximum Height (toe to top of stack):	51 feet

2. Site Visit Information

Stantec Inspection Team:	Thomas Ward EIT, Mark Jones EIT, and Mike Crouch
TVA Staff Present:	N/A
Field Inspection Date:	April 28, 2010
Weather/Site Conditions:	Cloudy with a high of 67 degrees Fahrenheit and wet ground conditions

3. History/Current and Future Operations

History:	Available information indicates the December 22, 2008 Dredge Cell incident initiated at the northwest corner of the cell, progressing within the Dredge Cell through Cells 2 and 3. The remaining intact portion of the Dredge Cell, comprised of what was formerly referred to as "Cell 1," is now referred to as the Relic.
Current Operations:	Production ash stacking operations have been suspended.
Future Planned Operational Changes:	The current plan indicates the Relic will be reduced in height. The maximum elevation will be EL 790 feet at the time of closure as per the current closure option.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Exterior Slopes and Benches

Vegetation:	Grass-covered, adequate coverage in most cases. Sparse coverage exists in several areas (Photos 29, 32, and 34).
Trees:	Few small trees observed at base of Dike D to the east of the southeast corner of the Dredge Cell just north of the Rim Ditch and Sluice Trench (Photo 38).
Erosion:	Erosion features on the dikes were observed, typically limited

	to minor washouts (Photo 27). Some minor erosion of ash from the test trench was also observed extending east and west on the middle bench (Photos 34 and 35). A clogged drain pipe will was observed (Photo 26). A localized erosion feature in an exposed area of ash was observed on the southwest corner (Photo 28).
Instabilities:	None observed.
Uniform Appearance	Yes.
Benches:	A total of five benches on the south side of the Relic varied on approximate height intervals between 6 to 15 feet and were approximately 20 feet wide, sloped toward the stack. Two benches were present on the southern half Dike D, with the top bench tapering off to the north.
Slope:	Dike C on the south side of the Relic (Cell 1) exhibited slopes ranging between approximately 3.25H:1V and 4.79H:1V. Dike D on the east side of the former Dredge Cell exhibited slopes between approximately 2.77H:11 and 3.08V:1V.
Height:	Approximately 51 feet.
Other:	None.

4.2. Perimeter Drainage Ditches and Down-Drains

Vegetation:	Grass-covered with occasional areas of cattails.
Rip-Rap Channel Lining:	None observed.
Erosion:	None observed.
Sedimentation in Ditches:	Some minor sedimentation observed.
Standing Water in Ditches or on Benches:	Observed at multiple locations on south side of Dredge Cell along Dike C (Photo 30) and on the east side of the Dredge Cell along Dike D (Photo 37).
Silted/Impeded Drainage Pipes:	Occasionally observed (Photo 26).
Other:	None.

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

The following maintenance activities and improvements have been performed for the

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Dredge Cell since the last annual inspection:

- Construction of the Test Embankment on the north side of the Relic was completed.
- Filling operations for ash storage in the Valley area on the east side of the Relic have been conducted.

6. Recommendations

The following maintenance recommendations are offered for the Dredge Cell. Priority codes are included in parentheses and are described in Enclosure K:

- Numerous areas of erosion and/or rutting with standing water were observed during the inspection include areas along both Dike C and Dike D (Photos 27, 29, 30, 37, and 38). It is recommended that these areas be drained and/or subsequently filled/graded and in accordance with current site protocols for erosion repair and temporary vegetative cover. Filling should consist of placing select ash or clay and compacting with small compaction equipment, such as a jumping jack, or hand-held tampers. Some excavation may be required to create foundation benches to allow compaction to occur on level ground. Erosion control blankets should be used for the areas on sloping ground. (Priority 4)
- A few areas lacking adequate vegetation coverage were noted in this inspection along Dikes C and D (Photos 25, 27, 29, and 32). Re-seeding of these areas should be performed to re-establish vegetation growth. Erosion control blankets should be used since the areas are on sloping ground. (Priority 4)
- Some small trees and brush are located on the exterior slopes. It is recommended that trees and brush be removed from these slopes in accordance with the general guidelines shown in Enclosure I. (Priority 4)
- It is recommended that the erosion feature encountered in exposed ash (Photo 28) be excavated and re-filled with suitable ash materials in accordance with general site erosion control protocols. (Priority 4)
- It is recommended that the clogged drainage pipe (Photo 26) outlet be cleared and the pipe be evaluated to determine its current working condition and promote positive drainage. (Priority 4)

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Photo 22

Shot rock haul road constructed on southwest corner of Dike C. Ponding water observed.



Photo 23

Shot rock built- up section of haul road at southwest corner of Dredge Cell, looking south.



Photo 24

View of vegetation on initial (lowest) dike of southern edge of Dredge Cell, looking east.

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Photo 25

Typical Dike C bench, looking east.



Photo 26

Clogged drainage pipe on south side of Dredge Cell Dike C.



Photo 27

Erosion washout on south face of Dredge Cell, second bench, with areas of sparse vegetation on lower benches visible in background.

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Photo 28

Ash erosion feature encountered at southwest corner of Dike C.



Photo 29

Minor erosion with sparse vegetation on south side of Dredge Cell.



Photo 30

Typical ponding and rutting for Dike C on south side of Dredge Cell, looking east.

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Photo 31

Typical view of Dike D, looking north from southeast corner of Dredge Cell.



Photo 32

Minor erosion with additional vegetative cover required along Dike D.



Photo 33

East face of Dike D looking east into Ash Pond. Rock-lined drainage feature observed midway down slope.

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Photo 34

View looking west on second bench of ash deposits on Dike C from erosion of test trench (see Photo 15).



Photo 35

View looking east on second bench of ash deposits on Dike C from erosion of test trench (see Photo 36).



Photo 36

Ash erosion at top of Dike C in the test trench.

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Photo 37

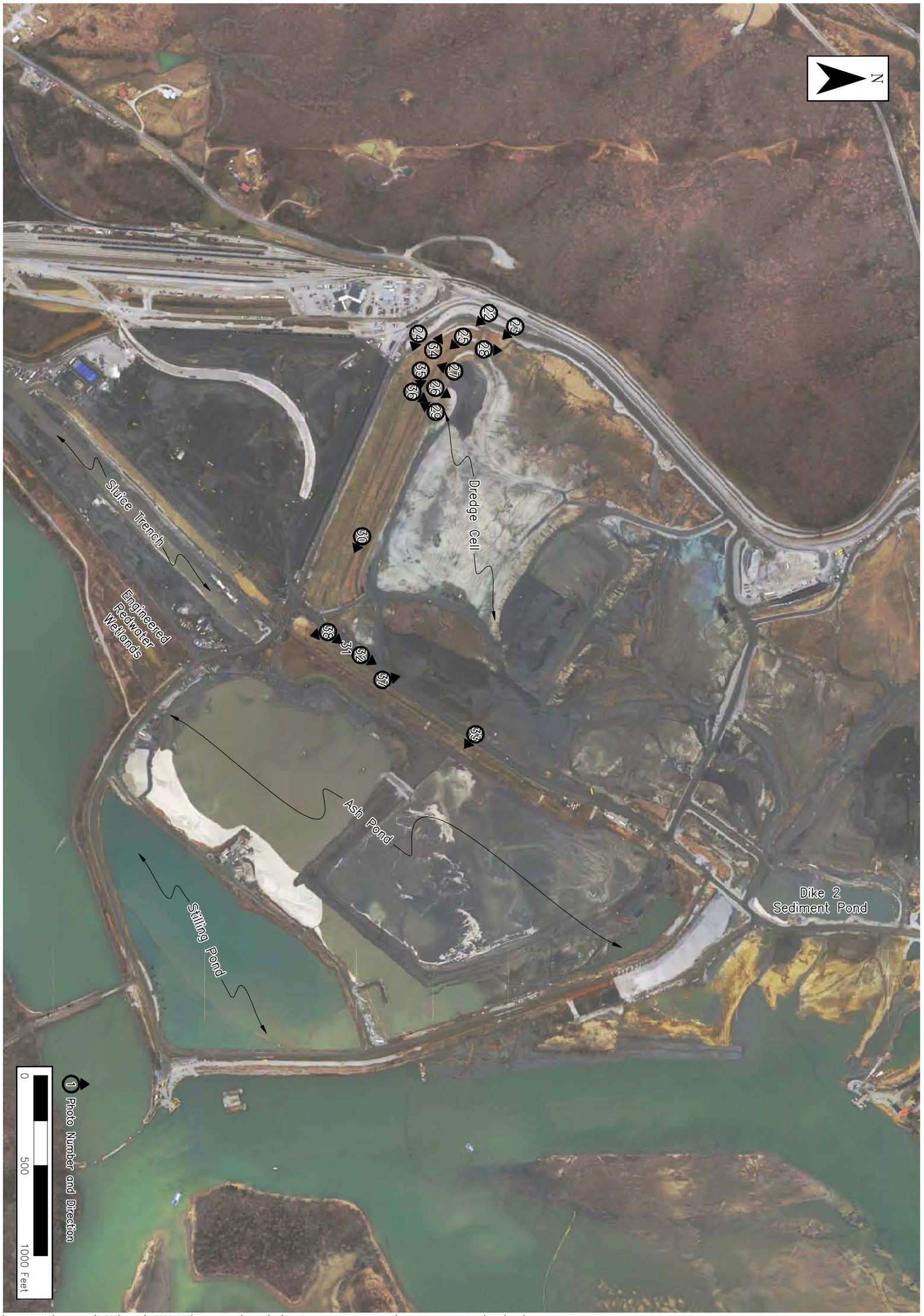
Dredge lines, rutting, and ponding on middle bench of Dike D, looking northwest.



Photo 38

Southeast corner of Dredge Cell just north of Sluice Trench/Rim Ditch.

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Sheet	1 of 1
PROJECT NO.	175550002
DATE	MAY 2010
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SCALE	1"=500'
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Plan View - KIF 2010
Annual Inspection - Dredge Cell

Tennessee Valley Authority
Kingston Fossil Plant
Kingston, Roane County, Tennessee

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Enclosure E

Engineered Redwater Wetlands

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TVA 2010 Annual Inspection Program
Kingston Fossil Plant (KIF)
Engineered Redwater Wetlands

1. General Facility Information

Facility Status:	Active	NID Identification:	Unknown
Surface Area (inside dikes):	3.31 acres	TVA Hazard Classification:	Not Classified
Maximum Height (toe to top of dike):	12 feet	Dike Length:	Approx. 900 feet
Plant Discharge to Facility:	Unknown	Current Pool Elevation:	749 feet

2. Site Visit Information

Stantec Inspection Team:	Thomas Ward EIT and Mark Jones EIT
TVA Staff Present:	Robert Snow (Jacobs)
Field Inspection Date:	April 21, 2010
Weather/Site Conditions:	Sunny with a high of 67 degrees Fahrenheit

3. History/Current and Future Operations

History:	The Engineered Wetlands between Dike C and the Intake Channel collect seepage from the limestone drain below the bottom ash trench (currently designated as the Sluice Trench). The wetlands aid in naturally removing iron and raising the pH of the water before it is pumped to the Ash Pond via two pumps.
Current Operations:	Numerous seeps of redwater through the east dike toward the Intake Channel were observed in March of 2010 in an area south of the Redwater Wetlands. Borings were drilled by Mactec with piezometers installed to monitor the situation. Ongoing stability and seepage analyses are being conducted by Geosyntec. Preliminary results suggest the dike is in no imminent danger of a significant breach or failure.
Future Planned Operational Changes:	Pending continued monitoring and analyses of observed seepage.

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4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Heavy brush, grass, and cattails within wetlands.
Trees:	Few relatively large trees observed on southwest end of interior slope of dike.
Wave Wash Protection:	N/A
Erosion:	Not observed.
Instabilities:	Not observed.
Animal Burrows:	Not observed.
Freeboard:	Approximately 1 to 2 feet for majority of actual Wetlands and 3 to 4 feet at collection pond.
Encroachments:	N/A
Slope:	Approximately 2H:1V (estimated).

4.2. Crest

Crest Cover and Slope:	The crest of the slope is overlain with crushed stone and is relatively flat.
Erosion:	Not observed.
Alignment:	No issues observed.
Settlement/Cracking:	Not observed.
Bare Spots/Rutting:	Some minor rutting and ponding is present.
Width:	Approximately 12 to 14 feet.

4.3. Exterior Slopes

Vegetation:	Heavy brush, grass, and tree-covered exterior dike slopes (Photo 41). Some areas lack coverage.
Trees:	Numerous trees were observed along the Intake

	Channel.
Erosion:	No significant erosion features were observed.
Instabilities:	None observed.
Uniform Appearance:	Yes.
Seepage:	Seepage observed at multiple locations near the waterline at the southern portions of the wetland. Monitoring of the seeps and ongoing analysis being conducted (Photo 45).
Benches:	None observed.
Foundation Drains, and Seepage Collection Systems:	None reported or observed.
Instrumentation:	Six piezometers were installed by Mactec in April of 2010 along two cross sections of the east dike to monitor the current seepage conditions.
Animal Burrows:	None observed; however, rabbits were observed, indicating burrows could be likely obscured within thick vegetation.
Slope:	Approximately 2.2H:1V to 1.5H:1V.
Height:	Approximately 15 feet (estimated).

4.4. Spillway Weirs/Riser Inlets

Number:	2
Size, Type and Material:	24-inch steel pipe morning glory spillways (-018 and -019).
Height of Riser Inlets:	Unknown.
Access:	Catwalks from edge of pond.
Joints:	Unable to observe.
Mis-Alignment:	Unable to observe.
Closed/Abandoned Conduits:	One 12-inch steel pipe (-017) northwest of the spillways is reportedly inactive.

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4.5. Outlet Pipes

Number:	2
Size, Type and Material:	12-inch plastic pipes with steel outlets in Ash Pond.
Headwall:	N/A
Joint Separations:	Unknown, unable to observe.
Mis-Alignment:	None observed.
Closed/Abandoned Conduits:	One 12-inch steel pipe (-017) northwest of the spillways is reportedly inactive.

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

The following activities and/or improvements have been performed within the Engineered Redwater Wetlands since the last annual inspection:

- None reported.

6. Recommendations

The following maintenance recommendations are offered for the Engineered Redwater Wetlands:

- Trees and brush are located on the interior and exterior slopes. It is recommended that trees and brush be removed from these slopes in accordance with the general guidelines shown in Enclosure I. (Priority 4)
- It is recommended that floating debris/vegetation observed within the discharge pump structures be removed so the pumps do not become clogged or damaged. (Priority 3)
- It is recommended that the seepage observed along the Intake Channel waterline continue to be monitored in the interim until mitigation designs are implemented in association with Ash Pond closure and plant conversion from wet to dry ash handling.

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Photo 39

Redwater drainage ditch along west side of access roadway immediately west of Intake Channel, looking north.



Photo 40

View of typical access road corridor along east side Engineered Redwater Wetlands, looking north.



Photo 41

Thick vegetation and trees between access road and Intake Channel.

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Photo 42

Discharge pump alongside shoreline of Intake Channel.



Photo 43

Construction debris located on west side of access road with thick vegetation in the Redwater Wetlands.



Photo 44

Thick vegetation and trees on the slopes of the Intake Channel with sparse cover near waterline, looking north/northeast from the access road.

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Photo 45

Typical seepage observed along Intake Channel.



Photo 46

Various debris floating near and within the two active morning glory spillways for the discharge pumps in collection pond.

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Sheet 1 of 1

PROJECT NO.	175550002
DATE	MAY 2010
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Plan View - KIF 2010 Annual Inspection
Engineered Redwater Wetlands

Tennessee Valley Authority
Kingston Fossil Plant
Kingston, Roane County, Tennessee

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Enclosure F

Peninsula Gypsum Pond

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TVA 2010 Annual Inspection Program
Kingston Fossil Plant (KIF)
Peninsula Gypsum Pond

1. General Facility Information

Facility Status:	Inactive	NID Identification:	Unknown
Surface Area (inside dikes):	51.2 acres (TVA Drawing 10W427-3)	TVA Hazard Classification:	Not Classified
Maximum Height (toe to top of dike):	28 feet	Dike Length:	5,904 feet
Plant Discharge to Facility:	Unknown	Current Pool Elevation:	Unknown

2. Site Visit Information

Stantec Inspection Team:	Thomas Ward EIT, Mark Jones EIT, and Leo Booth
TVA Staff Present:	N/A
Field Inspection Date:	April 30, 2010
Weather/Site Conditions:	Partly cloudy with a high of 84 degrees Fahrenheit

3. History/Current and Future Operations

History:	Excavation and construction began in 2008 in order to create a new facility to collect gypsum produced from the new plant scrubbers.
Current Operations:	Unknown.
Future Planned Operational Changes:	Unknown.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Sparse grasses to no vegetative cover (bare soil).
Trees:	None observed.
Wave Wash Protection:	None observed.

Erosion:	Some minor erosion features observed, most notably a washout on the south slope (Photo 47).
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	N/A.
Encroachments:	None observed.
Slope:	Interior slopes varied from approximately 2.73:1 to 3.92:1

4.2. Crest

Crest Cover and Slope:	Crushed stone.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	Varied from approximately 21 to 33 feet.

4.3. Exterior Slopes

Vegetation:	Grass-covered, ranging from well covered to bare soil.
Trees:	Trees were not present on any of the slopes. However, numerous large trees lined the edge of the peninsula beyond the toe of the exterior slopes along the south (Photo 48).
Erosion:	None observed.
Instabilities:	None observed.
Uniform Appearance:	Yes.
Seepage:	None observed.
Benches:	None observed.
Foundation Drains, and Seepage Collection Systems:	None reported or observed.

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Instrumentation:	None observed.
Animal Burrows:	None observed.
Slope:	Exterior constructed fill slopes varied from 2.91H:1V to 2.88H:1V. The cut slope to the north side of the pond was approximately 3.01H:1V.
Height:	Based on Lidar data, the pond appears to have a maximum pool elevation of approximately 768 feet, corresponding to an approximate maximum pool depth of 18 feet. The constructed dike on the south side is approximately 25 feet in height.

4.4. Spillway Weirs/Riser Inlets

Number:	3
Size, Type and Material:	5-foot diameter drop inlets, precast concrete.
Height of Riser Inlets:	5.5 feet.
Access:	N/A
Joints:	None observed.
Mis-Alignment:	None observed.
Closed/Abandoned Conduits:	None observed.

4.5. Outlet Pipes

Number:	5
Size, Type and Material:	Three 48-inch reinforced concrete pipes (RCPs) located at the western Gypsum Pond dike discharge into the storm water pond. Additionally, two 12-inch HDPE underdrain pipes drain into a pump station at the southwestern portion of the Gypsum Pond dike. Two pumps are included in the pump station and pump to the adjacent storm water pond via two 4-inch ductile iron pipes (D.I.P.), combining into one 4-inch D.I.P. Additionally, a 24-inch PVC overflow line to the storm water pond is located within the pump station.
Headwall:	N/A

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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

Ongoing construction since previous inspection.

6. Recommendations

The following maintenance recommendations are offered for the Peninsula Gypsum Pond. Priority codes are included in parentheses and are described in Enclosure K:

- Areas of erosion observed during the inspection include areas along the interior of the Gypsum Pond dike (Photo 47). It is recommended that these be filled/graded and seeded to re-establish vegetation. Filling should consist of placing clay and compacting with small compaction equipment, such as a jumping jack, or hand-held tampers. Some excavation may be required to create foundation benches to allow compaction to occur on level ground. Erosion control blankets should be used for the areas on sloping ground. (Priority 4)
- Many areas lacking adequate vegetation coverage were noted in this inspection along the perimeter of the Gypsum Pond (Photo 50). Re-seeding of these areas should be performed to re-establish vegetation growth. Erosion control blankets should be used since the areas are on sloping ground. (Priority 4)
- It is recommended that the pump outlet system be checked periodically for proper operation. Maintenance should be performed as needed.

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Photo 47

View of erosion feature on south slope looking to the south from north side. Vegetation not present in some areas.



Photo 48

Exterior slope on south side of gypsum pond with crushed stone access road at crest, looking southeast.



Photo 49

Drainage layer under construction on east side of Gypsum pond.

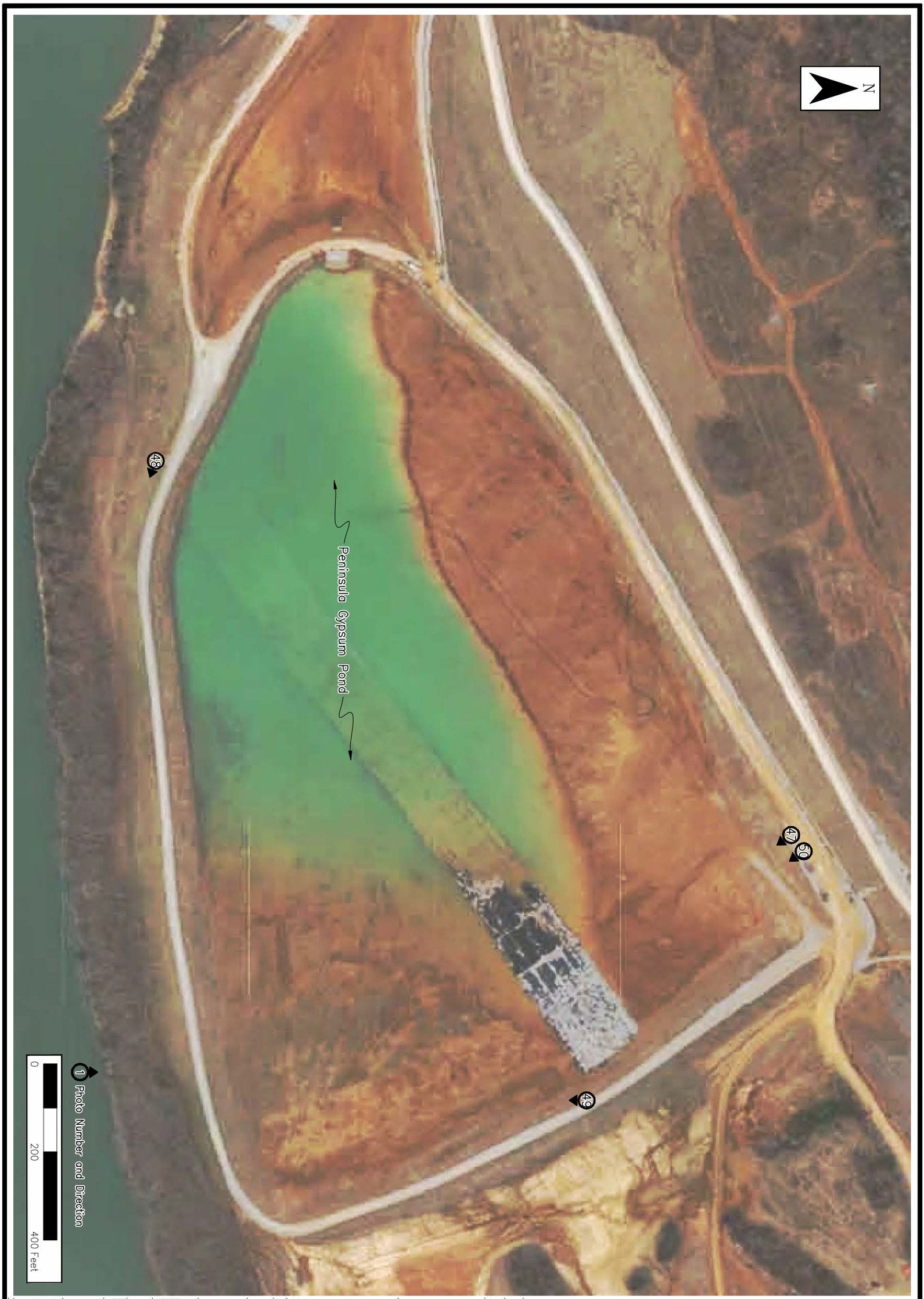
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Photo 50

Typical view of lack of vegetative cover. Several feet of water present in much of pond at time of inspection.

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Sheet 1 of 1

PROJECT NO.	175550002
DATE	MAY 2010
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CHECKED BY	TWW
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SCALE	1"=200'
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Plan View - KIF 2010 Annual Inspection
Peninsula Gypsum Pond

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Kingston Fossil Plant
Kingston, Roane County, Tennessee

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Enclosure G

Sluice Trench

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TVA 2010 Annual Inspection Program
Kingston Fossil Plant (KIF)
Sluice Trench

1. General Facility Information

Facility Status:	Active	NID Identification:	Unknown
Surface Area (inside dikes):	2.31 acres western, 3.87 acres eastern	TVA Hazard Classification:	Not Classified
Maximum Height (toe to top of dike):	19 feet	Dike Length:	N/A
Plant Discharge to Facility:	Unknown	Current Pool Elevation:	764 feet

2. Site Visit Information

Stantec Inspection Team:	Thomas Ward EIT and Mark Jones EIT
TVA Staff Present:	Robert Snow (Jacobs)
Field Inspection Date:	April 30, 2010
Weather/Site Conditions:	Sunny with a high of 67 degrees Fahrenheit

3. History/Current and Future Operations

History:	Previously used as the bottom ash trench with discharge directly from the plant, it has been converted into the current Sluice Trench to accommodate river dredging operations.
Current Operations:	Used as discharge point for dredge lines and initial retrieval point for ash being dredged out of the Emory River. Access near the trench is restricted due to safety concerns associated with the current mechanical dredging operations from the Sluice Trench and presence of large volumes of saturated ash near the Trench.
Future Planned Operational Changes:	Sluice Trench to remain active during dredging.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

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4.1. Interior Slopes

Vegetation:	None observed.
Trees:	None observed.
Wave Wash Protection:	None observed.
Erosion:	None observed.
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	None observed.
Encroachments:	Wet ash being mechanically dredged from Sluice Trench.
Slope:	Not observable; obscured by wet ash being removed from Sluice Trench.

4.2. Crest

Crest Cover and Slope:	Obscured by wet ash being removed from Sluice Trench. Access road constructed immediately east of the Sluice Trench of crushed stone.
Erosion:	None observed.
Alignment:	No issues.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	None observed.
Width:	Not observable.

4.3. Exterior Slopes

Vegetation:	Grass-covered, generally good coverage. Areas approaching the shoreline were sparsely covered at time of inspection due to recent lowering of the existing pool elevation of the reservoir.
Trees:	Numerous trees with some areas of thick underbrush along shoreline on eastern edge of original dike.
Erosion:	Occasional minor erosion features observed near the

	shoreline along the east dike.
Instabilities:	None observed.
Uniform Appearance:	Yes.
Seepage:	Seepage observed at multiple locations between the Sluice Trench and the Engineered Redwater Wetlands (Photos 53 and 54). Monitoring of the seeps and ongoing analysis being conducted by Geosyntec.
Benches:	N/A
Foundation Drains, and Seepage Collection Systems:	None reported or observed.
Instrumentation:	Six piezometers were installed by Mactec in April of 2010 along two cross sections of the east dike to monitor the current seepage conditions.
Animal Burrows:	None observed.
Slope:	Varies from approximately 4.25H:1V to 4.61H:1V. Slopes on the east dike varied from approximately 1.96H:1V to 3.36H:1V, flattening out to 5.23H:1V along the southern portion of the east dike.
Height:	19 feet

4.4. Spillway Weirs/Riser Inlets

Number:	N/A
Size, Type and Material:	N/A
Height of Riser Inlets:	N/A
Access:	Unable to access.
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	N/A

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4.5. Outlet Pipes

Number:	N/A
Size, Type and Material:	N/A
Headwall:	N/A
Joint Separations:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	N/A

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

The following activities and/or improvements have been performed within the Sluice Trench since the last annual inspection:

- Increase in river dredging operations in conjunction with Ballfield operations to facilitate removal of ash from site have resulted in the Sluice Trench being the initial point of mechanical dredging of incoming slurry. A sheet pile wall was also installed on the west side of the trench to aid in this effort.

6. Recommendations

The following maintenance recommendations are offered for the Sluice Trench:

- Trees and brush are located on the interior and exterior slopes. It is recommended that trees and brush be removed from these slopes in accordance with the general guidelines shown in Enclosure I. (Priority 4)
- It is recommended that the seepage observed along the Intake Channel waterline continue to be monitored. Stantec understands that Geosyntec has been tasked with this effort initiate a mitigation plan.
- Although no animal burrows were visible at the time of inspection, several rabbits were observed in the area, indicating burrows were likely obscured by thick vegetation. TVA should periodically monitor the area for animal burrows. If encountered, it is recommended that these burrows be repaired by collapsing the tunnels, cleaning loose soil, and refilling the resulting cavities/depressions with compacted clay. If the burrows are found to be severe and extensive, then filling with flowable fill may be more appropriate. (Priority 4)



Photo 51

Road constructed in corridor between Sluice Trench to the west and Intake Channel to the east. Typical minor surface rutting on east dike visible in foreground.



Photo 52

Typical view of thick vegetation and trees located along the Intake Channel, looking east.



Photo 53

Seepage located along the Intake Channel.

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Photo 54

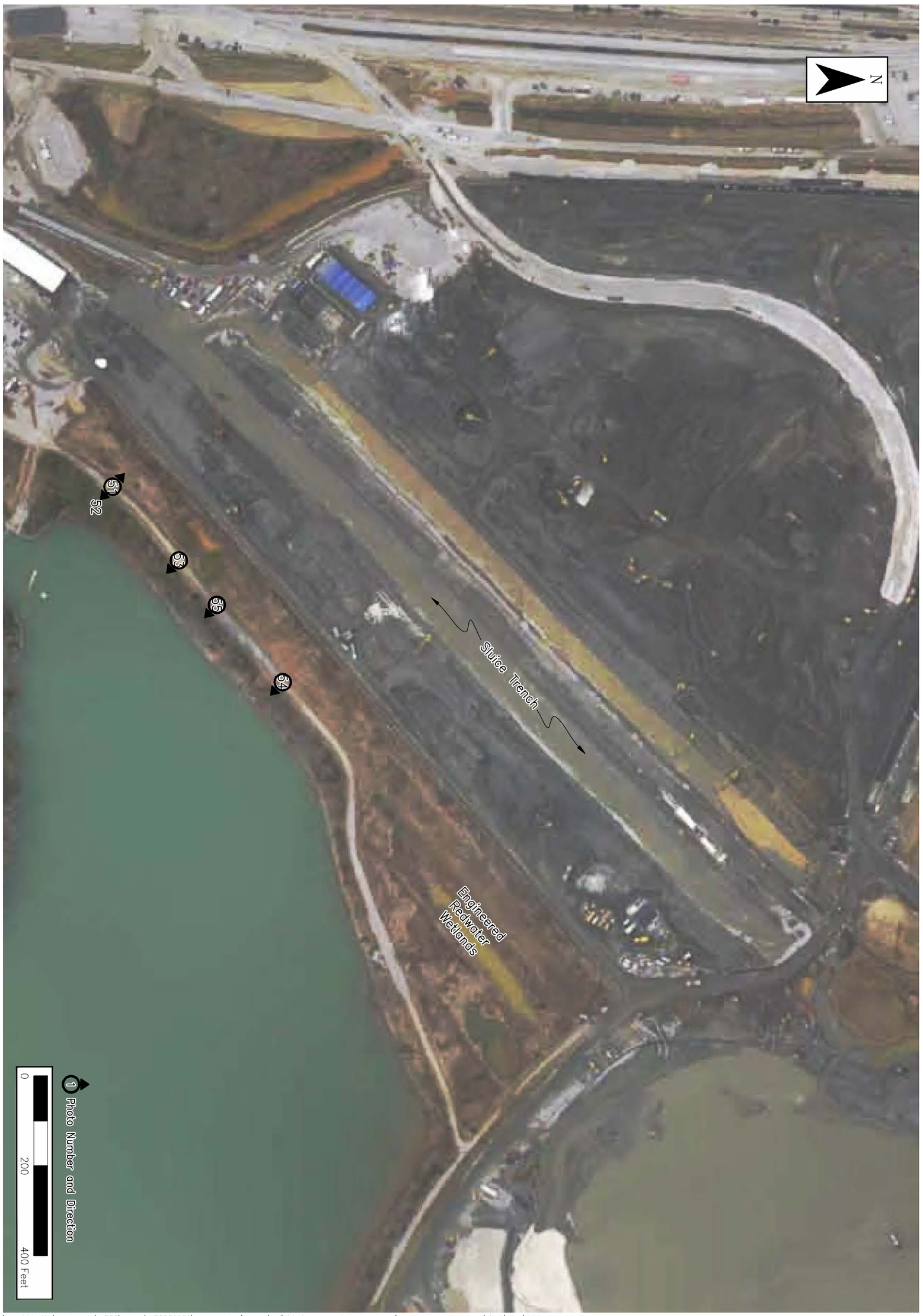
Typical view of redwater seepage observed, looking east into Intake Trench. Note: crushed stone in foreground placed as working platform for drilling operations that had recently been conducted to monitor seepage.



Photo 55

Debris located on the exterior slope, looking east.

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Sheet 1 of 1

PROJECT NO.	175550002
DATE	MAY 2010
DRAWN BY	TMM
CHECKED BY	TWW
CHECKED BY	MJJ
SCALE	1"=200'
REVISED	

Plan View - KIF 2010
Annual Inspection - Sluice Trench

Tennessee Valley Authority
Kingston Fossil Plant
Kingston, Roane County, Tennessee

Stantec

STANTEC CONSULTING SERVICES INC.
1859 Bowles Ave., Ste 250
St. Louis, Missouri
63026-1944
Tel. 636.343.3880
Fax 636.343.3554
www.stantec.com

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Enclosure H

Stilling Pond

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TVA 2010 Annual Inspection Program
Kingston Fossil Plant (KIF)
Stilling Pond

1. General Facility Information

Facility Status:	Active	NID Identification:	TN14504
Surface Area (inside dikes):	24.28 acres	TVA Hazard Classification:	Dike C: Significant
Maximum Height (toe to top of dike):	28 feet	Dike Length:	2,716 ft of Dike C and 2,163 ft of the Divider Dike.
Plant Discharge to Facility:	Unknown	Current Pool Elevation:	755 feet

2. Site Visit Information

Stantec Inspection Team:	Thomas Ward EIT, Mark Jones EIT, and Leo Booth
TVA Staff Present:	N/A
Field Inspection Date:	April 30, 2010
Weather/Site Conditions:	Partly cloudy with a high of 84 degrees Fahrenheit

3. History/Current and Future Operations

History:	Sluice water flows from the main Ash Pond to the Stilling Pond through five active spillways constructed in 2005. Water then discharges into the plant Intake Channel via six riser structure spillways equipped with discharge diffusers constructed in November 2003.
Current Operations:	Continuing normally as mentioned above.
Future Planned Operational Changes:	No changes are planned.

4. Stantec Field Observations

See attached Photos and Site Plan Drawing.

4.1. Interior Slopes

Vegetation:	Grass-covered, generally thick with good coverage.

	Some areas exhibited sparse vegetation near the waterline.
Trees:	Occasional small trees were observed near the water line.
Wave Wash Protection:	None observed.
Erosion:	No significant signs of erosion on interior slopes. However, possible seepage at one location through the Divider Dike into the Stilling Pond (Photo 66) was observed.
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	Approximately 10 to 11 feet.
Encroachments:	None observed.
Slope:	Varies from approximately 2.8H:1V to 1.59H:1V.

4.2. Crest

Crest Cover and Slope:	Crushed stone access road.
Erosion:	None observed.
Alignment:	No issues observed.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	Occasional minor rutting observed with potential for erosion at observed steep cut (Photo 62).
Width:	Approximately 12 to 15 feet.

4.3. Exterior Slopes

Vegetation:	Grass-covered, generally good coverage with some sparse areas observed (Photos 56 and 57).
Trees:	Numerous medium to large trees and brush were observed at the toe of Dike C running alongside the Intake Channel. None observed along remainder of perimeter.
Erosion:	Some significant rutting with standing water and adjacent

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	erosion washouts were observed on the exterior bench of Dike C (Photos 59 and 60).
Instabilities:	None Observed.
Uniform Appearance:	Slope on Divider Dike not uniform - generally steeper towards the southwest portion of the dike.
Seepage:	None observed.
Benches:	One bench, approximately 15 to 20 feet wide.
Foundation Drains, and Seepage Collection Systems:	None reported or observed.
Instrumentation:	Four slope inclinometers and four piezometers are present along the segment of Dike C adjacent to the Stilling Pond.
Animal Burrows:	None observed.
Slope:	Varies from approximately 1.77H:1V to 3.56H:1V.
Height:	28 feet.

4.4. Spillway Weirs/Riser Inlets

Number:	6
Size, Type and Material:	48-inch RCP push-together riser pipes with standard TVA steel skimmers.
Height of Riser Inlets:	Approximately 8.5 feet.
Access:	Catwalk from Dike C (Photo 63).
Joints:	Not observable.
Mis-Alignment:	Not observable.
Closed/Abandoned Conduits:	There is reportedly an unidentified 12-inch diameter pipe connected to weir -010 that penetrates the dike with no evidence of an outlet.

4.5. Outlet Pipes

Number:	6

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Size, Type and Material:	36-inch steel pipes.
Headwall:	No issues observed.
Joint Separations:	Unknown, unable to observe.
Mis-Alignment:	Unknown, unable to observe.
Closed/Abandoned Conduits:	Six 12-inch steel outlet pipes are reportedly in service to drain stormwater from the Stilling Pond to Watts Bar Lake, although only 3 could be located in the January 2010 pipe inventory inspection performed by Stantec. Historical drawings did not indicate any removal.

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:

- Buttressing of Dike C is ongoing and will continue south along the exterior slopes toward the Stilling Pond.

6. Recommendations

The following maintenance recommendations are offered for the Stilling Pond:

- During the inspection, Stantec observed an area of potential seepage on the Stilling Pond side of the Divider Dike (Photo 66). It is therefore recommended that TVA continue to monitor this area to observe changes in appearance in accordance with TVA's current inspection program.
- Trees and brush are located on the interior and exterior slopes. It is recommended that trees and brush be removed from these slopes in accordance with the general guidelines shown in Enclosure I. (Priority 4)
- Areas of extensive rutting and/or erosion on the middle bench of Dike C need to be repaired (Photos 59 and 60). It is recommended that these areas be drained, filled, graded, and subsequently seeded to re-establish vegetation. Filling should consist of placing clay in 6-inch lifts and compacting with small compaction equipment, such as a jumping jack or hand-held tampers. Some excavation may be required to establish benches to facilitate proper compaction on level surfaces. Erosion control blankets should be used on sloping areas. (Priority 4)
- Areas lacking adequate vegetation coverage should be re-seeded to establish proper vegetative cover (Photos 56 and 57). Erosion control blankets should be

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used on sloping areas. (Priority 4)

- Re-grading should be considered at the areas along the crest of Dike C near the catwalk to the spillways exhibiting very steep slopes (Photo 62) to reduce future potential for localized sloughing. (Priority 5)

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Photo 56

Typical view of access roadway on lower bench of Dike C with some minor rutting and exposed soil.



Photo 57

Sparse vegetation on exterior slopes of Dike C.



Photo 58

Boulder observed alongside dredge pipes on exterior slopes of Dike C adjacent to Intake Channel.

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Photo 59

Typical view of more significant rutting and standing water on bench of exterior slope; looking east adjacent to the Intake Channel with numerous trees present at toe of Dike C.



Photo 60

Typical erosion washout on exterior bench of Dike C.



Photo 61

Overflow pipe to discharge into Intake Channel.

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Photo 62

Steep cut approximately 5 feet high on crest of Dike C with exposed clay and minor rutting.



Photo 63

Catwalk access to spillways.



Photo 64

View of spillways looking northeast from Dike C.

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Photo 65

Slope protection at intersection of Dike C and Divider Dike in north corner of Stilling Pond; typical small trees present near waterline.



Photo 66

Possible seepage into Stilling Pond through Divider Dike, looking east.



Photo 67

Ash deposit with sparse vegetation along Divider Dike in Stilling Pond. Dike C in background.

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STANTEC CONSULTING SERVICES INC.
1969 Bowles Ave., Ste 250
St. Louis, Missouri
63026-1944
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Fax 636.343.3884
www.stantec.com

Stantec

Plan View - KIF 2010
Annual Inspection - Stilling Pond

Tennessee Valley Authority
Kingston Fossil Plant
Kingston, Roane County, Tennessee

PROJECT NO.	175550002
DATE	MAY 2010
DRAWN BY	TMM
CHECKED BY	TWW
CHECKED BY	MLJ
SCALE	1"=200'
REVISED	
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Sheet

1 of 1

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Enclosure I

General Guidelines for Tree Removal and Maintenance of Vegetation

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.

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Enclosure J

General Guidelines for Repair of Animal Burrows

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.

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Enclosure K

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

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APPENDIX A

Document 8

NPDES Permit No. TN0005452, TVA – Kingston Fossil Plant, Harriman, Roane County, Tennessee

STATE OF TENNESSEE
DEPARTMENT OF ENVIRONMENT AND CONSERVATION
401 CHURCH STREET
L & C ANNEX 6TH FLOOR
NASHVILLE TN 37243-1534

August 29, 2003

Ms. Lindy Johnson
TVA - Kingston Fossil Plant
1101 Market Street, LP 5D-C
Chattanooga, TN 37402-2801

Subject: NPDES Permit No. TN0005452
TVA - Kingston Fossil Plant
Harriman, Roane County, Tennessee

Dear Ms. Johnson:

In accordance with the provisions of the Tennessee Water Quality Control Act, Tennessee Code Annotated, Sections 69-3-101 through 69-3-120, the enclosed NPDES Permit is hereby issued by the Division of Water Pollution Control. The continuance and/or reissuance of this NPDES Permit is contingent upon your meeting the conditions and requirements as stated therein.

Please be advised that you have the right to appeal any of the provisions established in this NPDES Permit, in accordance with Tennessee Code Annotated, Section 69-3-110, and the General Regulations of the Tennessee Water Quality Control Board. If you elect to appeal, you should file a petition within thirty (30) days of the receipt of this permit.

If you have questions, please contact the Division of Water Pollution Control at your local Environmental Assistance Center at 1-888-891-TDEC; or, at this office, please contact Mr. Vojin Janjic at (615) 532-0670 or by E-mail at *Vojin.Janjic@state.tn.us*.

Sincerely,

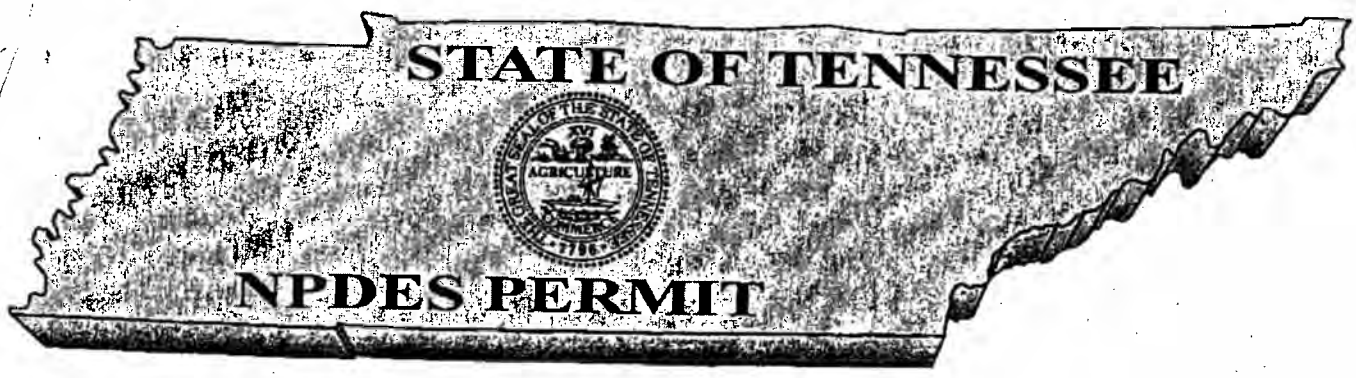
Robert L. Haley for

Saya Ann Qualls, P.E.
Manager, Permit Section
Division of Water Pollution Control

SAQ/VMJ

Enclosure

cc: Division of Water Pollution Control, Permit Section
Division of Water Pollution Control, Environmental Assistance Center - Knoxville
Ms. Connie A. Kagey, Sam Nunn Atlanta Federal Center, 61 Forsyth Street, SW, Atlanta, GA 30303-3104



No. TN0005452

Authorization to discharge under the
National Pollutant Discharge Elimination System (NPDES)

Issued By
Tennessee Department of Environment and Conservation
Division of Water Pollution Control
401 Church Street
6th Floor, L & C Annex
Nashville, Tennessee 37243-1534

Under authority of the Tennessee Water Quality Control Act of 1977 (T.C.A. 69-3-101 et seq.) and the delegation of authority from the United States Environmental Protection Agency under the Federal Water Pollution Control Act, as amended by the Clean Water Act of 1977 (33 U.S.C. 1251, et seq.)

Discharger: **TVA - Kingston Fossil Plant**

is authorized to discharge: **fly ash and bottom ash sluice water, storm water runoff, fire protection flushes and groundwater, coal yard runoff pond discharges including utility building drainage, coal pile and coal conveyor drainage, red water wetlands discharges, precipitator area washdown and roof drains, station sump discharges including boiler leakage, laboratory and analytical process water, boiler blowdown, miscellaneous equipment cooling and lubricating water, floor washing wastes, air conditioning cooling water, ID fan cooling water, ash system leakage and boiler bottom overflow, water treatment plant wastes, ammonia storage runoff, treated chemical and nonchemical metal cleaning wastes from Internal Monitoring Point 005 and nonchemical metal cleaning wastes from Outfall 001; once through condenser cooling water, storm water runoff, groundwater, raw water leakage and fire protection flushes, intake screen backwash, and boiler blowdown, from Outfall 002; storm water runoff, fire protection flushes, raw water leakage and noncontact cooling water from Outfall 006; storm water runoff and abandoned ash pond seepage from Outfall 007; drainage from sluice line trench from Outfall 008**

from a facility located: **In Harriman, Roane County, Tennessee**

to receiving waters named: **Clinch River at mile 2.9**

in accordance with effluent limitations, monitoring requirements and other conditions set forth herein.

This permit shall become effective on: **October 1, 2003**

This permit shall expire on: **August 31, 2008**

Issuance date: **September 1, 2003**

Robert L. Haley for SIG

Paul E. Davis, Director
Division of Water Pollution Control

CN-0759

RDAs 2352 and 2366

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APPENDIX A

Document 9

NPDES Permit No. TN0080870, TVA – Kingston Fossil Plant – Flue Gas Desulfurization System Discharges, Harriman, Roane County, Tennessee

STATE OF TENNESSEE
DEPARTMENT OF ENVIRONMENT AND CONSERVATION

DIVISION OF WATER POLLUTION CONTROL
401 CHURCH STREET
L & C ANNEX 6TH FLOOR
NASHVILLE TN 37243-1534

October 16, 2009

Mr. Leslie Nale
Plant Manager
Tennessee Valley Authority
714 Swan Pond Road KFP 1A-KST
Harriman, TN 37748

Subject: **NPDES Permit No. TN0080870**
TVA - Kingston Fossil Plant (KIF)
Harriman, Roane County, Tennessee

Dear Mr. Nale:

In accordance with the provisions of the Tennessee Water Quality Control Act, Tennessee Code Annotated, Sections 69-3-101 through 69-3-120, the Division of Water Pollution Control hereby issues the enclosed NPDES Permit. The continuance and/or reissuance of this NPDES Permit is contingent upon your meeting the conditions and requirements as stated therein.

Please note the attached permit includes corrections to typographic errors for sampling frequency for TSS and Oil and Grease, and for dike inspections under the TVA Dam Safety Program. Please be advised that you have the right to appeal any of the provisions established in this NPDES Permit, in accordance with Tennessee Code Annotated, Section 69-3-105(i), and the General Regulations of the Tennessee Water Quality Control Board. If you elect to appeal, you should file a petition within thirty (30) days of the receipt of this permit.

If you have questions, please contact the Division of Water Pollution Control at your local Field Office at 1-888-891-TDEC; or, at this office, please contact Mr. Bob Alexander at (615) 532-0659 or by E-mail at *Robert.Alexander@tn.gov*.

Sincerely,

Mr. Vojin Janjić
Manager, Permit Section
Division of Water Pollution Control

Enclosure
cc: DWPC, Permit Section & Knoxville Environmental Field Office
Ms. Karrie-Jo Robinson Shell, Environmental Engineer, EPA Region 4, Sam Nunn Atlanta Federal Center, NPDES Permit Section, 61 Forsyth Street SW, Atlanta, GA 30303
Ms. Lindy Johnson, TVA Environmental Compliance, 1101 Market Street, LP 5D, Chattanooga, TN 37402

STATE OF TENNESSEE

NPDES PERMIT

No. TN0080870

Authorization to discharge under the
National Pollutant Discharge Elimination System (NPDES)

Issued By

Tennessee Department of Environment and Conservation
Division of Water Pollution Control
401 Church Street
6th Floor, L & C Annex
Nashville, Tennessee 37243-1534

Under authority of the Tennessee Water Quality Control Act of 1977 (T.C.A. 69-3-101 et seq.) and the delegation of authority from the United States Environmental Protection Agency under the Federal Water Pollution Control Act, as amended by the Clean Water Act of 1977 (33 U.S.C. 1251, et seq.)

Discharger:	**TVA - Kingston Fossil Plant - Flue Gas Desulfurization System Discharges**
is authorized to discharge:	**Discharges from FGD detention pond into the Kingston Fossil Plant discharge channel via Outfall 01A**
from a facility located:	**in Harriman, Roane County, Tennessee**
to receiving waters named:	**Clinch River mile 2.5**

in accordance with effluent limitations, monitoring requirements and other conditions set forth herein.

This permit shall become effective on: **October 1, 2009**

This permit shall expire on: **September 30, 2013**

Issuance date: **September 22, 2009**

B Tabor FOR
Paul E. Davis, Director
Division of Water Pollution Control

CN-0759

RDAs 2352 and 2366

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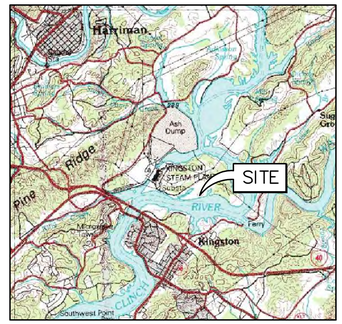
APPENDIX A

Document 10

"Construction Drawings, Phase 1A Area, Gypsum Disposal Facility, Kingston Fossil Plant – Peninsula Site," Drawing Nos. 10WXXX-1 through 10WXXX-16, Dated July, 2011, Prepared by Geosyntec Consultants

DRAWING INDEX

10WXXX–1	COVER SHEET
10WXXX–2	SITE CONDITIONS PRIOR TO THE DECEMBER 2010 DROP–OUT
10WXXX–3	OVER–EXCAVATION AND TEMPORARY STORMWATER DIVERSION BERM PLAN
10WXXX–4	BOTTOM OF SUBGRADE
10WXXX–5	BOTTOM OF SUBGRADE CONSTRUCTION CONTROL POINTS
10WXXX–6	TOP OF SUBGRADE
10WXXX–7	TOP OF SUBGRADE CONSTRUCTION CONTROL POINTS
10WXXX–8	BOTTOM OF 1 FOOT SCREENED CLAY
10WXXX–9	BOTTOM OF 1 FOOT SCREENED CLAY CONSTRUCTION CONTROL POINTS
10WXXX–10	TOP OF 1 FOOT SCREENED CLAY
10WXXX–11	TOP OF 1 FOOT SCREENED CLAY CONSTRUCTION CONTROL POINTS
10WXXX–12	DRAINAGE SYSTEM, DIVIDER DIKE, AND SETTLING POND LAYOUT
10WXXX–13	INTERIM LAYOUT OF THE PHASE 1A CELL
10WXXX–14	FINAL LAYOUT OF THE PHASE 1A CELL
10WXXX–15	DETAILS I
10WXXX–16	DETAILS II



LOCATION MAP



VICINITY MAP

CONSTRUCTION DRAWINGS
PHASE 1A AREA
GYPSUM DISPOSAL FACILITY
KINGSTON FOSSIL PLANT - PENINSULA SITE

DRAFT CONSTRUCTION DRAWINGS FOR REVIEW

Geosyntec consultants
1255 ROBERTS BOULEVARD, N.W., SUITE 200
KENNESAW, GEORGIA 30144 USA
PHONE: 678.202.9500

THIS DRAWING MAY NOT BE ISSUED FOR PROJECT TENDER OR CONSTRUCTION, UNLESS SEALED.

SIGNATURE

DATE

CONSTRUCTION DRAWINGS

RO	JULY 2011	M	RML								
REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT ID	AS CONST	REV ID

DISCIPLINE INTERFACE

SCALE: NONE EXCEPT AS NOTED

COVER SHEET

GYPSUM DISPOSAL FACILITY,
PHASE IA AREA

DESIGNED BY:	DRAWN BY:	CHECKED BY:	SUPERVISED BY:	REVIEWED BY:	APPROVED BY:	ISSUED BY:
M. ISOMEN	R.M. LOWRY					

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTODESK CIVIL 3D	DATE JULY 11	36	C	10WXXX–1	R 0

PLOT FACTOR: 1:1

C.A.D. DRAWING
DO NOT ALTER MANUALLY

GEOSYNTEC CONSULTANTS, INC.	0
TASK COMPLETED BY:	REV NO.

L:\CAD\TVKINGSTON FOSSIL PLANT- CONSTRUCTION FOR GYPSUM DISPOSAL FACILITY, PHASE 1A AREA (SHEETS)\DRAWINGS\10WXXX-1.DWG

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APPENDIX A

Document 11

Report of Hydrologic and Hydraulic Analysis, Ash Pond and Stilling Pond, TVA Kingston Fossil Plant, Roans County, Tennessee, Stantec Consulting Services, September 30, 2010

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Report of Hydrologic and Hydraulic Analysis

Ash Pond and Stilling Pond
TVA Kingston Fossil Plant
Roane County, Tennessee

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

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

September 30, 2010

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

September 30, 2010 rpt_003_175660008

Ms. Shannon Bennett
Tennessee Valley Authority
1101 Market Street, LP 5E-C
Chattanooga, Tennessee 37402

Re: Report of Hydrologic and Hydraulic Analysis
Ash Pond and Stilling Pond
TVA Kingston Fossil Plant
Roane County, Tennessee

Dear Ms. Bennett:

Stantec Consulting Services Inc. (Stantec) has been assisting TVA with risk assessment and mitigation for a number of facilities associated with its coal combustion processes at various fossil plants. The Ash Pond and Stilling Pond at the Kingston Fossil Plant were identified for Hydrologic and Hydraulic Freeboard Analysis as a part of our Phase 1 Assessment. The goal of this analysis was to develop a conceptual-level hydrologic and hydraulic runoff model of the area to help assess capacity, freeboard, and hydraulic operation of the Ash Pond and Stilling Pond during various hydrologic events. Results of this modeling effort and recommendations are included in the attached report.

The normal freeboard conditions were assessed and found to be adequate (>5 feet) according to TVA guidelines. Storm surge conditions were also assessed. In general, the capacity of the Ash Pond and Stilling Pond along with their spillways was found to be adequate for most storm events up to and including the 100-year return period. However, during the Probable Maximum Precipitation (PMP) event, there is a chance that the ponds could overtop. Stantec recommends that TVA assess the risk associated with a PMP event in comparison with the amount of time this pond system will remain in operation, and then consider options to reduce the flooding potential for this pond system if performance during the PMP event is required.

Potential options for improving operation during that period could include constructing a defined overflow pathway from the Ash Pond to the Stilling Pond and adding an emergency overflow spillway for the Stilling Pond in the event that overtopping occurs. Additional information from the modeling efforts and an explanation of potential improvements is included herein.

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We appreciate the opportunity to assist TVA with these assessment efforts and look forward to continuing work on other TVA facilities. Please do not hesitate to contact Stantec with any questions, concerns, or clarifications.

Sincerely,

STANTEC CONSULTING SERVICES INC.

C.W. Gaskill, PE
Water Resource Engineer

Erman Caudill, PE, CFM
Project Manager

/cmw

US EPA ARCHIVE DOCUMENT

Report of Hydrologic and Hydraulic Analysis

Ash Pond and Stilling Pond
TVA Kingston Fossil Plant
Roane County, Tennessee

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

September 30, 2010

Report of Hydrologic and Hydraulic Analysis
Ash Pond and Stilling Pond
TVA Kingston Fossil Plant
Roane County, Tennessee

Table of Contents

Section **Page No.**

Executive Summary ..... **iii**

1. Introduction **1**

2. Modeling Assumptions **1**

3. Methodology **2**

4. Input Data **3**
4.1. Watershed Parameters 3
4.2. Rainfall Data 4
4.3. Spillway Data 4
4.4. Pond Overflow and Normal Pool 6
4.5. Stage Storage Data 6
4.6. Spillway Rating Curves 6
4.7. Plant Process Flow 6

5. Results **7**
5.1. Capacity and Freeboard Results 7
5.2. Ash Pond Closure Scenario Conditions Results 7

6. Conclusions and Recommendations **8**

7. References **9**

US EPA ARCHIVE DOCUMENT

Table of Contents
(Continued)
List of Tables

Table		Page No.
Table 1.	Watershed Parameters	3
Table 2.	Rainfall Depths	4
Table 3.	Existing Principal Spillway and Emergency Spillway Data	5
Table 4.	Pond Tailwater Elevations	5
Table 5.	Pond Overflow Elevation	6
Table 6.	Estimated Peak Pool Elevations	7
Table 7.	Estimated Peak Pond Outflows	7
Table 8.	Estimated Peak Pond Outflows	7
Table 9.	Estimated Peak Pool Elevations – Ash Pond Closure Scenario	8
Table 10.	Estimated Peak Pond Inflows – Ash Pond Closure Scenario	8
Table 11.	Estimated Peak Pond Outflows – Ash Pond Closure Scenario	8

List of Appendixes

Appendix A Connectivity Map
Appendix B TVA Historical Drawings
Appendix C Rating Curves
Appendix D TVA Survey Data
Appendix E Wastewater Flow Schematic

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Executive Summary

A hydrologic and hydraulic study was conducted for the Ash Pond and Stilling Pond at the Kingston Fossil Plant in Roane County, Tennessee. The purpose of the study was to help assess freeboard requirements, capacity, and hydraulic operation of spillway systems in relation to the structural hazard classifications that would be appropriate in Tennessee using the effective size of the facilities. In order to perform the study, site visits were conducted, TVA personnel were interviewed, historical drawings and documents were reviewed, survey data was obtained, and hydrologic/hydraulic (H&H) modeling was performed.

An H&H model was developed to simulate storm water drainage and runoff from overland areas, process discharges, and pond interconnectivity by spillways based on our understanding of the geometry and design of the drainage and conveyance network. A map showing the hydraulic connectivity is attached as Appendix A. The model was used to assess the performance of the ponds during the 1-, 10-, 25-, 50- and 100-year 24-hour SCS Type II storms as well as a 6-hour Probable Maximum Precipitation (PMP) event.

Based on the data gathering efforts, collective review of the data available, and the modeling efforts, Stantec noted the following observations:

- Aside from the principal spillway systems, there are no defined emergency spillways or overflow paths. Ponds similar in size and capacity to these typically have emergency spillway systems to prevent overflows.
    - If the principal spillways were to become clogged, or if a heavy rainfall event were to cause any of the ponds to overtop, there are no defined and protected overflow paths to help prevent erosion of the dikes.
- Based on the modeling of the current conditions, the Ash Pond appears unable to pass the PMP event through the existing principal spillway system without overtopping the embankment. It does appear to be able to pass the 100-year storm event.
    - In the event of a PMP storm, the Ash Pond would overtop the dike between the Ash Pond and the Stilling Pond as well as the embankment between the Ash Pond and the Emory River.
- A potential closure scenario was modeled assuming the Stilling Pond would function as it currently does, but act as a settling basin for the to-be closed Ash Pond prior to discharge to the Emory River. As presently configured and under this operational scenario, the Stilling Pond appears unable to pass the PMP event through the spillway system without overtopping the embankment. It does appear to be able to pass the 100-year storm event.
    - In the event of a PMP storm during the above described closure scenario, the Stilling Pond would overtop the dike between the Stilling Pond and the Emory River.

- As we understand, the principal spillway risers in the Stilling Pond are targeted for potential replacement. Upgrading the principal spillways so that the Stilling Pond would pass the PMP event would result in upgrading the six (6) existing risers to approximately 7' diameter risers with 6' diameter barrels. A complete design would need to be performed to determine the exact up-sizing necessary, but this would not appear to be as effective as a formal emergency spillway system for controlling large storm events such as the PMP.

Based on the results of the analysis, Stantec recommends TVA consider the following:

- Assess the risk of a PMP event occurring during the remaining life of the pond system.

- If necessary, consider installing an emergency spillway & overflow connector between the Ash Pond and Stilling Pond.

- Consider installing an emergency spillway between the Stilling Pond and the Emory River if the principal spillway system is replaced or during the closure of the Ash Pond.

Stantec recommends that these potential improvements be further evaluated by TVA to determine if they are warranted and can be incorporated into future construction projects at the plant.

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Report of Hydrologic and Hydraulic Analysis
Ash Pond and Stilling Pond
TVA Kingston Fossil Plant
Roane County, Tennessee

1. Introduction

This study was conducted to help assess capacity and hydraulic operation of spillway systems and freeboard requirements in relation to the structural hazard classifications of the facilities at the Kingston Fossil Plant in Roane County, Tennessee. The ponds evaluated include the Ash Pond and Stilling Pond.

This analysis included field visits, review of historical TVA drawings and discussions with TVA personnel. This report details the assumptions, methodology, and results of the H&H analyses for the ponds analyzed.

2. Modeling Assumptions

a. Pipes are assumed to be flowing freely and are not clogged or leaking. Some of the pipes may, in actuality, be clogged and some of the older pipes may be leaking (especially older corrugated metal pipes). Elevations and flows determined for this analysis may not be applicable in those situations. This assumption is inherent in this type of analysis and is acceptable.

b. Wave action is not considered in this analysis. Overtopping is assumed to occur only when the elevation of the pond rises above the minimum surveyed crest elevation. In actuality, wave action would likely play a role in the overtopping of the ponds. The modeling performed for this work is conceptual in nature. Compensating for wave action is beyond this scope of work and would not change the outcome of the study.

c. The model takes into account a tailwater effect of the downstream water surface elevation (WSEL) of the Stilling Pond on the upstream outfall structures in the Ash Pond. A rating curve was developed for each storm event for the Ash Pond utilizing an expected downstream WSEL for the Stilling Pond. The expected WSEL of the Stilling Pond was generated using the HEC-HMS model as if the outfall structures of the Ash Pond were free flowing. This generated a maximum WSEL for the Stilling Pond. This WSEL was then used to generate a new rating curve for the Ash Pond. This gives a conservative value for the Ash Pond rating curves because only the maximum WSEL of the downstream Stilling Pond was used. All rating curves can be reviewed in Appendix C.

d. During the initial run for the 6-HR PMP event, the model ends when the WSEL rises to the top of the pond at an elevation of 765.00 ft. This elevation is the actual elevation of the top of the embankment. This action showed that the water in the Ash Pond would overtop the existing dike during the 6-HR PMP event. To estimate the amount of flow overtopping the dike and entering the downstream Stilling Pond, weir flow going over the top of the dike had to be modeled. In reality, not all of the overtopping flow would enter the Stilling Pond

as some of the flow would leave the Ash Pond and flow directly into the river. For the purposes of this model it was proposed that all the overtopping flow would enter the Stilling Pond.

e. For modeling purposes the spillway was conservatively estimated to be the length of the dike separating the two ponds at 2070 LF. In reality, a localized low area would serve as the primary flow path. To facilitate modeling, a new pond top elevation was used at 766.00 ft. The surface area of the pond at both the 766.00 ft and 765.00 ft elevations was estimated to be 83.58 acres. With these two values, a new stage-discharge rating curve and a stage-volume curve could be generated for the 6-HR PMP event. The 6-HR PMP Ash Pond rating curve can be reviewed in Appendix C.

f. NRCS/SCS TR-55 methodology was used for drainage area calculations. Wherever ash existed in the drainage area, it was treated as a Hydrologic Group C soil. The water surface of the ponds was considered to be impervious, to model 100% of the rainfall being captured in the pond.

g. A time of concentration was assumed to be 5 minutes (lag time of 3 minutes) for the existing conditions. This value was selected as a conservative assumption based on the varied flow path of the watershed subbasins.

h. For the closure scenario (described below), the existing process discharges and a drainage area of 320 acres was assumed to flow to the Stilling Pond. A time of concentration of 50 minutes (lag time of 30 minutes) and a curve number of 80 were also assumed. These assumptions were based on estimates of the anticipated closure conditions for the Ash Pond.

i. It should be noted, that this was an uncalibrated model and sufficient data from actual storms was not available to calibrate it. Stantec tested the sensitivity of the model to input parameters and found the overall general results of the model consistent throughout. This model is suitable for planning purposes, but it should not be used for simulation of actual storm events without further calibration efforts involving actual storm discharge and stage measurements. This model is suitable as a screening and planning tool, however Stantec would discourage its use beyond the current scope of work and the context described in this report.

3. Methodology

Rainfall-runoff relationships were determined using methods described by the NRCS in “Part 630-Hydrology” of the National Engineering Handbook (NEH4). SCS Curve Number Unit Hydrograph methods were used to generate runoff hydrographs for routing through the ponds in lieu of the more complex methods described in Chapter 21 of NEH4 and commonly implemented in NRCS TR-60 based methods.

A HEC-HMS model was developed and used to simulate runoff from the probable maximum precipitation (PMP) event in accordance with TVA design guidance. SCS Type II rainfall depth for the 1-, 10-, 25-, 50-, and 100-year storm events were taken from NOAA Atlas 14 for Kingston, TN. Rainfall depth for the 6-HR PMP event was taken from NOAA HMR-56. From

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Figure 23 *"6 hr 1-mi^2 PMP (in.) – eastern half of Tennessee River Watershed"* of HMR-56, the 6-HR PMP rainfall depth for the Kingston site was estimated to be 36.3 in.

The PMP event was formatted using the distribution chart included in NRCS TR-60, Figure 2-4 "*Dimensionless design storm distribution, auxiliary spillway and freeboard.*" A formatted 6-HR PMP chart can be found in Appendix C.

Pump flows are given for normal and full operation. The "Wastewater Flow Schematic" provided by TVA to Stantec states that the plant flows are 48.46 MGD (75 cfs). This value is used as a constant input in the model. The "Wastewater Flow Schematic" is included in Appendix E.

4. Input Data

4.1. Watershed Parameters

It is our understanding that process water enters the Ash Pond. The Ash Pond discharges to the Stilling Pond through an adjustable weir structure. Drawings utilized to develop the connectivity which was used in the creation of the hydrologic model are included in Appendix A. The following table lists the main hydrologic parameters of the watersheds draining to the ponds. The impervious area is a separate entry into the model and the curve numbers listed below are for the pervious sections of the drainage areas only.

Table 1. Watershed Parameters

Name	Drainage Area (acres)	Receiving Pond	Curve Number	*Percent Impervious	Estimated Lag Time (min)
Ash Pond Area	90	Ash Pond	71	*34	3
Ash Storage and Processing Area	40	Ash Pond	87	*0	3
Sluice Channel Area	22	Ash Pond	87	*23	3
Dredge Cell Dike Slope Area	23	Ash Pond	80	*0	3
Stilling Pond Area	29	Stilling Pond	87	*83	3
Stilling Pond Area (Closure)	320	Stilling Pond	80	0	30

*The amount of impervious area is not reflected in the Curve Number.

A potential closure scenario for the Ash Pond was also modeled assuming the Stilling Pond would act as a settling basin prior to discharge to the Emory River. The Ash Pond and Dredge Cell Areas were assumed to be filled and regraded to drain positively to the stilling pond and all the process water was routed through the Stilling Pond. Although the exact configuration may change during design, this assumption appears to be sufficient for this concept level model.

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4.2. Rainfall Data

Rainfall depths were taken from NOAA Atlas 14 for the storm events evaluated. The PMP event was formatted using the distribution chart included in NRCS TR-60, Figure 2-4 "*Dimensionless design storm distribution, auxiliary spillway and freeboard.*" Rainfall depths used in the HMS model are summarized below in Table 2.

Table 2. Rainfall Depths

Storm Event	Rainfall Depth (inches)
1-year 24-hour	2.84
10-year 24-hour	4.72
25-year 24-hour	5.52
50-year 24-hour	6.15
100-year 24-hour	6.80
6-hour PMP	36.3

4.3. Spillway Data

TVA drawings 10W421, 10N422, 10N423 depict the six risers in the Stilling Pond. The six riser structures were located and appeared to be consistent with current conditions during the field visit. Data was used from these drawings in the H&H analysis. Drawings can be found in Appendix B.

TVA drawings 10W425-24, 10W425-70, 10W425-77, 10W425-78, 10W425-79, 10W425-80 depict the five risers in the Ash Pond. The five riser structures were located and appeared to be consistent with current conditions during the field visit. Data was used from these drawings in the H&H analysis.

An Excel file provided by TVA titled "TVA Spillway Concrete Top Survey"; dated January 23, 2009 lists multiple plants with surveyed riser elevations and approximate riser invert elevations. The approximate riser invert elevations match the data given in TVA drawing 10W421. Data was used from these files in the H&H analysis. This was a conservative assumption used in the model. Stantec tested the sensitivity of this assumption and it did not overrule any of the findings discussed herein. A summary of the spillway data is listed below in Table 3.

The data gathered from these files were compiled to create the rating curves. The Ash Pond rating curves estimate the tailwater effects from the water surface elevation of the downstream Stilling Pond. The tailwater elevation changed with each storm event. Tailwater effects on the Stilling Pond were assumed to be equal to the 100-YR water surface elevation of the receiving river for all storm events. This was a conservative assumption used in the model. Stantec tested the sensitivity of this assumption and it did not overrule any of the findings discussed herein. A summary of the tailwater data is listed below in Table 4.

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Table 3. Existing Principal Spillway and Emergency Spillway Data

Pond	Weir/Orifice Structure	Weir Length (ft)	Weir Elevation (ft)	Orifice Opening (ft^2)	Orifice Invert (ft)	Data Source
Ash Pond	1	48	760.00	36	752.00	TVA drawings 10W425-70, 10W425-79, 10N425-80
Ash Pond	2	48	760.00	36	752.00	TVA drawings 10W425-70, 10W425-79, 10N425-80
Ash Pond	3	48	760.00	36	752.00	TVA drawings 10W425-70, 10W425-79, 10N425-80
Ash Pond	4	48	760.00	36	752.00	TVA drawings 10W425-70, 10W425-79, 10N425-80
Ash Pond	5	48	760.00	36	752.00	TVA drawings 10W425-70, 10W425-79, 10N425-80
Stilling Pond	A	48	756.54	36	746.00	"TVA Spillway Concrete Top Survey", 1-23-09
Stilling Pond	B	48	754.48	36	746.00	"TVA Spillway Concrete Top Survey", 1-23-09
Stilling Pond	C	48	754.44	36	746.00	"TVA Spillway Concrete Top Survey", 1-23-09
Stilling Pond	D	48	754.43	36	746.00	"TVA Spillway Concrete Top Survey", 1-23-09
Stilling Pond	E	48	754.45	36	746.00	"TVA Spillway Concrete Top Survey", 1-23-09
Stilling Pond	F	48	754.42	36	746.00	"TVA Spillway Concrete Top Survey", 1-23-09

Table 4. Pond Tailwater Elevations

Pond	Riser Structure	Storm Event	Tailwater Elevation (ft)
Ash Pond	1-5	1-YR	755.2
Ash Pond	1-5	10-YR	755.5
Ash Pond	1-5	25-YR	755.6
Ash Pond	1-5	50-YR	755.6
Ash Pond	1-5	100-YR	755.7
Ash Pond	1-5	6-hr PMP	761.4
Stilling Pond	A-F	100-YR	748.4

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4.4. Pond Overflow and Normal Pool

Table 5 shows the embankment elevation and the assumed normal pool elevations at each pond. The normal pool elevations were assumed to be equal to the principal riser elevation in the pond.

Table 5. Pond Overflow Elevation

Pond Name	Embankment Elevation (feet)	Normal Pool Elevation (feet)
Ash Pond	765.00	760.0
Stilling Pond	765.00	754.4

4.5. Stage Storage Data

Stage storage curves were developed for each pond based on data provided by TVA. For the Ash Pond and Stilling Pond, the stage-storage data came from AutoCAD files *Topo 02-27-10.dwg* and *MainAshPond 8-20-09.dwg*. Stage-storage curves are included in Appendix C for each pond. Survey data utilized can be found in Appendix D.

4.6. Spillway Rating Curves

Rating curves for the spillway systems were developed based on the geometric data available and weir, orifice, and culvert discharge relationships. Weir equations and coefficients were based on guidance provided in "Open Channel Hydraulics," V.T. Chow, 1959. Orifice equations and coefficients were based on guidance provided in "Handbook of Hydraulics," E. F. Brater and H.W. King, 1976. Culvert discharge ratings were developed using procedures outlined in "Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5 (HDS-5)," U.S. Department of Transportation Federal Highway Administration (FHWA) 1985.

The model takes into account a tailwater effect of the downstream WSEL of the Stilling Pond on the upstream outfall structures in the Ash Pond. A rating curve was developed for each storm event for the Ash Pond utilizing an expected downstream WSEL for the Stilling Pond. Tailwater elevation for the Stilling Pond was assumed to be the 100-YR WSEL of the receiving waterway (Emory River). Rating curves for each pond are attached in Appendix C.

4.7. Plant Process Flow

Pump discharge values used in the modeling match those provided on the wastewater flow schematic. As we understand, these discharge values are indicative of typical conditions during a normal full operation condition. According to the wastewater flow schematic, provided by TVA to Stantec the plant flows are 48.46 MGD (75 cfs). This value is used as a constant input in the model.

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5. Results

Results are summarized in the following sections for the capacity/freeboard analysis. The results shown are based on the assumptions described herein and should be considered approximate.

5.1. Capacity and Freeboard Results

Estimated peak pool elevations for the storms analyzed are shown in Table 6. Numbers in bold indicate that the pond is likely overtopping the embankment for the associated storm event. Table 7 shows the estimated peak pond inflows associated with each event and Table 8 shows estimated peak pond outflows associated with each event.

Table 6. Estimated Peak Pool Elevations

Pond Name	1-year 24-hour storm (ft)	10-year 24-hour storm (ft)	25-year 24-hour storm (ft)	50-year 24-hour storm (ft)	100-year 24-hour storm (ft)	6-hr PMP storm (ft)
Ash Pond	760.8	761.1	761.2	761.3	761.4	***765.4**
Stilling Pond	755.2	755.4	755.5	755.5	755.6	761.0

*Vertical sides assumed above top of dike elevation at 765 ft to model overtopping event.

Table 7. Estimated Peak Pond Outflows

Pond Name	1-year 24-hour storm (ft)	10-year 24-hour storm (ft)	25-year 24-hour storm (ft)	50-year 24-hour storm (ft)	100-year 24-hour storm (ft)	6-hr PMP storm (ft)
Ash Pond	138	221	262	295	334	2154
Stilling Pond	128	191	221	247	273	718

Table 8. Estimated Peak Pond Outflows

Pond Name	1-year 24-hour storm (ft)	10-year 24-hour storm (ft)	25-year 24-hour storm (ft)	50-year 24-hour storm (ft)	100-year 24-hour storm (ft)	6-hr PMP storm (ft)
Ash Pond	138	221	262	295	334	2154
Stilling Pond	128	191	221	247	273	718

5.2. Ash Pond Closure Scenario Conditions Results

A separate H&H basin model for the closure scenario was created. For the closure scenario, the Ash Pond Stilling Pond was found to be sufficient to pass the 100-year storm, but not the PMP. See Tables 9, 10 and 11 for a summary of the results.

Table 9. Estimated Peak Pool Elevations – Ash Pond Closure Scenario

Pond Name	1-year 24-hour storm (ft)	10-year 24-hour storm (ft)	25-year 24-hour storm (ft)	50-year 24-hour storm (ft)	100-year 24-hour storm (ft)	6-hr PMP storm (ft)
Stilling Pond	754.9	755.5	755.7	755.9	756.2	***766.0**

*Vertical sides assumed above top of dike elevation at 765 ft to model overtopping event.

Table 10. Estimated Peak Pond Inflows – Ash Pond Closure Scenario

Pond Name	1-year 24-hour storm (cfs)	10-year 24-hour storm (cfs)	25-year 24-hour storm (cfs)	50-year 24-hour storm (cfs)	100-year 24-hour storm (cfs)	6-hr PMP storm (cfs)
Stilling Pond	227	553	700	819	942	5960

Table 11. Estimated Peak Pond Outflows – Ash Pond Closure Scenario

Pond Name	1-year 24-hour storm (cfs)	10-year 24-hour storm (cfs)	25-year 24-hour storm (cfs)	50-year 24-hour storm (cfs)	100-year 24-hour storm (cfs)	6-hr PMP storm (cfs)
Stilling Pond	69	230	315	371	400	4962

6. Conclusions and Recommendations

Based on the data gathering efforts, collective review of the data available, and the modeling efforts, Stantec noted the following observations:

- Aside from the principal spillway systems, there are no defined emergency spillways or overflow paths. Ponds similar in size and capacity to these typically have emergency spillway systems to prevent overflows.
    - o If the principal spillways were to become clogged, or if a heavy rainfall event were to cause any of the ponds to overtop, there are no defined and protected overflow paths to help prevent erosion of the dikes.
- Based on the modeling of the current conditions, the Ash Pond appears unable to pass the PMP event through the existing principal spillway system without overtopping the embankment. It does appear to be able to pass the 100-year storm event.
    - o In the event of a PMP storm, the Ash Pond would overtop the dike between the Ash Pond and the Stilling Pond as well as the embankment between the Ash Pond and the Emory River.
- A potential closure scenario was modeled assuming the Stilling Pond would function as it currently does, but act as a settling basin for the to-be closed Ash Pond prior to discharge to the Emory River. As presently configured and under this operational scenario, the Stilling Pond appears unable to pass the PMP

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event through the spillway system without overtopping the embankment. It does appear to be able to pass the 100-year storm event.

- In the event of a PMP storm during the above described closure scenario, the Stilling Pond would overtop the dike between the Stilling Pond and the Emory River.

- As we understand, the principal spillway risers in the Stilling Pond are targeted for potential replacement. Upgrading the principal spillways so that the Stilling Pond would pass the PMP event would result in upgrading the six (6) existing risers to approximately 7' diameter risers with 6' diameter barrels. A complete design would need to be performed to determine the exact up-sizing necessary, but this would not appear to be as effective as a formal emergency spillway system for controlling large storm events such as the PMP.

Based on the results of the analysis, Stantec recommends TVA consider the following:

- Assess the risk of a PMP event occurring during the remaining life of the pond system.

- If necessary, consider installing an emergency spillway & overflow connector between the Ash Pond and Stilling Pond.

- Consider installing an emergency spillway between the Stilling Pond and the Emory River if the principal spillway system is replaced or during the closure of the Ash Pond.

Stantec recommends that these potential improvements be further evaluated by TVA to determine if they are warranted and can be incorporated into future construction projects at the plant.

7. References

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APPENDIX A

Document 12

Stantec Consulting Services Correspondence Dated October 11, 2011, "Re: Addendum, September 30, 2010 H&H Report – KIF Ash Pond/Stilling Basin, Dam Safety Regulation – Spillway Capacity Evaluation"

Stantec Consulting Services Inc.
1409 North Forbes Road
Lexington KY 40511-2050
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October 11, 2011 let_001_175601004

Mr. Scott Turnbow, PE
Tennessee Valley Authority
1101 Market Street, LP 5E
Chattanooga, Tennessee 37402-2801

Re: Addendum
September 30, 2010 H&H Report – KIF Ash Pond/Stilling Basin
Dam Safety Regulation-Spillway Capacity Evaluation

Dear Mr. Turnbow:

This addendum letter presents Stantec's supplemental Hydrologic and Hydraulic (H&H) analysis on the Ash Pond at Kingston. The initial H&H model and analysis was developed to determine freeboard and capacity in the Ash Pond with results summarized in a report dated September 30, 2010. The initial analysis was performed consistent with TVA's overall objective of evaluating storm freeboard and capacity on all major Fossil Power impoundments. That analysis included evaluation of the full Probable Maximum Precipitation (PMP) event consistent with the design criteria established in TVA's Programmatic Document. The use of the PMP in the initial analysis was intended to provide uniform performance benchmarking of all facilities as part of the overall engineering assessment program.

Consistent with TVA's directive that the design criteria presented in the Programmatic Document was not intended to be retro-active to existing facilities, this supplemental analysis adopts design criteria established within the Tennessee Dam Safety regulations which are viewed as the governing minimum design standards.

Spillway capacity and freeboard analysis design criteria are established in the dam safety regulations based on the structure hazard classification and size of the impoundment. The Ash Pond hazard classification is designated as significant based on the assessment that a failure is not expected to cause a loss of life. The structure falls into the regulatory intermediate category for size based on an estimated storage volume of approximately 1,900 acre-feet. Based on the metrics as outlined in the dam safety regulations, the Ash Pond spillway is required have the capacity to pass a design storm corresponding to one-half (½) the PMP event without overtopping the perimeter dikes.

Engineering methodology for this analysis is documented in the initial report. This supplemental analysis included revising the rating curve for the Ash Pond to reflect the anticipated tailwater condition in the Stilling Pond during the design storm event. The maximum tailwater elevation for the Stilling Pond was determined to be approximately 756.8'. The resulting rating curve and HEC-HMS Input and Output are provided as attachments to this letter.

The results of this analysis indicate that the Ash Pond reaches a peak elevation of 764.4' during the designated design storm which is below the crest of the dike (765.0'). Therefore, the Ash Pond is able to pass the ½-PMP rainfall event through the existing Ash Pond outlet structures without the pond overtopping and meets the pertinent requirements designated in Tennessee's Dam Safety regulations.

Stantec appreciates the opportunity to provide these TVA engineering support services. If you have any questions or need additional information, please contact Don Fuller at 859-619-8960.

Sincerely,

STANTEC CONSULTING SERVICES INC.

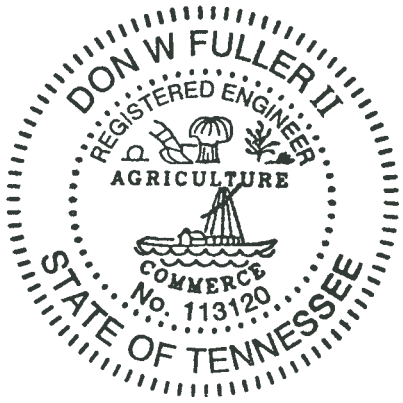
Michelle Meehan, PE
Project Engineer

Don Fuller II, PE
Principal

/mmm/rdr

2 Enclosures: Rating Curve and HEC-HMS Input-Output

c. Jamey Dotson, Michelle Cagley, John Dizer



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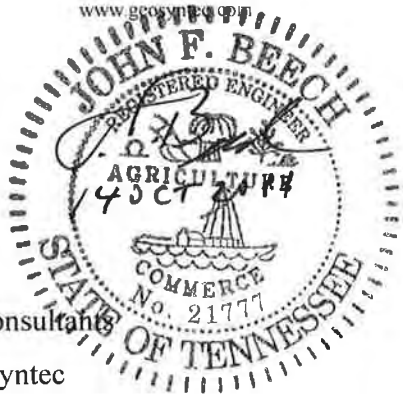
APPENDIX A

Document 13

Geosyntec Consultants Technical Memorandum, October 14, 2011, "Subject: Evaluating Stormwater Runoff in the Gypsum Disposal Area for the One-Third Probable Maximum Precipitation Event

Geosyntec consultants

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TECHNICAL MEMORANDUM

Date:	10/14/2011
To:	Jamey Dotson, Tennessee Valley Authority
Copies to:	Neil Davies, Mehmet Iscimen, Cuneyt Gokmen, Geosyntec Consultants
From:	Jay Beech, Jesus Sanchez, Mark Schillinger, Eban Bean, Geosyntec Consultants
Subject:	Evaluating Stormwater Runoff in the Gypsum Disposal Area for the One-Third Probable Maximum Precipitation Event

PURPOSE

The purpose of this technical memorandum is to evaluate the stormwater runoff in the Gypsum Disposal Area (GDA) located at the Kingston Fossil Plant (KIF) Peninsula Site for the One-Third (1/3) Probable Maximum Precipitation (PMP) event, as requested by Tennessee Valley Authority (TVA). The operational conditions of the GDA is analyzed in this memorandum is depicted in Figure 1. For this analysis the stormwater runoff conveyance for the GDA consists of: (i) the Phase IA area with the composite liner system and drainage layers in place; and (ii) construction spoils stockpiles in the unlined Phase IB area. The Phase IA and IB areas are separated by a divider dike. Stormwater runoff in the Phase IB area is pumped to the stormwater pond. The inflows in Phase IA from stormwater runoff and the slurrying of gypsum is conveyed to the settling pond and then pumped to the stormwater pond. Phase IA and IB areas can be conveyed to the stormwater pond by pumps that are situated near the low-spot of each area.

If stormwater runoff flows exceed the capacity of the pumps in the Phase I area then a portion of the excess can be conveyed by three 44-inch culverts that connect the stormwater pond to the Phase IA area (stormwater in Phase IB will overtop the divider dike and flow into Phase IA). The stormwater pond has an emergency spillway to prevent ponded water from overtopping the berm forming the pond.

If the stormwater runoff in the Phase IA area exceed the flow capacity through the three 44-inch culverts then the ponded water will overtop the GDA berm. Similarly, if total flows into the stormwater pond exceed the design capacity of the spillway then the ponded water in the stormwater pond will overtop the dike.

The hydrologic and hydraulic analyses of the Phase IA and Phase IB areas along with drainage from the surrounding landfill and direct rainfall into the adjacent stormwater pond were modeled using HydroCAD™ (HydroCAD, 2009). The modeling results were used to evaluate whether the

1/3 PMP Event would result in exceedance of flow capacity through: (1) the three 44-inch culverts from the Phase IA area to the stormwater pond and/or (2) the emergency spillway in the stormwater pond.

PROBABLE MAXIMUM PRECIPITATION

The PMP is defined in the National Oceanic and Atmospheric Administration's (NOAA) Hydrometeorological Report (HMR) No. 56 as "Theoretically the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year."

The 6-hour, 1-mi^2 PMP for the Site location is estimated to be 36.3 inches from HMR 56 (NOAA, 1986); by comparison, the 1,000-yr, 6-hr event for this location is 7.01 in. (NWS, 2011). The storm considered here is a 1/3 PMP, which is equivalent to 12.1 in.

Following guidance provided in HMR 56, the depth of the 1/3 PMP was then used to calculate the hourly cumulative rainfall depths within the event. Cumulative 1/3 PMP depths were then converted to 1-hr incremental depths.

Hourly rainfall depths were then arranged following guidance provided in HMR 56 with the intention to produce the greater runoff rates at the Site. The six 1-hour rainfall depths were arranged in the following order: 5, 4, 3, 2, 1, and 6, with 1 being the most intense hour and 6 being the least intense. The time series was then transformed into a 15 minute incremental time series which accounted for changing intensities over the hourly time steps. This produced a peak intensity of 8.8 in/hr for the most intense 15 minute period (2.2-in.). The rainfall time series was then converted to a mass fraction time series for incorporation into HydroCAD^{TM}.

HYDROLOGIC AND HYDRAULIC MODELING

A hydrologic and hydraulic model was developed for the 1/3 PMP rainfall event at the GDA for the Phase I area. The drainage areas from to both phases are conservatively assumed to be impervious and could be conceptualized as sheet flow that would drain to the low elevation for each respective area. The depth of water in the settling pond in Phase IA is assumed to be 5-ft (an elevation of 756 ft-MSL) prior to the start of the 1/3 PMP event. The stormwater pond water surface elevation (WSEL) was assumed to be at the elevation of the lowest pump (759.5 ft-MSL) prior to the rainfall event. It was assumed the pumps would malfunction during the extreme event. In addition, it was assumed that slurrying of gypsum would be discontinued during the 1/3 PMP event.

RESULTS AND CONCULSIONS

The HydroCAD model results indicate that the peak WSELs from the 1/3 PMP event in the stormwater pond, Phase IA area, and Phase IB area were 761 ft-MSL, 759 ft-MSL, and 762 ft-MSL, respectively. These peak WSELs indicate that the stormwater pond would not be hydraulically connected to the Phase IA area as the inlet invert of the riser structure, which is connected to the three 48-inch culverts, has an elevation of 765 ft-MSL. The Phase I Divider Dike's low elevation of 770 ft-MSL was higher than the peak WSELs in both the Phase IA and IB areas, indicating that the Phase IB Area would not be hydraulically connected via overtopping during this event. These results indicate the dikes would not be overtopped during a 1/3 PMP event and the runoff and direct rainfall into the stormwater pond would not be sufficient enough to reach the invert of the emergency spillway.

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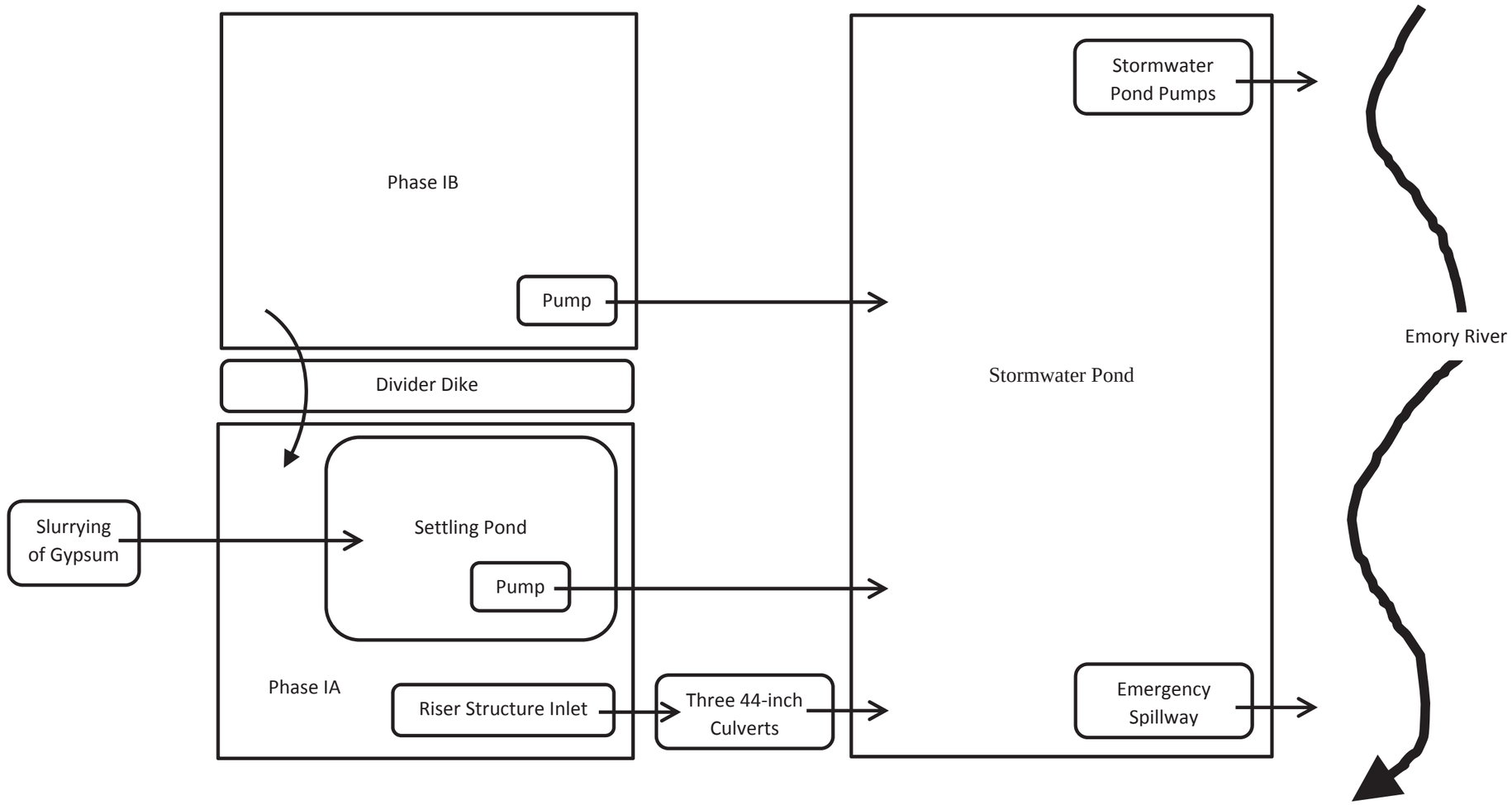
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APPENDIX A

Document 14

Stantec Consulting Service correspondence Dated October 5, 2011 "Re: Results of Seismic Slope Stability Analysis, Main Ash Pond, Stilling Pond, Dike C, Kingston Fossil Plant"

Stantec Consulting Services Inc.
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October 5, 2011 ltr_009_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
Main Ash Pond, Stilling Pond, Dike C
Kingston 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 Kingston Fossil Plant (KIF) 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 analysis for Dike C of KIF's Ash Pond Stilling Basin.

The seismic slope stability analysis results presented in this letter employ a pseudostatic approach and are representative of current conditions. For seismic assessment in upcoming closure design, 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 KIF, closure of the Ash Pond Stilling Basin is currently planned for approximately 2020.

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 Stantec's geotechnical report entitled Report of Geotechnical Exploration and Slope Stability for Dike C; Kingston Fossil Plant, Harriman, Tennessee; August 3, 2009.

- SLOPE/W software (from GEO-SLOPE International, Inc.) was used to perform the calculations.

- One existing SLOPE/W cross-section model was selected for analysis of Dike C. The selected section is representative of Dike C's lowest current static (long-term) factor of safety, with consideration given to proper representation of a release/breach.

- 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 coefficients. 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". The peak ground acceleration (or seismic coefficient) for a 500 year return period was selected from Table 20 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.

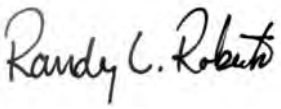
3. Results

The results of the pseudostatic stability analysis indicate a factor of safety of 1.4 for the section representing Dike C of the Ash Pond Stilling Basin. This exceeds the target of 1.0. Enclosure B contains a summary spreadsheet, SLOPE/W cross-section, and plan view showing the cross-section location.

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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APPENDIX A

Document 15

Geosyntec Consultants "Site Response and Seismic Deformation Analyses, Kingston Peninsula Permit Modification," November 10, 2010

Written by:	Neven Matasovic	Date:	11/24/2010	Reviewed by:	M. Othman	Date:	11/24/2010

Client:	TVA	Project:	Kingston Peninsula Permit Modification	Project/ Proposal No.:	GR4694	Task No.:	05

SITE RESPONSE AND SEISMIC DEFORMATION ANALYSES

PURPOSE

This calculation package presents the results of the site-specific seismic site response and seismic deformation analysis performed by Geosyntec Consultants (Geosyntec) for the coal-combustion residue (CCR) disposal facility (Landfill) at the Kingston Fossil Plant (KIF) near Harriman, Tennessee. The analyses are based upon the probabilistic seismic hazard evaluation performed by Geosyntec [2010], design ground motions developed for KIF by AMEC Geomatrix [2009], site-specific geotechnical investigations by MACTEC [2005; 2006], and pseudostatic slope stability evaluation of the Landfill performed by Geosyntec [2010].

BACKGROUND

The Kingston Fossil Plant (KIF) is near Harriman, Tennessee, approximately 30 miles west of Knoxville. The approximate location of KIF is shown in Figure 1. The approximate coordinates of the geometric center of the KIF are 35.8947 degrees North Latitude and 84.5035 degrees West Longitude. The proposed footprint for Landfill will occupy an area of approximately 92 acres. A plan view of the Landfill is shown in Figure 2. The landfill will be lined and capped with a composite landfill liner and cover. CCRs will be disposed off in the Landfill.

Geotechnical investigation of the Landfill area was performed by MACTEC [2005; 2006]. The results of this investigation served as a basis for Landfill design by Geosyntec [2010]. As a part of design considerations, Geosyntec developed a series of representative cross sections and performed static and pseudostatic slope stability evaluations. Representative cross sections analyzed by Geosyntec are shown in Figure 2. The results of pseudostatic stability evaluations indicated that, from the seismic stability standpoint, cross section 4 is the most critical. The lowest yield acceleration of sliding mass (k_y) of 0.28 g was calculated by Geosyntec [2010] for this cross section.

SEISMIC HAZARD EVALUATION

The seismic hazard for the KIF has been evaluated by AMEC Geomatrix [2009]. This evaluation was performed for an event with 2 percent probability of exceedance (PE) in 50 years (2,475 years return period). The results of AMEC Geomatrix evaluation indicate that seismic hazard at KIF is governed by a Moment Magnitude (M_w) 6 earthquake at an approximate distance of 22.4 miles (36 km) from the geometric center of the site. This design seismic event, referred to herein as the near-field event, is characterized with bedrock Peak Horizontal Ground Acceleration (PHGA) of PHGA = 0.11 g and relatively short duration of shaking (less than 5 seconds). The impact of a M_w 7.6 event on the New Madrid Seismic Zone (NMSZ) at an approximate distance of 270 miles (435 km) was found by AMEC Geomatrix to be of far lesser

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significance. The estimated bedrock PHGA from this M_w 7.6 event is on the order of 0.03 g. Therefore, by inspection, Geosyntec eliminated the M_w 7.6 on the NMSZ from further considerations.

The 2008 edition of the 2 percent PE in 50 years United States Geological Survey (USGS) seismic hazard map is reproduced in Figure 3. This seismic hazard map indicates that bedrock PHGA at the site is on the order of 0.2 g, i.e., almost double the value evaluated for KIF by AMEC Geomatrix [2009].

To provide a conservative basis for seismic design of the Landfill, consistent with the USGS map, Geosyntec selected bedrock PHGA of 0.2 g for seismic site response and seismic deformation analysis of the Landfill.

DESIGN GROUND MOTIONS

Design ground motions are required for seismic site response and seismic deformation analysis. AMEC Geomatrix [2009] developed a suite of three synthetic design ground motions (synthetic time histories) for the KIF site. These design ground motions conform to the spectral ordinates (i.e., frequency content) and duration of shaking of the design near-field event, i.e., they are representative of strong ground shaking in hypothetical bedrock outcrop at the approximate location of the Landfill that were induced by a M_w 6 earthquake at an approximate distance of 22.4 miles (36 km) from the Landfill.

Geosyntec adopted synthetic time histories developed by AMEC Geomatrix [2009] for the design near-field event. The acceleration response spectra of these time histories are shown in Figure 4. Also shown in Figure 4, for quality control purpose, are the target bedrock acceleration spectra and "seed ground motions", that served as a basis for spectral matching and development of design ground motions.

SITE RESPONSE ANALYSIS

General

Geosyntec conducted a site-specific seismic site response analysis for the Landfill. This analysis was conducted to account for the influence of both local soil conditions and Landfill waste fill on the design ground motions. Given the relatively flat site topography, relatively flat slopes of the Landfill, and horizontally layered ground conditions, a one-dimensional (1-D) site response analysis was deemed appropriate.

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Representative Site Response Model

A representative two-dimensional geotechnical model of the Landfill was developed on the basis of a site-specific geotechnical investigations by MACTEC [2005; 2006]. This 2-D model was developed at a location of the most critical cross section identified in a pseudostatic slope stability analysis by Geosyntec [2010] (pseudostatic analysis cross section 4).

The representative site response model is one-dimensional (1-D). This 1-D model was developed in the center of the critical pseudostatic failure surface. The 1-D site response model is schematically shown in Figure 5 as a system of lumped masses, springs, and viscous damping dashpots. It includes the following materials, top to bottom: (i) 130-ft thick pile of fly ash, with composite landfill liner and cover systems at its base and top, respectively; (ii) 35-ft thick deposit of medium stiff to stiff silty clay; and (iv) 15-ft thick deposit of soft silty clay. The site response model also includes crystalline bedrock at its base.

Material Parameters

Material parameters assigned by Geosyntec to the above listed materials are summarized in Table 1. Unit weights listed in Table 1 are from Geosyntec [2010]. Since majority of the waste material will consist of fly ash, the waste layer is modeled as a fly ash layer. Shear wave velocities assigned to the on-site materials were either directly measured (fly ash; AECOM, 2009] or indirectly estimated using the Imai and Tonouchi [1982] correlation between Standard Penetration Test (SPT) blow count and shear wave velocity. Modulus reduction and damping curves assigned to the materials listed in Table 1 were selected based upon material classification and/or Plasticity Indices (PI-s) reported in Geosyntec [2010] and engineering judgment. For example, the fly ash material was modeled as low plasticity silt [Ishihara et al., 1987 set of material parameters] while, based upon material description and recorded low SPT blow count, the stiff and soft clay materials were modeled as clay of high plasticity using the Vucetic and Dobry [1991] set of material parameters. The clay liner was modeled as clay (PI 50) using the Vucetic and Dobry [1991] set of material properties.

Numerical Model

One-dimensional non-linear site response analysis of the representative landfill/soil profile was conducted using the computer program D-MOD2000 [Matasovic, 1993; Matasovic, 2006; www.GeoMotions.com]. D-MOD2000 is a fully nonlinear code that solves the dynamic equation of motion in the time domain. The behavior of soil and soil-like materials is modeled in D-MOD2000 using the non-linear hysteretic constitutive model proposed by Matasovic and Vucetic [1993]. A small amount of viscous damping (typically on the order of 0.5 percent) was assigned across the profile to ensure the numerical stability of the solution.

Parameters for the Matasovic and Vucetic [1993] stress-strain constitutive model were developed by curve-fitting the modulus reduction and damping curves specified in Table 1.

The representative accelerograms, scaled to the design bedrock PHGA of 0.2 g, were applied to the transmitting base of the model in the D-MOD2000 analysis.

Results of Site Response Analysis

The results of site response analyses were monitored at the top of the Landfill and at the base of the site response model, i.e., just above crystalline bedrock. The results at the top of the Landfill were monitored in a form of Peak Ground Acceleration (PGA). The results at the base of the model were monitored in a form of peak average acceleration of sliding mass (k_{max}). The corresponding time history of k_{max} was saved and used as an input for seismic deformation potential evaluation of the Landfill.

The results of site response analyses indicate that design soil profile (site response model) does not amplify the design ground motions. Calculated PGA values at the Landfill surfaces are in the range of 0.19 to 0.21, i.e., almost identical to the input bedrock PHGA of 0.2 g. Calculated average acceleration of critical pseudostatic sliding mass is on the order of 0.08 g.

SEISMIC DEFORMATION ANALYSIS

Maximum seismically induced permanent displacements of the waste-fill-landfill subgrade system can be evaluated using Newmark-type [Newmark, 1965] seismic deformation analysis as coded in program SLIP2000 [Matasovic, 1996; 2007; www.GeoMotions.com]. However, as the calculated yield acceleration of the sliding mass (k_y = 0.28g) is significantly higher than the calculated peak average acceleration of sliding mass (k_{max} = 0.08 g), calculated permanent seismic displacements of waste fill and its subgrade, for deep-seated failure surfaces, are zero.

SUMMARY AND RECOMMENDATIONS

This letter report presents the results of the site-specific seismic site response and seismic deformation analyses performed by Geosyntec Consultants (Geosyntec) for a CCR landfill at the Kingston Fossil Plant (KIF) near Harriman, Tennessee. The analyses are based upon the probabilistic seismic hazard parameters evaluated based upon the 2008 edition of the USGS seismic hazard maps (2% PE in 50 years), design ground motions developed by AMEC Geomatrix [2009] for KIF, results of the site-specific geotechnical investigation by MACTEC [2005; 2006], and pseudostatic slope stability evaluation performed by Geosyntec [2010].

To evaluate the influence of local soil conditions and the Landfill on input (bedrock) ground motions, Geosyntec preformed a site-specific, seismic site response analysis. The results of this analysis indicate that the influence of local soil conditions and the Landfill in input ground motions is small.

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The results of site response analysis were further used to evaluate seismic deformation potential for deep-seated surfaces of Landfill presented in Figure 6 (cross section 4). The results of the seismic deformation analyses indicate that, as long as calculated yield acceleration of sliding mass is higher than 0.08 g, the maximum calculated permanent seismic displacements of sliding mass (i.e., Landfill Liner) will be zero. The lowest calculated yield acceleration by Geosyntec is 0.28 g.

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TABLES

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Table 1 – Material Parameters for Site Response Analysis

MATERIAL	UNIT WEIGHT	SHEAR WAVE VELOCITY	MODULUS REDUCTION AND DAMPING(1)
Landfill Cover	120 pcf	771 ft/s	Vucetic and Dobry [1991] (PI = 30)
Fly Ash	100 pcf	600 ft/s	Low Plasticity Silt (Ishihara et al, 1987)
Landfill Liner	125 pcf	984 ft/s	Vucetic and Dobry [1991] (PI = 50)
Medium Stiff to Stiff Silty Clay	120 pcf	712 ft/s	Vucetic and Dobry [1991] (PI = 100)
Soft Silty Clay	105 pcf	394 ft/s	Vucetic and Dobry [1991] (PI = 100)
Crystalline Bedrock	155 pcf	2,500 ft/s	Transmitting Base Assumed

Notes:
(1) Modulus reduction and damping curves listed here were curve-fitted to evaluate the parameters of the Matasovic and Vucetic [1993] constitutive model which is embedded in program D-MOD2000.

FIGURES

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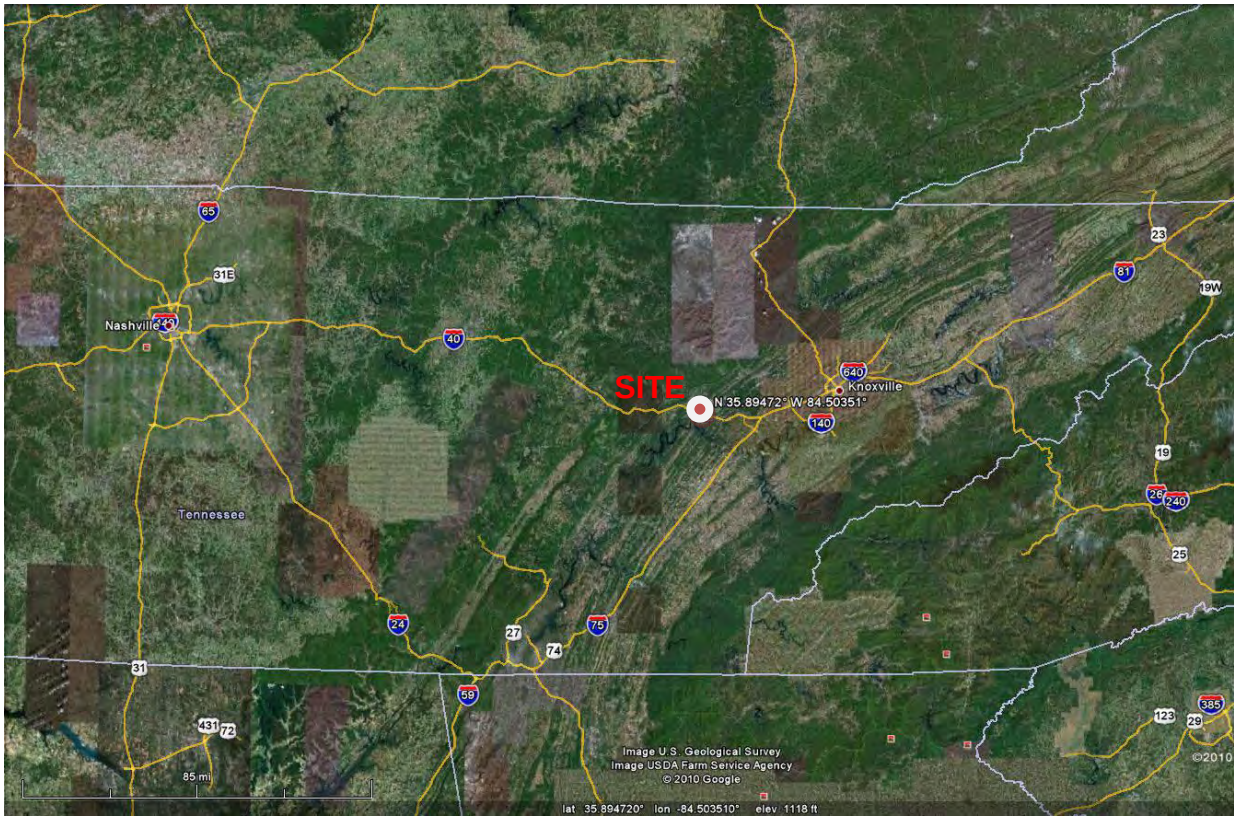


Figure 1. Site Location Map.

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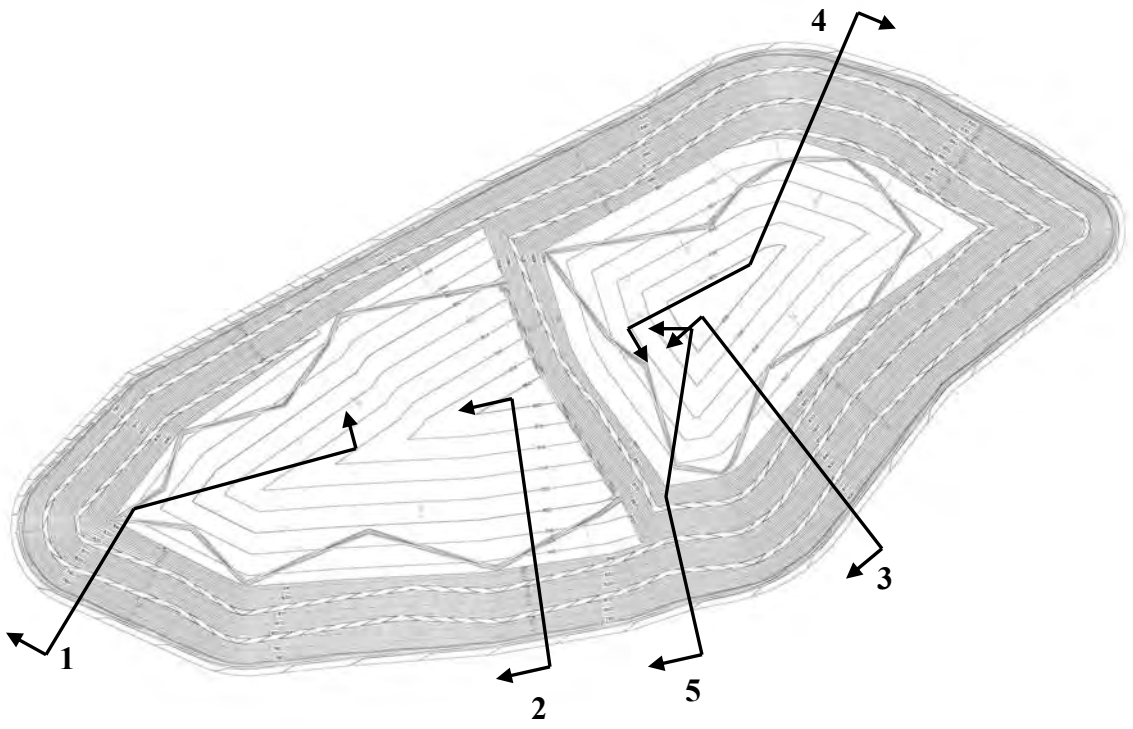


Figure 2. Plan View of the Landfill with indicated Cross Sections evaluated Pseudostatically by Geosyntec [2010].

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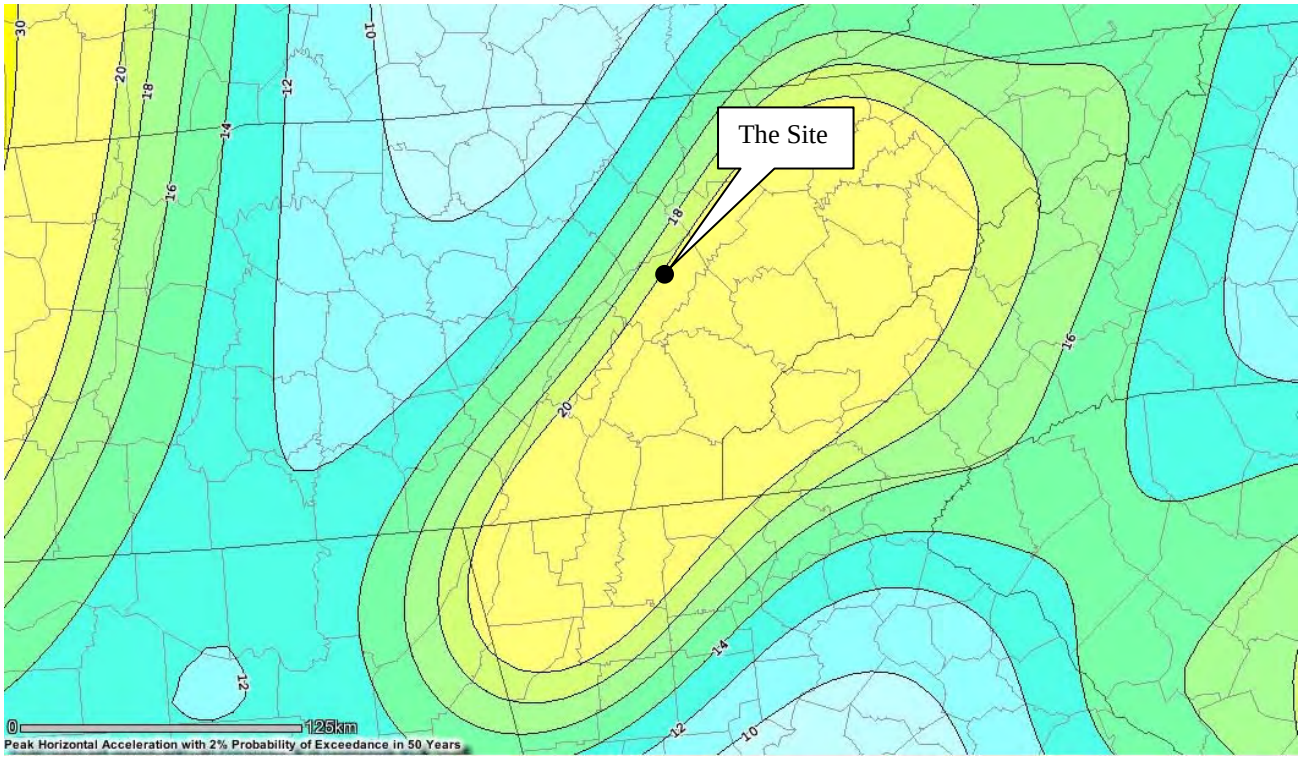


Figure 3. Contours of bedrock Peak Ground Acceleration (PGA) with a 2 Percent Probability of Exceedance in 50 Years. Adopted from USGS web site; USGS [2008].

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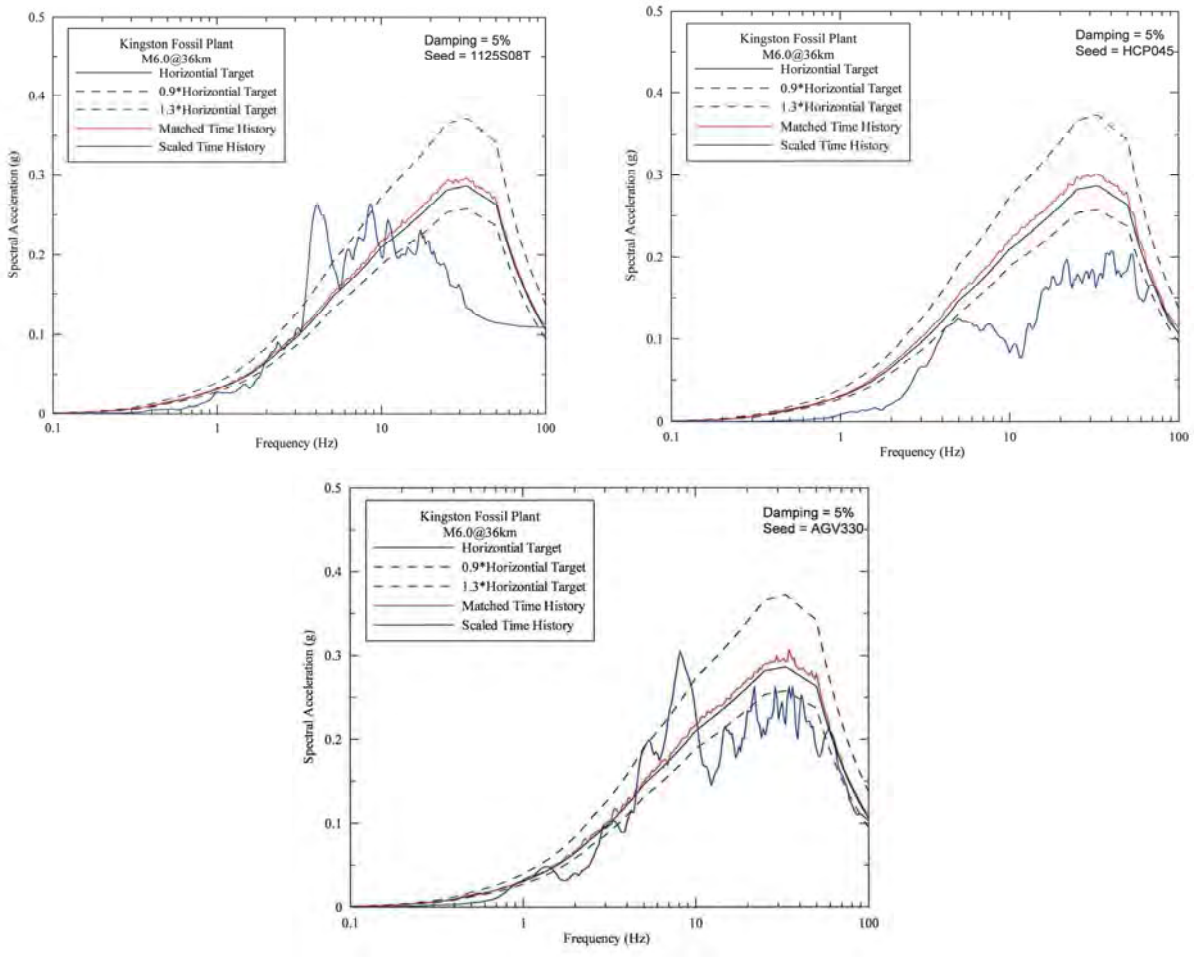


Figure 4. Acceleration Response Spectra of Design Ground Motions. Adopted from AMEC Geomatrix [2009].

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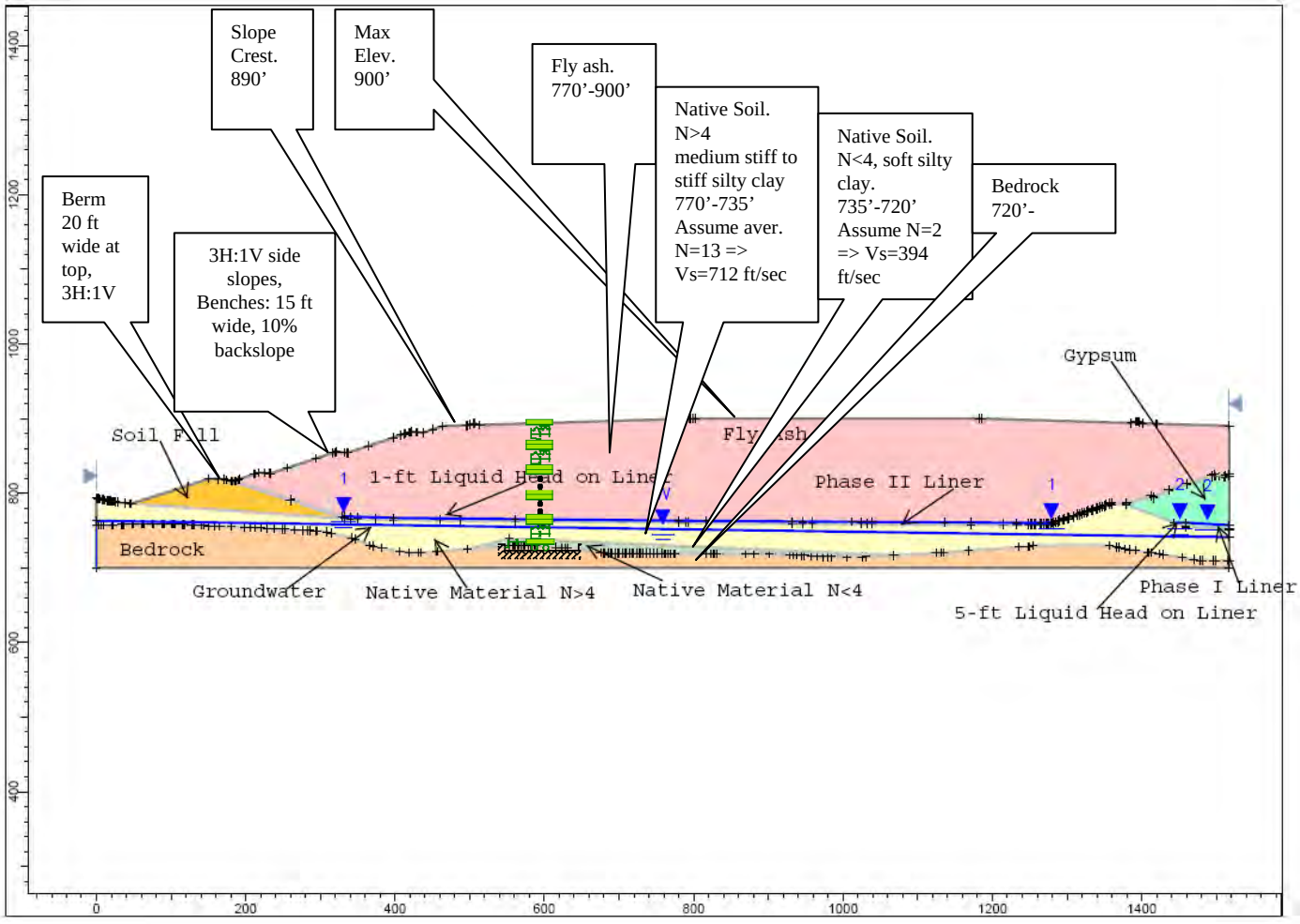


Figure 5. Annotated Cross Section 4 with Material Parameters used in Site Response Analysis. The Dynamic Site Response Model is also shown in this Figure. Adopted from Geosyntec [2010].

Notes:

1-Undrained shear strengths were derived from the CPT soundings based on the method developed by Schmertmann [1978]. An N_k factor of 19 was chosen to calibrate the CPT data to the triaxial data.

2-Undrained shear strengths based on CPT soundings in the soft native material (N<4) are indicated. An average undrained shear strength of 800 psf was chosen based on these data.

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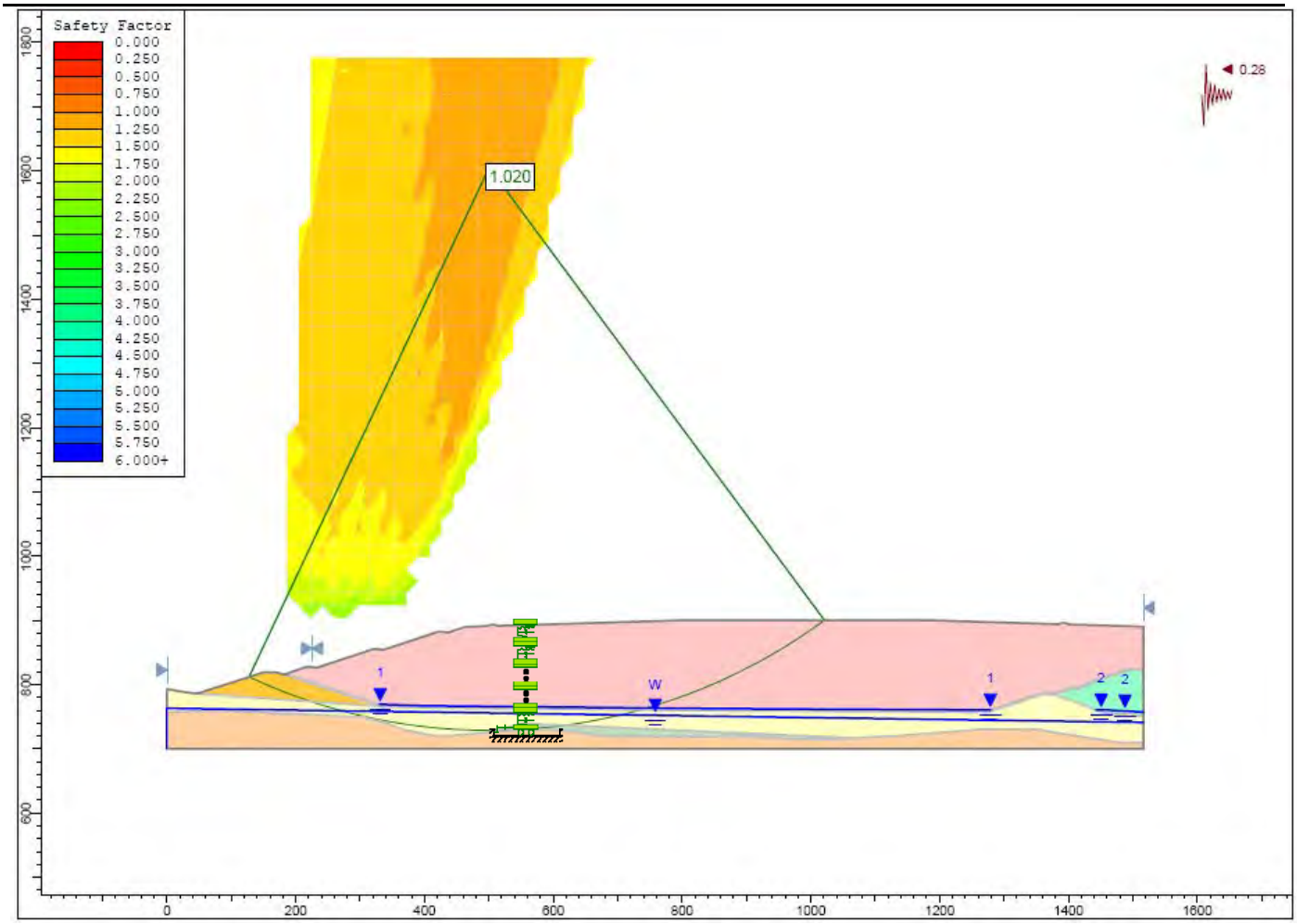


Figure 6. Calculation of Yield Acceleration for Section 4. The Dynamic Site Response Model is also shown in this Figure. Adopted from Geosyntec [2010].

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APPENDIX A

Document 16

Design Report for Dike C Buttress, Kingston Fossil Plant, Harriman, TN, Stantec Consulting Services, Inc., March 1, 2011

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VOLUME 2 OF 3:
FACILITIES DESIGN AND CONSTRUCTION REQUIREMENTS

TVA COAL COMBUSTION PRODUCTS MANAGEMENT PROGRAM

MASTER PROGRAMMATIC DOCUMENT

(REVISION 1.0)

Prepared for:

Tennessee Valley Authority (TVA)
1101 Market Street
Chattanooga, TN 37402-2801

December 7, 2009

URS

1375 Euclid Avenue
Cleveland, Ohio 44115
216-622-2400
Project No. 13813279

TABLE OF CONTENTS

INTRODUCTION 1

SECTION 1 DESIGN CRITERIA – REGULATORY STANDARDS 1-1

1.1 Facility Types 1-1
1.1.1 Landfills 1-1
1.1.2 Ash Impoundments 1-2
1.1.3 Surface Water Ponds 1-5
1.1.4 Leachate Storage Facilities 1-6
1.2 References and Sources 1-7
1.3 General Design Criteria 1-7
1.3.1 Siting Criteria 1-7
1.3.2 Site Investigations 1-9
1.3.3 Ash Characterization 1-23
1.3.4 Liner and Cap System 1-27
1.3.5 Volume and Life Calculations 1-32
1.4 Stability Analysis 1-34
1.4.1 Hydrostatic Uplift Analysis 1-34
1.4.2 Slope Stability Analysis 1-35
1.4.3 Settlement Analysis and Liner Strain Evaluation 1-49
1.4.4 Shallow Translational Failure Analysis 1-52
1.4.5 Anchor Trench Analysis 1-55
1.5 Leachate Management System 1-56
1.5.1 Leachate Generation Calculations 1-56
1.5.2 Pipe Sizing and Spacing and Leachate Drainage Layer 1-57
1.5.3 Leachate Storage Facility Sizing 1-60
1.5.4 Leachate Extraction System Sizing 1-62
1.5.5 Pipe Strength Calculations 1-64
1.5.6 Leachate Conveyance 1-65
1.5.7 Leachate Loadout, Treatment, and Disposal 1-68
1.6 Surface Water Run-off and Erosion Control 1-69
1.6.1 Soil Erosion Calculations 1-69
1.6.2 Surface Water Calculations 1-72
1.6.3 Sedimentation Basin Design 1-75
1.7 Impoundment Systems 1-77
1.7.1 Seepage Analysis 1-77
1.7.2 Impoundment Stability Analysis 1-81
1.7.3 Settlement Evaluations 1-86
1.8 Spillway / Drainage Systems 1-86
1.9 Ash Storage Pond and Liner Design Criteria 1-90
1.9.1 Recommendations for Dike Construction 1-90
1.9.2 Vertical Separation Distance above Groundwater 1-90
1.9.3 Liner System 1-91
1.9.4 Protective Drainage Cover 1-91
1.9.5 Liner Penetrations 1-92
1.9.6 Pond Discharge Structure 1-92
1.9.7 Splitter Dikes 1-92

US EPA ARCHIVE DOCUMENT

TABLE OF CONTENTS

1.9.8 Pond Size/Configuration 1-93
1.9.9 Future Dam Raisings 1-93
1.10 Visibility 1-93
1.11 Access, Perimeter, Haul, & Service Roads 1-94
1.11.1 Access Roads 1-94
1.11.2 Perimeter Roads 1-96
1.11.3 Haul Roads 1-97
1.11.4 Service Roads 1-97
1.12 Surveying 1-99

SECTION 2 REPORTING 2-1
2.1 Basis of Design Report 2-1
2.1.1 Definition 2-1
2.2 Calculation Documentation 2-2
2.3 Trigger Point Memorandum 2-3
2.4 Permit Regulatory Requirements Summary 2-4
2.5 Other Related Documentation 2-4

SECTION 3 REGULATORY REQUIREMENTS 3-1
3.1 Engineering Certification 3-1
3.1.1 Dry Ash Landfills 3-1
3.1.2 Wet Ash Ponds (Impoundments and Dams) 3-2
3.2 Borrow Source Requirements 3-3
3.2.1 Sampling 3-3
3.2.2 Testing 3-4
3.2.3 Stockpile Management 3-5
3.3 Agency Notifications and Restrictions 3-6
3.3.1 Dry Ash Landfill 3-6
3.3.2 Wet Ash Ponds (Impoundments and Dams) 3-8
3.4 General Construction Permits (Local, State, and Federal) 3-9
3.4.1 Dry Ash Landfills 3-9
3.4.2 Wet Ash Ponds (Impoundments and Dams) 3-11

SECTION 4 PROCEDURAL REQUIREMENTS FOR CONSTRUCTION 4-1
4.1 Overview of Bid, Evaluation, and Contract Award Process 4-1
4.1.1 Developing the Scope of Work 4-1
4.1.2 Developing a List of Bidders 4-2
4.1.3 Bidding 4-2
4.1.4 Developing Proposal Evaluation Criteria 4-2
4.1.5 Evaluating the Proposal 4-3
4.1.6 Awarding the Contract 4-3
4.2 Construction Management 4-3
4.2.1 Construction Manager – TVA Employee 4-3
4.2.2 Resident Engineer – TVA Employee / Third Party 4-4
4.3 CQA Consultant – Third Party 4-5

US EPA ARCHIVE DOCUMENT

TABLE OF CONTENTS

4.3.1 Certifying Engineer 4-5
4.3.2 Project Engineer 4-5
4.3.3 Field Technicians 4-5
4.3.4 Scheduler / Administrative Personnel 4-5
4.3.5 Soils / Geosynthetics Laboratory 4-5
4.4 Construction Documentation Preparation 4-6

SECTION 5 CLOSURE REQUIREMENTS 5-1
5.1 Closure Requirements 5-1
5.1.1 Dry Ash Landfills 5-1
5.1.2 Wet Ash Ponds (Impoundments) 5-2

SECTION 6 POST-CLOSURE REQUIREMENTS 6-1
6.1 Post-Closure Requirements 6-1
6.1.1 Dry Ash Landfills 6-1
6.1.2 Wet Ash Impoundments 6-1

SECTION 7 EMERGENCY OPERATIONS AND CONTINGENCIES 7-1
7.1 Emergency Operations and Contigencies 7-1
7.1.1 Fire 7-7
7.1.2 Power Outage 7-7
7.1.3 Leachate Outbreaks 7-7
7.1.4 Inclement Weather 7-8
7.1.5 Spills and Releases 7-8
7.1.6 Local Transportation Issues 7-8
7.1.7 Other Emergencies 7-8
7.2 Communication Procedures 7-10

US EPA ARCHIVE DOCUMENT

TABLE OF CONTENTS

List of Tables

Table 1.1.2-1: Comparison of State and Federal Regulations: Definition of "Dam"

Table 1.1.2-2: Comparison of State and Federal Regulations: Dam Hazard Classifications

Table 1.3.1-1: Laboratory Soil and Rock Test Procedures

Table 1.3.1-2: Investigation Methods and Objectives: Phase 2 Hydrogeologic Investigations and Landfill Siting Studies

Table 1.3.1-3: Generalized Representation of Relationship Between Site Complexity and Data Required

Table 1.4.2-1: Allowable Displacement

Table 1.7.2-1: Cases to be analyzed for static stability of impoundment systems

Table 1.7.2-2: Minimum factors of safety for static stability analyses of Cases to be analyzed for static stability of impoundment systems.

Table 1.9-1: Comparison of State and Federal Regulations: Spillway Design Flow Requirements

Table 1.9-2: Recommended TVA Spillway Criteria for Wet Ash Disposal Impoundments

Table 3.2.2-1: ASTM Specifications and Recommended Frequencies for Soil Borrow Source Material Investigations in Kentucky

Table 3.2.2-2: ASTM Specifications and Recommended Frequencies for Soil Borrow Source Material Investigations

List of Figures

Figure 1.1.1-1: CCP Landfill Typical Cross-Section

Figure 1.3.1-1: Alabama State Hydrogeologic Report Requirements

Figure 1.3.1-2: Kentucky State Hydrogeologic Report Requirements

Figure 1.3.1-3: Tennessee State Hydrogeologic Report Requirements

Figure 1.3.3-1: Liner System Typical Cross-Section

Figure 1.3.3-2: Alternate Closure Cap System Typical Cross-Section

Figure 1.4.2-1: Soil Liquefaction Classification

List of Appendices

Appendix 1: TVA Ash Impoundment Inventory Sheet

Appendix 2: Construction Site Organization Chart

Appendix 3: References

US EPA ARCHIVE DOCUMENT

TABLE OF CONTENTS

Master List of Acronyms

ACAA	American Coal Ash Association
ADEM	Alabama Department of Environmental Management
ALF	Allen Fossil Plant
AOS	Apparent Opening Size
ARAP	Aquatic Resources Alteration Permit
ARPA	Archaeological Resources Preservation Act
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BMP	Best Management Practice
BOD	Basis of Design
BRF	Bull Run Fossil Plant
BSL	Business Support Library
C&DD	Construction and Demolition Debris
C2P2	Coal Combustion Products Partnership
CBMPP	Construction Best Management Practices Plan
CBR	California Bearing Ration
CCP	Coal Combustion By Product
CEQ	Council on Environmental Quality
CFR	Code of Federal Regulations
CGP	Construction General Permit
COF	Colbert Fossil Plant
CPT	Cone Penetration Testing
CQA	Construction Quality Assurance
CS&PD	Clean Strategies & Project Development
CUF	Cumberland Fossil Plant
DEP	Department of Environmental Protection
DOT	Department of Transportation
DSWM	Division of Solid Waste Management
E&S	Erosion and Sediment
EA	Environmental Assessment
EAP	Emergency Action Plan
ECM	Erosion Control Matting

US EPA ARCHIVE DOCUMENT

EIS	Environmental Impact Statement
EPA	Environmental Protection Agency
ESA	Endangered Species Act
FEMA	Federal Emergency Management Agency
FERC	Federal Regulatory Energy Commission
FGD	Flue Gas Desulfurization
FHWA	Federal Highway Administration
FML	Flexible Membrane Liner
FOIA	Freedom of Information
FOTG	Field Office Technical Guide
GAF	Gallatin Fossil Plant
GAI-LAP	Geosynthetic Accreditation Institute Laboratory Accreditation Program
GVW	Gross Vehicle Weight
HASP	Health and Safety Plan
HDPE	High Density Polyethylene
HELP	Hydrologic Evaluation of Landfill Performance
HRIM	Haul Road Inspection Manual
IDW	Investigative derived wastes
IW	Industrial Waste
JOF	Johnsonville Fossil Plant
JSA	Job safety analysis
JSF	John Sevier Fossil Plant
KAR	Kentucky Administrative Regulations
KDEP	Kentucky Department of Environmental Protection
KIF	Kingston Fossil Plant
KPDES	Kentucky Pollutant Discharge Elimination System
KRS	Kentucky Revised Statutes
LLDPE	linear low density polyethylene
MS4	Municipal Separate Storm Sewer Systems
MSHA	Mine Safety and Health Administration
MSW	Municipal Solid Waste
MSWL	Municipal Solid Waste Landfill
NAVFAC	Naval Facilities Engineering Command

US EPA ARCHIVE DOCUMENT

NDSP	National Dam Safety Program
NEHRP	National Earthquake Hazards Reduction Program
NEPA	National Environmental Policy Act
NHPA	National Historic Preservation Act
NOAA	National Oceanic and Atmospheric Administration
NOI	Notice of Intent
NOT	Notice of Termination
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
O&M	Operation and Maintenance
O.C.	On Center
OEPA	Ohio Environmental Protection Agency
ONRW	Outstanding National Resource Water
OSHA	Occupational Safety and Health Administration
PAE	Program Administrator of Environmental
PAF	Paradise Fossil Plant
P.E.	Professional Engineer
PGA	Peak Ground Acceleration
PMP	Probable Maximum Precipitation
PPE	Personal Protective Equipment
PRB	Powder River Basin
PVC	Polyvinyl Chloride
QA/QC	Quality Assurance/Quality Control
QCP	Qualified Credentialed Professional
RCRA	Resource Conservation and Recovery Act
RFP	Request for Proposal
RHO&M	Routine Handling, Operations, and Maintenance
RLS	Registered Land Surveyor
RO-DS	Reservoir Operations, Dam Safety
RSL	Recompacted Soil Liner
RUSLE	Revised Universal Soil Loss Equation
SAF	Shawnee Fossil Plant
SCD	Soil Conservation District

US EPA ARCHIVE DOCUMENT

SCS	Soil Conservation Service
SEM	Scanning Electron Microscope
SPT	Standard Penetration Test
SSD	Stopping sight distance
SWCC	Soil and Water Conservation Committee
SWPPP	Stormwater Pollution Prevention Plan
T&O	Technology and Operations
TADS	Training Aids for Dam Safety
TCA	Tennessee Code Annotated
TCLP	Toxicity Characteristic Leaching Procedure
TDEC	Tennessee Department of Environment and Conservation
TDH	Total Dynamic Head
TPE	Third Party Engineering
TR	Technical Release
TRM	Turf Reinforced Matrix
TSS	Total Suspended Solids
TVA	Tennessee Valley Authority
UIP	Underground Injection Permit
USEPA	United States Environmental Protection Agency
USACE	United States Army Corps of Engineers
USCS	United Soil Classification System
USDA	United States Department of Agriculture
USEPA	United States Environmental Protection Agency
USET	United Southeastern Tribes
USGS	U.S. Geological Survey
USWAG	Utility Solid Waste Activities Group
VOC	Volatile Organic Compound
WCF	Widows Creek Fossil Plant

US EPA ARCHIVE DOCUMENT

The Programmatic Document was designed as a management tool to provide guidance for the effective management and planning of Coal Combustion By Product (CCP) streams generated by Tennessee Valley Authority (TVA). The Document addresses engineering design, environmental compliance, safety, training, data management, procedural requirements for construction, inspection, monitoring, and operation of CCP facilities, and applies to all CCP materials generated by TVA. Existing facilities where these materials are managed includes ash ponds (or impoundments), wet stacks, dry landfills, surface water ponds, and chemical ponds. New facilities will include ash ponds, dry landfills, surface water ponds, leachate ponds, and chemical ponds. The Programmatic Document only applies to chemical ponds with respect to structural integrity.

Volume 2 of the Programmatic Document establishes design criteria for all elements of CCP facilities, and includes a Master and Abbreviated version. This volume of the Programmatic Document, **Master Volume 2**, is generally organized to:

1. Define CCP facility types;

2. Define key design elements of CCP facilities;

3. Present minimum state and federal standards and identify industry standards associated with each design element ; and,

4. Provide a selected design standard for each design element.

Similarly, the Abbreviated Volume 2 contains the same information as the Master Version with the omission of the minimum state and federal standards, thereby providing the selected standard for each topic. The Programmatic Document shall be periodically reviewed to reflect the most current CCP streams, design references, regulations and management protocols. The Document should be considered current as of the revision date and number provided on the Document cover.

US EPA ARCHIVE DOCUMENT

1.1 FACILITY TYPES

This section describes the facility types discussed in Volume 2 including landfills, ash impoundments, surface water ponds, and leachate storage facilities.

1.1.1 Landfills

Definition

Landfills are defined for the purpose of this Document as an area of land or an excavation in which wastes are placed for disposal or temporary storage and that is not a land application unit, surface impoundment, injection well, or waste pile (any non-containerized accumulation of solid, non-flowing waste that is used for treatment or storage). For TVA purposes, landfills will be used for deposition of CCPs in solid form. Landfills for CCP can be used for permanent disposal or temporary deposition. When landfills are used for temporary deposition, it is to enable beneficial reuse of the CCP upon its removal.

There are several design and construction components to landfills considered standard across the solid waste management industry. These generally include a bottom liner system, leachate collection system, leachate conveyance system, leachate storage or treatment system, waste disposal area, final closure cap system, surface water management system, haul and service roads, groundwater management system, and operational and service infrastructure. The design requirements for each of these components are described in more detail in Volume 2 Section 1 of this Document. A generalized schematic of a landfill, which illustrates these key components, is presented in Figure 1.1.1-1.

Landfill Regulations

Following the Ruling on the Bevil Amendment in 1980, large volume wastes generated from the combustion of coal were determined to be exempt from federal hazardous waste regulations under Subtitle C of RCRA section 3001(b)(3)(C). CCPs are not currently regulated at the federal level and therefore, regulations for disposal of CCPs are administered at the State level.

As of the date of this Document, USEPA is in the process of establishing/negotiating an appropriate classification and regulatory designation for CCP including Federal regulations for management. It is anticipated that the regulations for management of CCPs at the Federal level will be put in-place and will require more stringent regulation of CCPs at the State level. This Document is limited to the current status of the regulations, and does not speculate on the content of the future Federal and state-specific regulations.

Alabama: In Alabama, CCPs are specifically exempted from the definitions of “solid waste” and “industrial waste”. Bottom ash and fly ash are defined as a “Special Waste” per Alabama Department of Environmental Management (ADEM) 335-13-1-.03(134). However, ashes from combustion of fossil fuels at electric or steam generating facilities are excluded per ADEM 335-

US EPA ARCHIVE DOCUMENT

13-1-.03(13). Therefore, there is no landfill permitting requirements for CCP materials in Alabama.

Kentucky: CCPs are defined as a "Special Waste" per KRS Chapter 224.50-760(1)(a) and are therefore treated separately than solid wastes. The requirements for management and disposal of special wastes are presented in 401 KAR Chapter 45 and the technical and operating requirements for special waste landfills are specified in 401 KAR 45:110. There are very limited requirements specified in 401 KAR 45:110 with respect to specific design, construction and/or operational requirements for Special Waste landfills. Specifics regarding new facility permitting are provided in 401 KAR 45:30.

Tennessee: Disposal of CCPs is permitted in Class II (non-hazardous industrial wastes, commercial wastes and fill) Disposal Facilities, as defined in TDEC 1200-1-7-.01(3). Management of CCP is specifically regulated in Tennessee as presented in TDEC 1200-1-7-.04, which has specific design, permitting, construction and operation requirements for Class II Disposal Facilities.

1.1.2 Ash Impoundments

Definition

Ash impoundment can be defined as a collection of water, wastewater, or liquid-borne materials that is contained by an artificial structure (dam), into which CCPs are placed for disposal or temporary storage. All ash impoundments shall be managed by the TVA Dam Safety Program.

For reference, Table 1.1.2-1 compares the state and federal definitions of dams. Table1.1.2-2 compares the state and federal hazard classifications.

Table 1.1.2-1
Comparison of State and Federal Regulations: Definition of "Dam"

Criteria	Alabama	Kentucky	Tennessee	Federal[2]
Any artificial barrier that impounds water	Yes	Yes	Yes	Yes
Impounds wastewater or liquid-borne materials	Yes	No	No	See tailings dams, below
Height[3]	≥25 feet	≥25 feet	≥20 feet	≥25 feet
Storage Volume[4]	≥50 acre-feet	≥50 acre-feet	≥30 acre-feet	≥50 acre-feet
Other criteria that trigger classification as a dam:	Probable **loss of human life** due to failure or improper operation regardless of height or storage volume	N/A	N/A	This lower size limitation should be waived if there is potentially significant downstream hazard. In addition to conventional structures, this definition specifically includes "tailings dams", embankments built by waste products disposal and retaining a disposal pond.
	Probable **loss of critical infrastructure** due to failure or improper operation regardless of height or storage volume [1]	N/A	N/A	

US EPA ARCHIVE DOCUMENT

Note 1: If the dam and/or the appurtenant utility and ash handling facilities are considered to be critical infrastructure, then any artificial barrier that impounds water, wastewater or liquid-borne materials meets the definition of a dam.
Note 2: Federal criteria referenced from the Federal Guidelines for Dam Safety, prepared by the Interagency Committee on Dam Safety, U.S. Department of Homeland Security, Federal Emergency Management Agency, June 1979, Reprinted April 2004.
Note 3: Federal and state guidelines measure height of dam from the top of the dam crest to the lowest point where the original streambed intersects the downstream toe, or the lowest point along the toe.
Note 4: Federal and state guidelines measure storage capacity at the top of the dam crest, i.e., the maximum possible impounded capacity, irrespective of the elevation of spillways or outlet works.

Table 1.1.2-2
Comparison of State and Federal Regulations: Dam Hazard Classifications

Category	Alabama[1]	Kentucky[1]	Tennessee[1]	Federal
High Hazard	High Hazard Potential Dam: Failure or improper operation will cause probable loss of human life.	Class C: Failure would cause loss of life or serious damage to homes, commercial buildings, utilities, highways or railroads.	Category 1 : Failure would probably result in: loss of human life; excessive economic loss due to damage to downstream properties; excessive economic loss, public hazard, or public inconvenience due to loss of impoundment and/or damage to roads or any public or private utilities.	High Hazard Potential Dam**:** Failure or misoperation will probably cause loss of human life.
Moderate Hazard	Moderate Hazard Potential Dam: Failure or improper operation results in no probable loss of human life but will cause probable loss of critical infrastructure.	Class B: Failure would cause significant damage to property and project operation, but loss of life is not envisioned.	Category 2 : Failure may damage downstream private or public property, but such damage would be relatively minor and within the general financial capabilities of the dam owner. Public hazard or inconvenience due to loss of roads or any public or private utilities would be minor and of short duration. Chances of loss of life would be possible but remote.	Significant Hazard Potential Dam: Failure or improper operation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns.
Low Hazard	Low Hazard Potential Dam: A dam not assigned the moderate or high hazard potential classification.	Class A: Failure results in loss of structure itself but little or no additional damage to other property.	Category 3: Failure may damage uninhabitable structures or land but such damage would probably be confined to the dam owner's property. No loss of human life would be expected.	Low Hazard Potential: Failure or improper operation results in no probable loss of human life and low economic and/or environmental losses. Losses are principally limited to the owner's property.

1. According to the TVA Dam Safety Program, state dam safety regulations do not apply. Rather, the TVA Dam Safety Program follows Federal programs in these regards. The following sections for each state are therefore provided for reference only.

TVA operates coal-fired power plants in Alabama, Kentucky and Tennessee. Dam safety regulations define a "dam" similarly in all three states and federal programs. Once an ash impoundment is closed (in accordance with the provisions herein), it is no longer considered a possible dam structure.

US EPA ARCHIVE DOCUMENT

US EPA ARCHIVE DOCUMENT

TVA Ash Impoundment Inventory

CCP Engineering will use the TVA Ash Impoundment Inventory Sheet (Appendix 1) to collect information on all wet CCP disposal impoundments, such as location, power plant served, the name of stream impounded (if applicable), USGS quadrangle sheet that shows the site location, original designer, construction contractor, year constructed, height, length, impoundment capacity and related information. Completed inventory sheets will be forwarded to the TVA Reservoir Operations, Dam Safety Group, who will use the information to determine:

- Does the impoundment meet the definition of a dam?
- What is its probable hazard category?
- Should the structure be managed in accordance with the TVA Dam Safety Program?

As appropriate, TVA Reservoir Operations, Dam Safety will accept the facility into its ongoing dam safety program, and will decide what if any investigations or analyses such as dam break inundation mapping are needed to definitively establish the hazard category of the dam.

TVA Reservoir Operations, Dam Safety will assume responsibility for dam safety activities for the ash impoundment classified as a dam, including scheduling of an initial dam inspection, scheduling and frequency of future dam inspections, and the nature and extent of additional investigations that may be required. This may include items such as spillway capacity evaluations, slope stability analyses, seismic evaluations, seepage analyses or other technical activities that are found to be necessary. TVA Reservoir Operations, Dam Safety will also assume responsibility for making repairs, improvements or modifications to the dam as required to promote continued safe operation of the impoundment.

Ash Impoundment Regulations

According to the TVA Dam Safety Program, state dam safety regulations do not apply. Rather, the TVA Dam Safety Program follows Federal programs in these regards. The following sections for each state are therefore provided for reference only,

Alabama: Alabama currently has no dam safety regulations. However, a bill entitled "Alabama Dam Inventory and Classification Act" is under consideration of the state legislature as of February 2008.

Kentucky: Dam safety regulations are in KAR Chapter 4, Title 401, Water Resources, last amended in 1990.

Tennessee: Regulations for dam safety are found in the TDEC, Division of Water Supply, Chapter 1200-5-7, effective February 2001.

Federal: Guidelines for dam safety are found in Federal Guidelines for Dam Safety, prepared by the Interagency Committee on Dam Safety, U.S. Department of Homeland Security, Federal Emergency Management Agency (FEMA), June 1979. Water quality from ash impoundment

discharges is regulated under the NPDES regulations, under the terms of the facility's NPDES permit.

1.1.3 Surface Water Ponds

Definition

Surface water ponds related to management of stormwater discharges from landfills can be defined as earthen structures used to temporarily retain and/or detain surface water (non-contact water) away from the limits of the landfill or other disturbed area (clean earth disturbances) within the landfill facility. This non-contact water is surface water that has not come into contact with the waste materials and therefore is not considered leachate. Surface water ponds are used for temporarily retaining and/or detaining flows from active construction areas to allow for sediment to settle out of the surface water prior to discharge. In this case, surface water ponds are also referred to as sediment basins. Surface water ponds are also used as permanent stormwater management control structures for stormwater conveyed from a final closure or post-construction area.

Surface water ponds generally consist of an excavated area surrounded by a containment berm that may be formed by either excavated soils (cut) or embankment soils (fill). Inlets and outlets for surface water ponds can be via inlet ditches or piped structures, both typically operating by gravity flow. Riser structures may be used to control the rate and quantity of discharge from the ponds. Typically, surface water ponds are not constructed with bottom liner systems, since the surface water is not contaminated and should not pose a risk to groundwater. Design requirements for surface water ponds are discussed in Volume 2, Section 1 of this Document

Surface Water Pond Facility Regulations

State/Federal: The requirements for the design, use, and monitoring of surface water ponds will be based on the requirements included in the approved site-specific NPDES permit administered by the state regulatory agency (ADEM, KDEP or TDEC). The specified state regulatory agency is responsible for administering the state NPDES and stormwater management program, which may closely follow the federal NPDES requirements. The state NPDES programs regulate the surface water discharge limits from contact water (leachate) and non-contact water (stormwater), or a combined flow of leachate and stormwater.

The design guidance provided in either state specific guidance manuals (available in Alabama and Tennessee, but not in Kentucky), or in the federal Natural Resources Conservation Service (NRCS) standards of the *Field Office Technical Guide* (FOTG) – *Section IV – Practice Standards and Specifications*, are used as the basis for developing surface water pond practices, as well as other best management practices (BMP), for inclusion in the site-specific stormwater pollution prevention plan (SWPPP). The development and submittal of the SWPPP is a regulatory requirement of the general construction permit under state NPDES programs.

Sediment control basins, along with other erosion and sediment (E&S) controls are also regulated at the state level via the state's NPDES program. However, many counties have their

US EPA ARCHIVE DOCUMENT

own Soil Conservation Districts (SCDs), which are overseen by the NRCS of the United States Department of Agriculture (USDA). The County SCDs also typically offer technical guidance in the development of stormwater and E&S control design and associated planning. Unless granted regulatory or enforcement authority by the State or County, the SCDs and their technical guidance documents do not have the force of law, rather provide guidance for preventing or minimizing the related problems of stormwater management and E&S control.

While regulations for stormwater management are administered at the state level, certain counties or municipalities – typically ones that are closest to or more directly affect significant or sensitive water bodies or locations defined within Municipal Separate Storm Sewer Systems (MS4) regions – may have additional stormwater or E&S requirements. Some of these additional requirements may include detailed design submittals, increased requirements for use of various BMPs, different design storm events, etc. For this reason, it is necessary that the design engineer/consultant preparing the permitting and design documents check with the applicable county or municipality where the facility is sited to determine if there are additional regulatory requirements specific to the facility location.

1.1.4 Leachate Storage Facilities

Definition

Leachate storage facilities are those facilities that temporarily store leachate produced in the landfill or ash impoundment prior to final treatment or disposal. Leachate is liquid that is generated when the materials (CCPs) are compressed. The liquid originates from the material itself, or when surface water infiltrates through the landfill cover materials and/or CCPs. In addition, any surface water that comes in contact with the CCPs is also considered leachate and shall be treated as such. Of the three states, only Kentucky has defined leachate. Kentucky defines leachate as any liquid, including any suspended components in the liquid that has percolated through or drained from waste materials.

Leachate storage facilities typically consist of either an above or below ground leachate storage tank or a leachate storage pond. Leachate can be conveyed to the storage facility by gravity or by forcemains from the operational cells of the landfill.

Key Components of Leachate Storage Facilities

Leachate storage tanks are typically constructed of specialized plastics, fiberglass or metallic with specially treated internal coatings or glass lining to limit corrosion and be compatible (nonreactive) with the chemicals in the leachate. The storage tanks can be single-walled or double-walled. If the tank is single-walled, it is required that a system for secondary containment is included (vault for below grade tanks or perimeter containment for above ground tanks) to reduce the potential for groundwater contamination. The storage tank typically consists of an inlet conveying leachate from a collection manhole with control valving, and an outlet. Leachate is typically pumped from the leachate storage tank to a loadout facility (for truck hauling), treatment system, or connection to a sanitary sewer.

US EPA ARCHIVE DOCUMENT

Leachate storage ponds are excavated ponds constructed with a bottom liner system and groundwater monitoring wells to enable monitoring of any potential leaks through from the liner system. The storage pond typically consists of an inlet conveying leachate from a collection manhole with control valving. Leachate is typically pumped or otherwise conveyed from the leachate storage pond to a loadout facility (for truck hauling), treatment system, connection to a sanitary sewer, or a receiving surface water body under an NPDES permit.

Leachate Storage Facility Regulations

As discussed in the Landfill Regulations section (Section 1.1.1), Federal regulations do not govern the design and construction of leachate storage facilities. The State regulations for leachate storage facilities are the same as specified in Section 1.1.1.

1.2 REFERENCES AND SOURCES

References are provided in Appendix 3.

1.3 GENERAL DESIGN CRITERIA

1.3.1 Siting Criteria

Definition

Siting criteria are the exclusionary requirements/guidance to regulate the permitting and ultimate construction of a facility. Siting criteria often include items such as prohibited areas and buffer zones/distances that are to be maintained with respect to the location of a landfill.

Minimum Design Standards

Alabama: In Alabama, CCPs are specifically exempted from the definition of solid waste and industrial waste. Bottom ash and fly ash are defined as a "Special Waste" per Alabama Department of Environmental Management (ADEM) 335-13-1-.03(134). However, ashes from combustion of fossil fuels at electric or steam generating facilities are excluded per ADEM 335-13-1-.03(13). As such, there are no has no specific siting criteria requirements associated with CCP materials

Kentucky: In Kentucky, CCP wastes are considered special wastes, regulated under 401 KAR 45. 401 KAR 45:130 provides siting requirements for new special waste landfills and horizontal expansions to permitted waste boundaries. Requirements include maintaining buffer zones, restricting disposal in floodplain areas, and conforming to site suitability criteria. Site suitability criteria includes satisfying upper aquifer monitoring and corrective action requirements.

Tennessee: Siting criteria requirements for Class I and II facility are provided in TDEC Rule 1200-1-7-.04(2). Siting criteria to be considered includes impact to endangered species, floodplain location, wetland restrictions, karst terrain, seismic impact zones, unstable areas, and maintaining buffer zones.

US EPA ARCHIVE DOCUMENT

US EPA ARCHIVE DOCUMENT

Selected Design Standards

For a new landfill, all applicable state regulatory siting criteria shall apply. However, in states where specific siting criteria is not defined, landfills shall be located in compliance with the following items:

- Facilities shall not be located in a manner that contributes loss of endangered species, or adverse modification of their habitat.
- Facilities must not be located in a 100-year floodplain unless it can be proven that the location will not restrict the flow of the 100-year flood nor reduce temporary floodplain storage capacity, and the facility is designed, constructed, operated, and maintained to prevent washout of any waste.
- Facilities shall not be located in wetlands, unless the applicant can demonstrate under USEPA Clean Water Act, Section 404, that a practical alternative does not exist. Additionally, the applicant shall demonstrate that construction of the landfill will not cause or contribute to violations of any applicable state water quality standard; or, violate any applicable toxic effluent standard or prohibition under Section 307 of the Clean Water Act. The applicant shall also demonstrate that the facility will not cause or contribute to significant degradation of wetlands by establishing the integrity of the landfill and the ability to protect ecological resources. Lastly, as required under Section 404 of the Clean Water Act the applicant shall show that steps have been taken to attempt to achieve no net loss of wetlands.
- For a facility proposed in an area of highly developed karst terrain, the applicant must demonstrate that there is no significant potential for surface collapse, and the location will not cause any significant degradation to the local ground water resources.
- Facilities shall not be located within 200 feet of a fault that has had displacement in Holocene time, unless the applicant demonstrates that an alternative setback distance of less than 200 feet will prevent damage to the structural integrity of the landfill unit and will be protective of human health and the environment.
- Facilities shall not be located in seismic impact zones, unless the applicant demonstrates that all containment structures, including liners, leachate collection systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth materials.
- Facilities proposed for locations in an unstable area must demonstrate that engineering measures have been incorporated to ensure integrity of landfill components. The demonstration shall consider the following factors: soil conditions that may result in significant differential settling; geologic or geomorphologic features; and man-made features or events (both surface and subsurface).
- Additionally, the following buffer distances/zones shall be maintained when siting a landfill:
    - 100 feet from all property lines;
    - 500 feet from all residences, unless the owner of the residential property agrees in writing to a shorter distance;

- 500 feet from all wells determined to be downgradient and used as a source of drinking water by humans or livestock;
- 200 feet from the normal boundaries of springs, streams and lakes; and
- 50 feet from the property line for all constructed appurtenances.

1.3.2 Site Investigations

Site investigations for landfill and impoundment systems shall include geotechnical and hydrogeological explorations. These investigations include exploratory drilling and sampling, field testing, and laboratory testing, and are necessary to establish the subsurface soil, rock, and groundwater conditions to support the facility design. Often, similar drilling and sampling tasks are required for both types of investigations. Costs and schedule can usually be optimized if both investigations are coordinated in conjunction with one another, so that duplication of tasks is eliminated or minimized. The Engineer should be cognizant of this during planning of these investigations.

1.3.2.1 Geotechnical Investigations

Definition

The objective of the geotechnical investigation is to collect sufficient information to define the subsurface conditions at a site as it relates to facility design and construction, and use this information to provide recommendations and perform geotechnical analyses to support the facility design. Important aspects of the geotechnical investigation include defining the stratigraphy, extent and engineering properties of subsurface soils and rock influenced by the facility, groundwater conditions on the site, and the nature and extent of any geologic hazards (such as karst formations and mining activity) that exist on the site as they may influence facility construction and operation.

Minimum Design Standards

Minimum design standards for landfills and ash impoundments on a state-by-state and federal level are described below.

Alabama: Alabama has no specific requirements for geotechnical investigations associated with landfills.

Alabama provides no specific requirements for geotechnical investigations related to impoundment systems.

Kentucky: In Kentucky, CCPs are generally considered special wastes, which are regulated under 401 KAR 45. There are no specific requirements for geotechnical investigations supporting special waste facilities, other than a general statement in 401 KAR 45:110, Section 1 (5) which states that the engineering design for a special waste landfill shall consider the properties of the soil underlying the facility. Requirements for geotechnical investigations for new municipal solid waste facilities (contained wastes) are given in 401 KAR 47:170, Section 3. The requirements for geotechnical investigations for contained waste facilities include a

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minimum of nine soil borings per 1000 ft x 1000 ft area of the facility (roughly one boring per 2.5 acres), and a minimum of four rock core borings per facility, with the requirement of an additional rock core boring for each 25 acre increment for facilities whose area is greater than 50 acres. Requirements for reporting information obtained from geotechnical investigations are given in 401 KAR 47:180, Section 6.

Kentucky provides no specific requirements for geotechnical investigations related to impoundment systems.

Tennessee: Tennessee has no specific requirements for geotechnical investigations associated with landfills. Requirements for hydrogeologic investigations are given in 1200-1-7-.04(9)(a).

Tennessee provides guidance for hydrogeologic investigations in its Technical Guidance Document 001, "Hydrogeologic Investigation Guidance Document", January 1, 1993. This document does not make specific reference to geotechnical investigations. Major requirements listed in the document include:

- Borings shall be drilled on an equivalent triangular grid pattern having 200 foot spacing between holes. Holes are to be drilled to a depth of 20 feet below the bottom of the clay liner or to the top of rock, whichever is shallower.
- At least one boring per site or per ten acres of the site shall be drilled to a depth of at least 70 feet below the top of the proposed clay liner or at least 20 feet into bedrock, whichever is shallower. Continuous bedrock cores are required in these borings.
- A complete grain size analysis, natural moisture content, and Atterberg limit tests, shall be performed on a representative sample from each significant stratum encountered.
- For each three acres proposed for landfilling a minimum of one hydraulic conductivity test (ASTMD D5084) shall be conducted on a Shelby tube sample at a random sampling interval approved by the Division. The samples shall be taken from within the proposed geologic buffer.

Tennessee provides no specific requirements for geotechnical investigations related to impoundment systems.

Selected Design Standard

The geotechnical investigation shall be performed prior to detailed design and construction of the landfill or impoundment site and shall be planned and conducted by or under the supervision of a professional engineer specializing in geotechnical engineering. In general, the investigation shall include the following tasks and considerations:

- A study of the general geologic setting of the site area shall be made during planning of the investigation. Publicly available information such as bedrock geology and topographic maps, karst maps, groundwater resource maps and reports, mine maps and soil surveys should be consulted. This study should be used to gain an understanding of the local geology and to identify potential geologic hazards that may be present in the area of the site. Additionally, a thorough search of existing documents, records, plan drawings, etc related to the site should be performed, including research into the past history and use of the site.

US EPA ARCHIVE DOCUMENT

- For all sites, a site reconnaissance shall be performed prior to beginning the geotechnical investigation. Areas with karst formations or mining activity should be evaluated by qualified geologists knowledgeable in the identification of karstic/mine subsidence features. Subsidence features (such as caves, sinkholes, losing streams, etc) on site should be thoroughly described and mapped as part of the reconnaissance. Consideration should be given to performing special investigations to delineate the location, extent, and most appropriate mitigation techniques for subsidence features as deemed appropriate by the Geotechnical Engineer. Such investigations may include the use of geophysical and/or electromagnetic surveys, dye tracing, etc.
- Consideration should be given to performing a preliminary investigation prior to implementing the full geotechnical investigation. The preliminary investigation may consist of several borings performed across the site, to establish the basic characteristics of the subsurface. Information gained from this investigation can be used in planning for the main geotechnical investigation that is to follow – such as boring locations and depth, location and characteristics of geologic strata of interest, etc.
- The scope of the geotechnical investigation should include, at a minimum: soil sampling using the Standard Penetration Test (SPT) method, per ASTM D 1586; collection of undisturbed samples when soft, cohesive soils are encountered per ASTM D 1587; and, where bedrock is encountered within a depth corresponding to 50 ft below the base of the facility, rock coring should be performed per ASTM D 2113. In areas with karst or mining activity, rock coring may be required to depths exceeding 50 ft. If the site conditions warrant and if deemed appropriate by the Geotechnical Engineer, other sampling and field testing methods, such as Cone Penetration Testing (CPT), dilatometer or pressuremeter testing, and shear vane testing, may also be considered in addition to the above testing methods. Appropriate observation of groundwater levels during and after drilling should be performed. Monitoring wells or piezometers should be installed and monitored as necessary to establish the position of the water table, as necessary to support the geotechnical design of the facility.
- Soil borings should be drilled at the following minimum frequency:

  Landfills: Minimum of one boring per four acres, unless a greater frequency is specifically required by the state or if subsurface conditions such as karst or mining activity dictate otherwise. To the extent practicable, borings should be located on a uniform grid pattern across the area of the site within the limits of proposed CCP placement. Additional borings should be drilled at locations of proposed structure foundations, haul roads, or other site features, at the discretion of the Geotechnical Engineer.

  Impoundment Systems: Borings should be drilled at a minimum spacing of 200 ft on center (O.C.) along the centerline of the alignment of the impoundment structure (i.e., earth dam, dike, etc.). For impoundment structures with a total base width exceeding 100 ft, sets of two borings shall be drilled at each 200 ft interval along the centerline. One boring shall be located normal to the centerline on the downstream side and one boring shall be located normal to the centerline on the upstream side. The two borings should be roughly equidistant from the centerline. Borings sufficient to characterize the soil

US EPA ARCHIVE DOCUMENT

deposits underlying the area within the impoundment should also be performed, at the discretion of the Geotechnical Engineer.

- Soil borings should be drilled to sufficient depth to identify and characterize critical subsurface materials that will be influenced by the facility or may affect the stability of the facility. The final depth of borings will depend on site-specific conditions, and should be determined by the Geotechnical Engineer. Minimal guidelines for soil boring depths are as follows:

  Landfills: Borings should extend to the top of competent bedrock or other competent material that will resist substantial settlement and has enough strength to preclude shear failure through it.. The investigation should further include an appropriate amount of bedrock coring to characterize the strength and other engineering properties of the competent bedrock and assess karst/mining activity.

  Impoundments: Borings should extend to the top of competent bedrock or other competent material that will resist substantial settlement and has enough strength to preclude shear failure through it. Borings shall also extend to a stratum that is considered to be impervious or minimally pervious with respect to seepage beneath the impoundment. Where bedrock is encountered, the investigation should further include an appropriate amount of rock coring to characterize the strength and other engineering properties of the competent bedrock and assess karst/mining activity.

- For projects where high seismicity is a controlling aspect of design, additional field testing to determine dynamic properties of the foundation soils may be warranted. This may include downhole seismic testing per ASTM D 7400, crosshole testing per ASTM D 4428, or seismic CPT. Such testing should be performed under the guidance and supervision of an experienced geotechnical engineer or geophysicist.

- For impoundment projects, where the seepage characteristics of the foundation soils are required, field tests to determine permeability in-situ such as pump tests, slug tests, or piezocone testing may be warranted. Such testing should be performed under the guidance and supervision of an experienced hydrogeologist or geotechnical engineer.

- Laboratory Testing Program: The geotechnical investigation should include a laboratory testing program performed on soil and rock samples collected from the borings, as well as on materials scheduled for use in constructing the facility. The laboratory testing program should be designed to define the index and engineering properties (including at a minimum, soil unit weights, shear strength parameters, compressibility parameters, and permeability) of all distinct soil and rock units encountered within the borings. This data will be essential in supporting assumptions made in the geotechnical analyses performed for the facility. A sufficient number of tests should be specified to define the range of material characteristics that may be anticipated within each critical soil and rock unit. A list of test procedures for soil and rock materials that should be considered for use is provided in Table 1.3.1-1 as follows:

US EPA ARCHIVE DOCUMENT

Table 1.3.1-1
Laboratory Soil and Rock Test Procedures

Test Procedures	Test Standard
Index Properties	
Water Content	ASTM D 2216
Atterberg Limits	ASTM D 4318
Particle Size Analysis	ASTM D 422
Specific Gravity of Soils	ASTM D 854
Determination of Shear Strength	
Direct Shear Test of Soils	ASTM D 3080
Triaxial Test (U-U)	ASTM D 2850
Triaxial Test (C-U)	ASTM D 4767
Unconfined Compression	ASTM D 2166
Moisture-Density Relationships (Compaction Testing)	
Standard Proctor Test	ASTM D 698
Modified Proctor Test	ASTM D 1557
Compressibility Testing:	
1-D Consolidation Test	ASTM D 2435
Hydraulic Conductivity:	
Permeability Test	ASTM D 2434

Characterization of Strength of Geosynthetic Interfaces: A laboratory testing program establishing the shear strengths of the various interfaces to be used in the design of the cap and liner systems of the facility (soil and geosynthetics) should be performed as part of the construction sequencing, but when determined to be appropriate by the Geotechnical Engineer due to site conditions or design constraints, laboratory testing can be incorporated into the scope of the geotechnical investigation, if the specific materials to be used are known during the design phase. Testing should consist of, at a minimum, 3-point direct shear tests performed per ASTM D 5321 and with a recommended maximum strain rate of 0.04 in/min for all interfacing involving a clay or GCL. Strain rates of up to 0.2 in/min may be used for geosynthetic to geosynthetic interfaces that are not expected to build up pore water pressure during the laboratory test such as a geomembrane to geotextile interface. The tests should determine both peak and residual shear strength parameters (friction angle and adhesion) for each interface being examined. Care should be taken when a non-linear shear strength envelope is apparent, such is common with many geosynthetic interfaces, especially

US EPA ARCHIVE DOCUMENT

those involving a GCL or some clay materials. The Engineer may consider additional testing points in the range of anticipated normal stresses to further characterize the interface shear strength.

Characterization of Earthen Materials to Be Used In Embankment Systems: For projects including embankment systems, appropriate laboratory testing should be performed to characterize the index, strength, consolidation and compaction characteristics of the earthen materials used to construct the embankments and associated features. The list of testing presented above is generally applicable for these systems. Other tests may also be performed based on project-specific requirements and as determined to be appropriate by the Engineer.

Characterization of CCP Materials: Physical and chemical properties of the CCP materials to be disposed of at the facility should be determined, as described in the Ash Characterization Section (Section 1.3.2), and as appropriate for the project. The results of this characterization will be used to establish input parameters related to the CCP materials for use in the stability analyses and other engineering analyses that support the landfill or ash pond design.

1.3.2.2 Hydrogeologic Studies

Definition

Hydrogeologic studies are required by regulatory guidance to evaluate the suitability of proposed landfill sites with respect to the subsurface environment. A suitable subsurface environment is typically one in which the near-surface materials (soils) can be used or otherwise managed to help create the landfill containment (liner and/or capping materials), and in which the uppermost saturated zone (aquifer) can be readily monitored to detect the presence or absence of a potential release from the landfill. Other factors in the evaluation include the character of the aquifer and its use or potential for use as a water supply.

Minimum Design Standards

Each state stipulates the minimum content and order for the hydrogeological report. The most current version of this form should be obtained from the appropriate state at the initiation of the project.

The hydrogeologic report summarizing the investigation methods and results may be required for regulatory review and comment prior to the engineering design.

Specific requirements for the specific state should be reviewed prior to initiating an investigation since there are unique requirements for each state. An overview of requirements for the subject states is provided below.

Alabama: Hydrogeologic investigation requirements for siting coal ash facilities are not explicitly identified in Alabama environmental regulations. Rather, they defer to the intent of the administrative code as determined by ADEM. As a conservative guide, ADEM Form 439, Solid Waste Disposal Permit Application, page 11, provides a detailed list of data that ADEM requires

US EPA ARCHIVE DOCUMENT

to be submitted with a permit application. These requirements should be incorporated into the investigation at the onset. The following are unique requirements to the state of Alabama:

- The applicant may be required to pay ADEM to conduct an independent site investigation.
- A minimum of three borings converted to piezometers will be required for site characterization.
- ADEM must approve boring locations prior to installation.

Kentucky: Unique requirements for Special Waste Landfills in Kentucky stipulate the following:

- Baseline groundwater characterization requires monitoring for a CCP specific list of constituents provided 401KAR45:160 Section 71.
- Cannot be located within 250 feet of a sinkhole or other karst feature suggesting rapid transmission of water to the water table.
- Cannot be located within the 100-year floodplain.
- Operations must be capable of monitoring the uppermost aquifer in a manner that will detect the targeted constituents. Furthermore, the operator must be able to demonstrate that corrective action of the uppermost aquifer is capable of being performed in accordance with 401 KAR 45:160, or propose an equivalent statistical procedure that provides reasonable confidence that the migration of leachate from the site is capable of being detected.

Tennessee: The hydrogeologic report requirements are provided in 1200-1-7-.04(9)(a) of the TDEC Rules. The hydrogeological investigation and report must include the following:

- If the facility will be placed in a karst terrain, the investigation must demonstrate that:
  - There is no significant potential for collapse.
  - That the groundwater flow mechanism is not conduit flow.
  - The facility will not degrade local groundwater resources.
  - Piezometers, geophysical survey, or dye trace studies will be installed/performed as required by TDEC.
- If a fault is located within 200 ft of the proposed location that underwent displacement in Holocene time, it will be necessary to prove the facility will be safe from seismic issues.
- Bottom clay must be 1x10-5 cm/s and 10 ft, or 1x10-6 cm/s and 5 ft, above seasonal high water level (unconfined) or the top of a confined aquifer.
- The quantity of all sample sets must be consistent with the appropriate statistical procedures.

US EPA ARCHIVE DOCUMENT

Selected Design Standard

Investigation

Hydrogeologic studies for landfill siting shall be conducted in two phases. The first phase involves desktop studies, a minimal site investigation to support a fatal flaw analysis. The second phase involves site-specific field studies to generate data in support of the evaluation and potentially an extended fatal flaw analysis.

The elements of the investigation process are as follows:

1. Literature review (typically Phase 1 only)
2. Work Plan Development
3. Site Investigation
4. Data Analysis and Reporting

The investigation process is iterative until sufficient data is collected to determine if the site is suitable for a landfill.

Literature Review

The objective of the literature review is to gather and evaluate the available site specific, local, and regional hydrogeologic data including:

- Existing site information from previous studies. For developed properties this may include geotechnical and groundwater investigations.
- Published material regarding
    - Geology. Geologic reports from the U.S. Geological Survey (USGS) or the applicable state geological survey.
    - Hydrogeology. USGS water resource investigations, state-generated groundwater resource maps, and other special reports.
    - Topography. USGS topographic maps and any local or site-specific maps.
    - Flood insurance maps. Obtained from the FEMA National Flood Insurance Program.
    - Aerial photographs. Obtained from state transportation and agricultural departments (National Agricultural Imagery Program), public-domain internet services, and private services.
    - Soil maps. Obtained from the U.S. Department of Agriculture local Soil Conservation Service extension.
    - Applicable regulations
    - Permits for nearby facilities. Obtained through www.epa.gov/enviro/ or a Freedom of Information (FOIA) request.

US EPA ARCHIVE DOCUMENT

- A well inventory to identify all private, commercial and agricultural wells within at least one mile of the site. This information is available on-line for some states and can only be obtained through direct contact in others.

Following completion of the literature review, there may be sufficient information to conduct an initial fatal flaw analysis. Elimination as a result of a fatal flaw analysis at this stage should be based on readily apparent criteria such as regulatory prohibitions or grossly unsuitable physical characteristics.

Work Plan Development

The findings of the literature review or the summary report from the previous investigation shall be used to develop a work plan for onsite investigations. The following provides an overview of the items included in a work plan:

- Soil borings to inspect subsurface conditions.
- Conversion of site borings to monitoring wells or piezometers with which to test the uppermost aquifer conditions.
- Groundwater monitoring for a sufficient period of time to provide a statistically defensible background data set, which is often four to eight monitoring events.
- Groundwater elevation measurements to establish the seasonal high water table and/or the range of groundwater flow conditions.
- Aquifer testing to characterize the aquifer transmissivity, permeability, and other elements required to characterize the direction, rate and mechanism of flow within the aquifer.
- Laboratory testing may be required to document soil permeability and other characteristics.

The work plan document should clearly identify the data to be collected, how it will be collected, and where it will be collected. The components of the work plan include:

- Identification of data gaps
- Identification of investigative methods to be implemented.
- Identification of investigation locations

Data gaps shall be identified prior to selecting the investigative methods to be used. The appropriate investigative methods to provide the targeted data should be included in the work plan. Potentially applicable investigative methods are listed on Table 1.3.1-2.

US EPA ARCHIVE DOCUMENT

Table 1.3.1-2
Investigation Methods and Implementation Considerations

Investigative Method	Implementation Considerations
Photogrammetric Study	Aerial photos- preferably from multiple years, stereo pairs, taken in winter and prior to development are preferred
Test Pits	Utility clearance
Geophysical Survey	Survey lines must be cleared to allow vehicle access
Borehole Geophysics	Methods will be limited based on the type of casing used to hold open the formation, if required
Soil borings & Monitoring Well Installation	▪ Investigative derived waste (IDW) plan ▪ Backfill as stipulated by State ▪ Use appropriately licensed contractor ▪ Register borings/monitoring wells with state if required ▪ Utility clearance
Soil sampling	▪ QA/QC samples should be collected according to the work plan or QAPP ▪ Sample protocol should be provided in workplan ▪ Contracted lab should provide appropriate sample jars
Groundwater Elevation Survey	Measure depth of water from top of casing (TOC) to the 1/10th of foot
Groundwater Sampling	▪ QA/QC samples should be collected according to the work plan or QAPP ▪ Sample protocol should be provided in workplan ▪ Contracted lab should provide appropriate sample jars
Slug Test	
Pumping Test	
Packer Test	
Dye Trace Study	UIC permit required
In-Situ Flow Direction	Method is time-intensive if well recover slowly
Surface Water Elevation Survey	All wells and piezometers must be surveyed to the nearest 0.01 ft.
Surface Water Sampling	▪ QA/QC samples should be collected according to the work plan or QAPP ▪ Sample protocol should be provided in workplan ▪ Contracted lab should provide appropriate sample jars

After the investigative methods have been selected, the final step is selection of the investigative locations. In Phase 1 investigations locations may be biased to identify potential fatal flaws such as sinkholes. Phase 2 investigation locations shall be spatially unbiased to document the homogeneity or heterogeneity of the site.

The work plan, particularly a Phase 2 work plan, may require review by the regulatory agency prior to implementation. Proceeding without regulatory buy-in may result in an additional mobilization to collect specific data required by the agency.

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Hydrogeologic investigations must be tailored to the specifics of the site, disposal unit, permit requirements, geology, and regulatory agency. Hydrogeologic investigations are typically required to be conducted directly by or under the supervision of a registered or licensed geologist, or otherwise qualified professional.

A boring plan must be included in the work plan. The boring plan shall identify proposed boring locations, drilling methods, sampling frequency and sampling method. A justification for the location and depth of each boring shall be provided. General guidelines on boring frequency and depth are described below.

The frequency of borings is dependent upon the phase of the investigation, the site size, and hydrogeologic complexity. Five soil borings are typically sufficient for a Phase 1 investigation to confirm the regional and local hydrogeological setting. Phase 2 investigations require a greater spatial density of borings (up to one boring per 10 acres) to ensure the homogeneity of the strata and aquifer beneath the site.

All borings must be advanced a minimum of 25 feet below ground surface. Increasing the boring depth for one or all borings may be necessary if groundwater or bedrock has not been encountered. If groundwater is reached prior to bedrock and bedrock is not expected within 25 feet of the groundwater elevation, then the boring can be halted otherwise it should be advanced to the bedrock interface.

Two downgradient and one upgradient monitoring well installations are typically sufficient for a Phase 1 investigation. As with soil borings, a greater spatial density of wells will be required to ensure the homogeneity or define the heterogeneity of the site in Phase 2. The following information must be considered before installing a monitoring well:

- Data to be obtained.
- Well must be located in an area or of a construction type that will not be damaged during site operations.
- Well construction methods must prevent downward migration of surface water.
- If a confining unit is present, surface casing must be installed into the confining unit and the well drilled through the casing to termination.
- Wells that are not necessary to the investigation should be abandoned according to state-specific procedures as soon as possible to reduce the risk of possible contamination to the aquifer. Abandonment may require state regulatory approval.

Aquifer characterization is typically performed only in Phase 2 investigations. Characterization includes testing by slug tests, pumping tests or packer tests to obtain information on transmissivity, storativity, and permeability. In areas of high secondary porosity such as karst or fractured rock, a dye trace study may be required.

Modifications to the work plan should be expected if site conditions deviate from those anticipated.

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Site Work/Data Collection

Work plan implementation begins with the non-invasive methods first, then investigation of overburden, followed by (or simultaneously with) investigations of the uppermost aquifer. The following outlines the typical progression of site work for Phase 2 of a project:

1. Perform surface geophysical survey.
2. Confirm boring and monitoring well locations based on geophysical survey.
3. Install soil borings.
4. Convert soil borings to monitoring wells or piezometers. All borings that will not be converted to monitoring wells or piezometers should be abandoned upon completion.
5. Survey monitoring wells and piezometers to ± 0.01 ft.
6. Collect water level measurements at least twice ten days apart.
7. Perform pumping tests or other appropriate aquifer characterization methods.

All site work shall be thoroughly documented in serialized, bound logbooks or equivalent. Errors should be lined through once, dated and initialed. Lines should not be skipped. Once a page, is complete it shall be signed and dated. Boring logs and monitoring well diagrams are frequently recorded on forms specifically designed for this purpose. Critical information from these forms shall also be included in the bound logbook:

- boring or monitoring well name
- location relative to fixed landmarks or coordinates using a handheld GPS unit
- total depth drilled
- drilling and sampling method
- reason for ending the boring
- well construction information, including:
  - total depth of monitoring well
  - length of screened interval, specifically the distance from highest and lowest slots to the bottom of the well
  - well material type and diameter
  - filter pack type and amount
  - annular seal type and amount
  - grout seal and amount type
  - type of surface completion

Investigative derived wastes (IDW) will be generated during most intrusive investigations. The IDW should be handled/disposed of appropriately. For greenfield sites, this may simply be spreading drill cuttings and discharging groundwater on ground surface. For other sites, wastes shall be handled in a manner consistent with local, state and federal regulations, as dictated by a waste characterization analysis. The TVA Environmental group should be contacted regarding IDW prior to initiation of field activities.

Data Analysis and Reporting

It is critical to evaluate data as it is collected for the following reasons:

- To ensure that the data has been collected in an appropriate manner such that the data is representative and reproducible.
- To verify that the data collected will increase the accuracy of the developing conceptual site model. If the data is not required by regulation and it does not increase the accuracy of the developing conceptual site model (CSM) then it may be appropriate to cease that data collection method in lieu of a more appropriate method.
- To continuously evaluate of the CSM, allowing for modifications to the field program when unexpected data is encountered.
- To collect additional data while personnel and equipment are still mobilized, if the data set is determined to be inadequate

Conceptual Site Model (CSM)

The hydrogeologic study results are used to develop a CSM. The CSM synthesizes the data into a graphical representation of the geology, geologic structure, and groundwater flow at the site. The CSM is a dynamic model that starts on a broad basis and evolves as more information is generated.

The CSM should characterize groundwater conditions on site as they may apply to the design and operation of the proposed facility such as:

- The character of the unsaturated zone (soil, rock or other) between the facility and the underlying saturated zone.
- The character and identity of the geologic materials in which the uppermost saturated zone (aquifer) and potentially interconnected aquifers occur on site.
- The ability of the aquifer to transmit groundwater and the character of that flow (whether diffuse or discrete).
- The path of groundwater flow from upgradient (recharge) to downgradient (discharge) areas.
- The gradient and rate of groundwater flow through the uppermost aquifer.
- The local use of groundwater for water supply.

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The amount of information needed to develop a sufficient CSM to the support site selection will depend on the complexity of the site. Table 1.3.1-3 provides a generalized representation of how the site complexity increases the amount of data required to develop the CSM. All sites must be evaluated at the regional (~100 mile radius), local (facility) and the site specific scale.

Table 1.3.1-3
Generalized Representation of Relationship Between Site Complexity and Data Required

Investigative and Data Analysis Methods	Least Complex	Moderately Complex	Highly Complex
	Shallow, unconfined overburden aquifer	Confined overburden aquifer above bedrock	Bedrock aquifer with fractures, jointing and high secondary porosity
	homogenous → isotropic		heterogeneous → anisotropic
Cross Sections	2	2	2
Piezometers	3	3 per geologic unit	3 per geologic unit
Soil Borings	3	8	15
Monitoring Wells	1	5	15
Hydraulic Conductivity	slug tests	pumping test	pumping tests
Potentiometric Surface Maps	2 at least 2 wks apart	quarterly for 1/2 year	quarterly for 1 yr
Stratigraphic Maps	yes	yes	yes
Flow Nets		1	multiple
Borehole Geophysics		yes	yes
Geological Mapping of Bedrock and Structure		yes	yes
Surface Geophysics		yes	yes
3-D Geology/Lab Testing			yes
Conceptual Models			yes
Core Drilling			yes
Geochemical Groundwater Comparisons			yes
In-Situ Flow Direction			yes
Dye Trace Studies			yes
Groundwater Modeling			yes
Fracture Trace Analysis			yes
Fault Mapping			yes

US EPA ARCHIVE DOCUMENT

1.3.3 Ash Characterization

1.3.3.1 Chemical Characterization

Definition

Chemical characterization of CCP materials can be conducted to evaluate regulatory classification, leachate treatment requirements, chemical compatibility with landfill components, etc. The chemical characteristics of CCP materials will vary based on factors including coal source, coal processing mechanism, scrubbing technology, boiler type, other emissions control technologies, and conveyance type (especially wet sluice versus dry conveyance). Chemical characterization can include more conventional leach testing such as synthetic precipitation leaching procedure (SPLP), and toxicity leaching characteristic procedure (TCLP), column leaching) and totals analysis (digestion), but could include other analyses such as x-ray diffraction, x-ray fluorescence, scanning electron microscope (SEM), etc. depending on the purpose of the characterization.

The TCLP procedure is commonly used and utilizes an extraction fluid designed to mimic the environment of a typical municipal solid waste landfill. The TCLP method may yield higher concentrations of heavy metals compared with other methods that may more represent a CCP disposal facility such as the SPLP method (SW-846 Method 1312) or a deionized (DI) water leach (e.g. ASTM D3987). If chemical compatibility or prediction of heavy metal concentrations for treatment evaluation and/or fate and transport modeling are of critical importance methods using a more representative extraction fluid may be warranted.

All TCLP extracts should be analyzed for the 8 RCRA Metals listed in 40 CFR §261.24 (Arsenic, Barium, Cadmium, Chromium, Lead, Mercury, Selenium, and Silver). In addition, TCLP samples should be analyzed for other constituents commonly found in CCP materials, including Chloride, Sulfate, Aluminum, Antimony, Beryllium, Boron, Calcium, Cobalt, Copper, Iron, Magnesium, Manganese, Nickel, Thallium, Titanium, Vanadium, and Zinc. Organic compounds are not typically found in CCP materials, but should be evaluated if there is reason to believe they may be present.

Minimum Design Standards

Alabama: In Alabama, "ashes" are defined as the solid residue from the burning of wood, coal, coke or combustible material used for heating or burning or incineration of solid waste. The terms "solid waste", garbage", and "ash", as defined in ADEM solid waste rules, do not include ash resulting from the combustion of coal at electric or steam generating plants. Therefore, no specific testing requirements or frequency of testing is specified in the state of Alabama.

Kentucky: In Kentucky, CCPs are generally considered special wastes, which are regulated under 401 KAR 45. There are few specific requirements for chemical characterization of special wastes. 401 KAR 45:110, Section 1 (1) states that the engineering design of a special waste facility should consider the physical and chemical characteristics of the waste, including compatibility with the liner system, cap system, and water that comes into contact with the waste. In addition, 401 KAR 45:210 Section 3 (2) states that the special waste material must not be a hazardous waste material as defined in 401 KAR Chapter 31. Determination of the special

US EPA ARCHIVE DOCUMENT

waste material as hazardous/non-hazardous is done through the TCLP procedure and comparison with the hazardous levels in 40 CFR §261.24. No frequency of testing is specified.

Tennessee: The existing TVA CCP facilities in Tennessee are permitted as Class II Industrial Waste Landfills. It is assumed that future solid waste landfills will also be constructed as Class II facilities. The definition of Industrial Wastes states that the material cannot be deemed hazardous as regulated under Subtitle C of RCRA. This implies that TCLP analysis of the material cannot yield constituent levels greater than those listed in 40 CFR §261.24. Rule 1200-1-7-04 (a)(4) requires that a potential alternative liner system must be chemically compatible with the leachate generated by the waste. Similarly, Rule 1200-1-7-04 (a)(5) states that the leachate collection system must be chemically resistant to the leachate produced by the waste material. No specific testing requirements or frequency of testing is specified.

Federal: No specific chemical characterization of the material is required.

Selected Design Standard

With respect to chemical characterization, the characteristic that is of most importance to the design and maintenance of a containment facility is the leachable constituents of the ash, and in particular leachable heavy metals and salts. The chemical characteristics for each CCP material should be evaluated as follows:

- The primary goals for chemical analysis of the CCP material is to evaluate if the material leaches constituents in concentrations that exceed state or federal regulations (in particular levels greater than those listed in 40 CFR §261.24) and to evaluate chemical compatibility with containment facility components. In order to allow comparison with 40 CFR §261.24 limits for hazardous waste, a TCLP procedure (EPA SW-846 Method 1311) should be performed.
- If no historic data exists for the CCP material, samples should be collected and analyzed using TCLP on enough individual samples to allow for statistical significance (typically eight samples). Thereafter, samples should be collected and analyzed at least annually or when any significant fuel source change (i.e. change from eastern coal to PRB coal), after a significant process modification including, but not limited to, the addition/modification of pollution control equipment, boiler modification, coal processing modification, etc. State regulations and/or permit conditions may require more frequent sampling events.
- An annual analysis of an undiluted leachate sample may be performed in lieu of a TCLP analysis on the CCP material if there have been no other significant process changes to the generating facility, coal source, etc.
- Other types of testing can be performed as required for various projects. For example, column tests, sequential batch tests, batch testing with various solid to liquid ratios, etc. may be more useful for groundwater modeling projects than a TCLP or SPLP test.

US EPA ARCHIVE DOCUMENT

1.3.3.2 Physical Characterization

Definition

Determination of the physical characteristics of the CCP material may include material strength, grain size distribution, Atterberg Limits, material compaction characteristics, material density, moisture content requirements for handling and compaction, compressibility, dynamic property evaluation, and hydraulic conductivity. The importance of these parameters will vary depending on the parameter's importance to the design of the facility.

Minimum Design Standards

Alabama: No specific physical characterization of the material is required.

Kentucky: No specific physical characterization of the material is required.

Tennessee: No specific physical characterization of the material is required.

Federal: No specific physical characterization of the material is required.

Selected Design Standard

Required testing of CCP materials for physical properties is to be determined by the design Engineer. It is the design Engineer's responsibility to determine what physical characteristics are important for a specific facility.

Frequency of testing shall also be determined by the Engineer. It is recommended that all tests for critical parameters be completed in duplicate on distinct samples. It is also recommended that critical physical properties of CCPs, as determined by the Engineer, be repeated upon significant change in fuel source or after a significant process modification including, but not limited to, the addition/modification of pollution control equipment, boiler modification, coal processing modification, etc.

TVA has a database of physical properties of its CCP materials in a report by Law Engineering entitled "Fly Ash, Bottom Ash, and Scrubber Gypsum Study" (Contract No. TV-92657V, Phase 1) completed in 1995 (referred to herein as "TVA CCP Database"). This source may provide an adequate basis for material properties. It is the Designer's responsibility to evaluate whether system changes since 1995 may have altered properties of the CCP material or whether the data is adequate and representative. It is of particular importance to ensure that strength testing data performed as part of this study were performed over the range of normal stresses that represent the design of a new facility for critical applications.

If the results from the TVA CCP Database are to be used for design, it is recommended that index testing such as grain size distribution and standard compaction testing be performed to establish whether the materials are similar enough in basic physical characteristics to use other physical characteristics shown in the database such as shear strength.

The following tests are recommended for CCP materials if the material properties are deemed important to the design:

US EPA ARCHIVE DOCUMENT

US EPA ARCHIVE DOCUMENT

Grain Size Distribution

- All CCP materials should be tested in accordance with ASTM D422. For more coarse grained CCPs, sieve alone may provide adequate data for the design. For other design applications, the designer may want to perform sieve with hydrometer to better classify the fine material.

Hydraulic Conductivity

- Materials should be evaluated for hydraulic conductivity under the conditions (density, moisture content, gradient, etc.) that the material will be placed during construction of the intended design.
- Gypsum, and fly ash (ponded and dry) should be tested for hydraulic conductivity using ASTM D5084.
- Bottom ash should be tested for hydraulic conductivity using ASTM D2434
- Other non-gypsum scrubber byproducts should be tested for hydraulic conductivity using ASTM D5084 if they are generally fine-grained and are anticipated to have a permeability of less than 1×10^{-5} cm/s. If the non-gypsum scrubber material is granular in nature, samples should be tested using ASTM D2434.
- Materials that are pozzolonic (self-cementing) in nature should be prepared and allowed to cure prior to testing (typically 7 to 30+ days). Depending on the nature of the material, curing time may or may not affect the hydraulic conductivity of the material, but allowance for curing time will be more representative of the long-term properties of the material.

Shear Strength

- Shear strength for all CCPs should be determined at the lowest density and highest moisture content that are anticipated for the project.
- In general, effective stress shear strengths for all CCP materials should be determined by the direct shear method (ASTM D3080), or by triaxial testing with pore pressure measurements (ASTM D 4767). Other test methods, such as the direct simple shear test (ASTM D 6528) may also be used, at the discretion of the Geotechnical Engineer and as appropriate. Total stress shear strengths should be determined using triaxial testing (ASTM D 4767).
- Materials that are pozzolonic (self-cementing) in nature should be prepared and allowed to cure prior to testing (typically 7 to 30+ days). Depending on the nature of the material, curing time may or may not affect the shear strength of the material, but allowance for curing time will be more representative of the long-term properties of the material.

Atterberg Limits

CCP materials are generally non-plastic. If Atterberg Limits are desired by the design engineer, they should be performed as per ASTM D4318

Compaction and Density Testing

In general, compaction testing for all CCP materials should be conducted using standard effort (ASTM D698) unless the project warrants the material to be compacted at higher densities with greater effort during construction.

- The compaction characteristics of coarse grained material such as bottom ash and boiler slag may be determined using a one-point proctor test using standard effort at the discretion of the design engineer. Compaction curves for bottom ash and boiler slag with low fines content are typically "flat". Bottom ash should be compacted moist, at its typical as-received moisture content (e.g. bottom ash from pond, freely drained) or at a value established by historical data.
- Care must be taken when oven drying CCP materials containing hydrated water, such as gypsum and non-gypsum FGD material. ASTM 2216 recommends that materials that contain hydrated water be dried at 60 degrees Celsius or using a desiccator at room temperature. Regardless if low temperature or room temperature drying techniques are used, care must be taken to ensure that free water content is being determined in relation to compaction testing and not total water content (which would include hydrated water).

Consolidation Testing

- Most CCP materials exhibit low consolidation coefficients (<0.1) for moderate to well compacted CCP materials and is generally not a critical concern for most applications. If the designer desires consolidation data for CCP materials, it should be performed per ASTM 2435. The sample should be prepared in a manner that represents the moisture content and density of the condition being evaluated. This is of particular consequence when evaluating the compressive behavior of ponded fly ash material.

Other Testing

Other material testing for design purposes should be determined by the design engineer.

1.3.4 Liner and Cap System

1.3.4.1 Liner System

Definition

A liner refers to a continuous layer of natural or man-made materials, beneath and/or on the sides of a surface impoundment, landfill, or landfill cell, which restricts the downward or lateral escape of wastes, waste constituents, or leachate. Liners for disposal facilities are typically composite systems constructed of one or more layers of a low permeability synthetic material such as high density polyethylene (HDPE), linear low density polyethylene (LLDPE), or polyvinyl chloride (PVC) and a low permeability compacted clayey soil.

Minimum Design Standards

Alabama: There are no known minimum design standards for liners for CCP facilities in Alabama. Design requirements for liners of Municipal Solid Waste (MSW) facilities are given in 335-13-4-.18. MSW facilities are to be equipped with a composite liner system consisting of

US EPA ARCHIVE DOCUMENT

two components; the upper component must consist of a minimum 40 mil (60 mil if using HDPE) flexible membrane liner (FML), and the lower component must consist of at least a two-foot layer of compacted soil with a hydraulic conductivity of no more than 1 x 10^{-7} cm/sec. The FML component must be installed in direct and uniform contact with the compacted soil component.

Alabama has an additional requirement of a leachate collection system. A leachate collection system shall be required that is designed and constructed to maintain less than a 12-inch depth of leachate over the liner. Refer to Section 1.5 for leachate management system design standards.

Kentucky: There are no known minimum design standards for liners used for special waste facilities in Kentucky. Minimum design standards for liners of contained facilities for other types of waste are given in 401 KAR 48:080. For contained facilities a primary and secondary composite liner system is required. The primary liner system shall consist, at a minimum, (from bottom to top): a 36 inch clay layer with a permeability of 1 x 10^{-7} cm/sec; a primary synthetic liner with a demonstrated hydraulic conductivity less than 1 x 10^{-12} cm/sec and a thickness of at least 60 mils; a 12 inch drainage layer with permeability of 1 x 10^{-2} cm/sec or a layer of equivalent performance; and a filter fabric. The secondary liner system shall consist of (from bottom to top): a 12 inch soil layer with a permeability of 1 x 10^{-7} cm/ sec; a secondary synthetic liner as specified above for the primary liner; a 12 inch drainage layer with a permeability of 1 x 10^{-2} cm/ sec; and a filter fabric. The secondary soil and synthetic components may be replaced with an existing naturally occurring soil material meeting the requirements given in 401 KAR 48:080, Section 1(2).

Additional requirements in Kentucky:

- Liner subgrade shall be sufficiently dry, compacted and structurally sound to ensure that the first lift and all succeeding lifts of soil placed over the landfill subgrade can be adequately compacted to the design requirements. Proofrolling of the subgrade is required, with a minimum 100,000 pound loaded four tire scraper (20 cubic yard size) or equivalent procedure and equipment approved by the cabinet.
- Strains in all geosynthetic components are required to be maintained at less than 10%;
- The slope of liner components shall be no less than 3% percent toward the main leachate collection line, 1% along the main leachate collection line, and no greater than 50% at any point.

Tennessee: Liners for Class II facilities in Tennessee are required to meet minimum design requirements for liners of Class I facilities, which are given in 1200-1-7-.04(4)(a)(1), unless otherwise approved by the State. In summary, a composite liner system is required, consisting of two components: the upper component must consist of a minimum 30-mil (60-mil for HDPE) geosynthetic FML, and the lower component must consist of at least a two-foot layer of compacted soil with a hydraulic conductivity of no more than 1.0 x 10^{-7} cm/sec. The FML component must be installed in direct and uniform contact with the compacted soil component. Liners shall be sloped such that, excluding excavation side slopes, the slope of the liner shall not exceed 25%.

US EPA ARCHIVE DOCUMENT

Additional requirements in Tennessee:

- Geologic Buffer: The liner system shall be placed on top of a geologic buffer which has a maximum hydraulic conductivity of 1.0×10^{-5} cm/sec and measures at least ten feet from the bottom of the liner (or have a maximum hydraulic conductivity of 1×10^{-6} cm/sec and measures at least 5 ft from the bottom of the liner) to the seasonal high water table of the uppermost unconfined aquifer or the top of the formation of a confined aquifer.
- Leachate Collection System: A leachate collection and removal system is required immediately above the liner that is designed, constructed, maintained, and operated to collect and remove leachate from the facility. Refer to Section 1.5 for leachate management system design standards.

Federal: No liner system is specified for CCP facilities.

Selected Design Standard

The design of liner systems shall based on the following standards, which have been determined based on the state requirements given above. Alternate and equivalent systems may also be applicable, if acceptable to the regulators:

Thickness and Materials: Where specific State requirements are given for CCP facilities (such as in Tennessee), the liner system should be designed to meet those specific requirements. Where no specific requirements are given, the basic liner system design should consist of (from top to bottom) a drainage layer; an FML; and a recompacted soil liner (RSL). These layers should meet the following minimum guidelines:

Drainage Layer: Should consist of a minimum 12-inch thick layer of granular soil, a geocomposite material with an equivalent transmissivity, or a combination of geocomposite and granular soil materials designed to maintain less than a 30 centimeter (12 inch) depth of leachate above the liner system. A geotextile separator, or graded granular filter, should be included between the top of the drainage layer and the emplaced CCP. Refer to Section 1.5 for leachate management system design standards.

FML: FMLs should have a minimum thickness of 30 mils (60-mils if HDPE). As an industry standard, typically HDPE or PVC materials are used in liner system FML applications. The FML should be physically and chemically resistant to chemical attack by the waste, leachate, or other materials to which it may come into contact. The FML should be placed in direct contact with the RSL layer. The FML should be protected from the drainage layer by an appropriate cushion, such as a geotextile fabric.

RSL: RSL should consist of at least 24-inches of material meeting classifications of CL, CH, or CL-ML per the United Soil Classification System (USCS). It should be demonstrated by appropriate laboratory testing that the material has a maximum permeability of 1×10^{-7} cm/sec under moisture and density conditions consistent with those under which it will be placed in the field. RSL should be free from organic and other deleterious materials, and sharp materials which may pose puncture hazards to the FML.

US EPA ARCHIVE DOCUMENT

A generalized schematic of this basic liner system, which illustrates the key components, is presented in Figure 1.3.3-1.

Additional design considerations include:

Geologic Buffer: If specifically required by the state, the landfill design shall include a geologic buffer between the liner system and groundwater. Buffer materials may consist of or include the existing soil materials below the facility, if these materials meet the state requirements. If the existing materials do not meet the requirements, compacted fill materials meeting the requirements shall be included in the design to act as the geologic buffer.

Liner Slope: Unless specific state requirements are more stringent, the facility should be designed such that the liner system has a minimum slope of at least 2% toward leachate collection piping and a minimum of 0.5% along leachate collection piping, after including the effects of estimated facility settlements. Where state requirements are more stringent, those requirements should be incorporated into the design.

Anchor Trenches: In general, anchor trenches for geosynthetic components of the liner system should be provided around the entire perimeter of the landfill facility. Anchor trenches should be designed to provide resistance against pullout of the materials. Anchor trenches should be designed per Section 1.4.5.

Shear Strength: The liner design should include the determination of combinations of peak and residual shear strength properties (friction angle and cohesion) which satisfy requirements for global slope stability of the facility, as described in Section 1.4.

Working Strain: The facility should be designed such that the maximum predicted strain induced in any FML components is less than 5%, under all or part of the weight of the facility, and considering the predicted settlements under the facility, unless specific data from the synthetic manufacturer allows for a higher strain.

1.3.4.2 Cap System

Definition

The cap system, also referred to as the final cover system, is the cover material and components that are spread and compacted on the top and side slopes of a facility which will be permanently exposed to the environment. The cap system is placed at the end of the life of the facility and is designed to inhibit and control the movement of CCP, dust, water, and gases both into and out of the emplaced CCP. Caps are typically composite systems constructed of one or more layers of: a vegetative cover, protective soil material, a drainage layer, and a low permeability layer of recompacted soil or a geosynthetic material (such as HDPE).

Minimum Design Standards

Alabama: There are no known minimum design standards for caps of CCP waste facilities in Alabama. Design requirements for caps of MSW facilities are given in 335-13-4-.20. In summary, the MSW requirements state: The final cover system must be comprised of an erosion layer(s) underlain by an infiltration layer(s). The infiltration layer must be comprised of a

US EPA ARCHIVE DOCUMENT

minimum of 18 inches of earthen material and/or a synthetic layer that has a permeability less than or equal to the permeability of any bottom liner system or natural subsoils present, or a permeability no greater than 1 x 10^{-5} cm/sec, whichever is less. The erosion layer must consist of a minimum 6 inches of earthen material that is capable of sustaining native plant growth.

Kentucky: There are no minimum design standards for caps used for special waste facilities. Minimum design standards for caps of contained facilities are given in 401 KAR 48:080, Section 8. In summary, the contained facilities requirements state: At a minimum the final cap shall consist of a layered system. Each layer shall have the same slope of between five and twenty-five (25) percent. The components, listed from bottom to top, are: a filter fabric or other material approved by the cabinet; a 12 inch sand gas venting system with a minimum hydraulic permeability of 1 x 10^{-3} (a layer that is not needed in a CCP landfill as it does not generate gases); a filter fabric or other material approved by the cabinet; an 18 inch clay layer with a maximum permeability of 1 x 10^{-7} cm/sec; for areas of the final cap with a slope of less than 15 percent, a 12 inch drainage layer with a minimum permeability of 1 x 10^{-3} cm/sec; and a 36 inch vegetative soil layer.

Tennessee: Unless otherwise approved by the State, caps for Class II facilities in Tennessee are required to meet minimum design requirements for caps of Class I facilities, which are given in 1200-1-7-.04(4)(a)(6) and 1200-1-7-.04(8)(a)(1). Section 1200-1-7-.04(8)(a)(1) states: "The depth of final cover system shall be at least 36 inches of soil of which a minimum of 12 inches shall be for the support of vegetative cover. The design of the final cover system shall be such that the infiltration volume of water will be equal to or less than the percolation volume through the bottom liner system or a design which includes a compacted soil layer of at least 24 inches which has a permeability no greater than 1 x 10^{-7} cm/sec, whichever is less. This design shall be supported by the use of the Hydrologic Evaluation of Landfill Performance (HELP) model or other equivalent method approved by the Commissioner".

Federal: No cap system is specified for CCP facilities.

Selected Design Standard

The design of cap systems should be based on the following standards, which have been determined based on the state requirements given above. Alternate and equivalent systems may also be applicable, if acceptable to the regulators:

Thickness and Materials: Where specific state requirements are given for CCP facilities (such as in Tennessee), cap design should follow those requirements. Otherwise, the basic cap system should consist of the following layers, at a minimum (from top to bottom): A layer of cover soil material, with a total thickness that is equal to or greater than the local frost penetration depth and of which the upper 6-inches is capable of supporting vegetation (topsoil); A minimum 24-inch thick layer of recompacted soil with a permeability of less than 1 x 10^{-7} cm/sec and meeting USCS soil classifications of CL, CL-ML, or CH should be placed below the cover soil. Additionally, dense vegetative (non-woody) growth at the surface of the cap shall be specifically included in the design.

US EPA ARCHIVE DOCUMENT

An alternate cap system may be considered, if clay materials are unavailable or difficult to obtain. The alternate system should consist of the following layers, from top to bottom: Minimum 24-inches of cover soil, of which the upper 6-inches is capable of supporting vegetation (topsoil); a nonwoven geotextile separator, or graded granular filter,; a drainage layer consisting of either 12-inches of granular soil materials or a geocomposite material; a nonwoven geotextile separator (if a geocomposite drainage layer is selected, the geotextile separator may be omitted); and a FML with a minimum thickness of 30-mils (40 mils if LLDPE or HDPE). As an industry standard, typically LLDPE or PVC materials are used in cap system FML applications. The FML should be physically and chemically resistant to chemical attack by the CCP, leachate, or other materials to which it may come into contact. The FML should be placed directly on top of a prepared, compacted subgrade that is free from sharp materials that pose a puncture hazard to the FML. Additionally, dense vegetative (non-woody) growth at the surface of the cap shall be specifically included in the design. A generalized schematic of this alternate cap system, which illustrates the key components, is presented in Figure 1.3.3-2.

Infiltration Analysis: The cap system should be designed to restrict infiltration of surface water into the facility such that all requirements for operating head on the leachate collection system at the base of the facility are met. This design should be evaluated by use of the HELP model. HELP Model analysis is described in Section 1.5.1.

Cap Slope: Where specific requirements on maximum slope are given by a state (such for facilities in Alabama), the design should follow those requirements. If there are no specific requirements, the slope of all cap system layers should not exceed 3 Horizontal to 1 Vertical (3H:1V), unless it is explicitly demonstrated by analysis that all cap system components and the underlying CCP subgrade will be stable at a steeper slope.

Stability: If geosynthetics are to be used in the cap, the cap design should include the determination of combinations of peak and residual shear strength properties (friction angle and cohesion) which satisfy minimum guidelines for shallow translational slope stability, as described in Section 1.4.4. Additionally, anchor trenches should be provided for geosynthetic components as required to provide resistance against pullout of the materials. Anchor trenches should be designed per Section 1.4.5.

Surface Water Management and Soil and Erosion Control: In general, benches/terraces should be provided in the design of the cap system, to limit surface erosion and convey stormwater off of the landfill cap system. The spacing of benches shall be designed in accordance with soil and erosion control requirements described in Section 1.6.1. The design requirements for surface water management structures, including benches/terraces, are described in Section 1.6.2. The design requirements for sediment basins are described in Section 1.6.3.

1.3.5 Volume and Life Calculations

Definition

Volume and life calculations are performed to determine the disposal volume, operational lifetime, and soil balance for the overall facility, as well as for interim phases within the facility. These calculations are based on estimates of CCP generation rates, in-place CCP density, and the

US EPA ARCHIVE DOCUMENT

geometry and areal limits of phases within the facility design. The calculations provide a basis to measure disposal rates and to schedule construction to provide for additional disposal capacity.

Minimum Design Standards

Alabama: There are no minimum design standards for volume and life calculations for special waste facilities or MSW facilities in Alabama.

Kentucky: There are no minimum design standards for volume and life calculations for special waste facilities in Kentucky. However, 401 KAR 47:190 Section 2 requires submission of phasing plans within the application for the proposed facility. Development of these plans will inherently require an analysis of volume and life.

Tennessee: For Class I, II, and III facilities that require a permit, 1200-1-7-.04(9)(c)(9) and (10) require narrative descriptions to be provided of the volume and type of waste to be disposed of in the facility, as well as the total acreage to be filled in each phase.

Federal: There are no minimum design standards for volume and life calculations for CCP facilities.

Selected Design Standard

The landfill design shall include specific volume and life calculations and a technical memo summarizing these calculations.

Volume and life calculation standards for new facilities should be based on the following considerations:

- The desired working life of the facility. Regarding facility life, TVA has the following internal goals:
    - Facilities are to be designed for a minimum 20 year working life, if possible.
    - Facility planning should be such that design of new facilities should begin a minimum of six years prior to the end of the working life of the previous facility.
- Estimates of annual CCP generation rates, as well as potential for future increases/decreases in these rates during the lifetime of the facility.
- Estimates of the in-place volume of the CCP within the facility. These should be based on prior experience as well as results of the ash characterization program.
- In general, separate volume and life calculations should be performed for each construction phase of the facility.
- The volume of any required cover, soil liners and caps, berms, etc. should be accounted for in the available volume and life calculations of any phase within the facility.

US EPA ARCHIVE DOCUMENT

To obtain accurate results, it is recommended that available phase and facility volumes be computed using computer-aided design and drafting techniques and software packages, rather than via hand calculations.

1.4 STABILITY ANALYSIS

The following criteria have been developed to define performance standards and an overview of methods of engineering evaluation for the construction of new dry ash landfills. Analyses and evaluations are currently being performed by others for wet ash landfills and will be incorporated at a later date.

1.4.1 Hydrostatic Uplift Analysis

Definition

Soils, geosynthetics, or CCP materials that are placed underneath a temporary or permanent phreatic surface may be subject to hydrostatic forces, especially when the phreatic surface lies above materials or components with low permeability, such as the base liner system. If the driving force of the hydrostatic pressure at the base of a facility component exceeds the resisting force, primarily provided by the weight of the component and any soils placed over it, the component may be physically uplifted or deformed. Other effects such as heave or instability of the subgrade or excessive groundwater infiltration at the facility base may also occur. The hydrostatic uplift analysis evaluates the driving and resisting forces and determines a factor of safety against uplift, defined as the ratio of the resisting to the driving forces.

Minimum Design Standards

Alabama: There are no minimum design standards for hydrostatic uplift analysis in Alabama.

Kentucky: There are no minimum design standards for hydrostatic uplift analysis in Kentucky.

Tennessee: There are no minimum design standards for hydrostatic uplift analysis in Tennessee.

Federal: There are no federal minimum design standards for hydrostatic uplift analysis.

Selected Design Standard

The selected design standard includes the following components:

- Contours defining the highest temporal phreatic and potentiometric (confined or pressurized water table) surfaces on site shall be established based on the results of the hydrogeological and geotechnical investigations.

- Using the design base grades of the facility and the contours of the phreatic and potentiometric surfaces, an elementary hydrostatic uplift analyses should be performed. Methods of elementary analysis are presented in the Ohio EPA's "Geotechnical and Stability Analyses for Ohio Waste Containment Facilities", Chapter 7. In summary, this analysis involves computing the hydrostatic uplift at the base of a facility component or

US EPA ARCHIVE DOCUMENT

at the base of a confining subsurface layer (based on the elevation of the phreatic or potentiometric surface relative to the elevation of the base of the component or confining layer), and comparing it with the total vertical stress acting at the base of the component or layer. The factor of safety, defined as the ratio of the total vertical stress and the hydrostatic uplift pressure, should be greater than 1.4. The elementary analysis should include cases which consider all interim or permanent configurations of the facility and subsurface soils that may be prone to hydrostatic uplift (such as a point in time during landfill construction at which the liner has been placed and minimal or no cover is present over the liner) and should delineate areas of the facility footprint exhibiting factors of safety less than the design criteria presented above. If such areas are identified through the elementary analysis, an effort should be made to adjust the design of the facility's base grades to yield a safety factor satisfying the above recommended design criteria.

- If redesign to satisfy the design criteria established by the elementary analysis is not feasible, then more rigorous analyses may be performed to demonstrate that the factors of safety are higher than predicted by the elementary analysis. Such analyses may include steady state or transient seepage analyses that predict the magnitude of hydrostatic uplift forces on the liner system, or other analyses deemed appropriate by the Geotechnical Engineer.
- In the case that the factor of safety requirement for uplift cannot be met with more rigorous analyses or by redesign of the facility's base grades, then specific engineering modifications to lower the phreatic or potentiometric surfaces, such as underdrain systems, pumping wells, etc., may be incorporated into the design, to raise the factor of safety against hydrostatic uplift. If required, these systems should remain active until such time as the weight of materials placed over the component or layer of concern is sufficient to counteract the hydrostatic uplift forces with a factor of safety of 1.4.

1.4.2 Slope Stability Analysis

1.4.2.1 Deep-Seated Failure Analysis (Static)

Definition

Deep seated failure analyses evaluate the potential for mass slope instabilities. Qualitatively, failure geometries with a maximum thickness greater than 10 ft may be considered to be deep-seated. Deep-seated failures may involve the CCP alone, or pass through the CCPs and into the underlying foundation materials, and are generally analyzed as being rotational (having a circular geometry) or translational (having a block-like geometry). The potential for deep-seated instability is dependent on factors such as slope geometry, shear strength of the CCPs, foundation materials, liner materials, and groundwater conditions. The analysis of deep-seated instabilities is performed using computer slope stability analysis software, which generally use limit equilibrium analysis and the method of slices.

Analyses under static conditions evaluate the potential for deep-seated instabilities under normal, service conditions of the facility, either during its construction or post closure.

US EPA ARCHIVE DOCUMENT

Minimum Design Standards

Alabama: There are no known minimum design standards for deep-seated stability analyses in Alabama.

Kentucky: There are no specific minimum design standards for deep-seated stability analyses for special waste facilities or for contained waste facilities in Kentucky. However, for contained waste facilities, 401 KAR 48:080 Section 3 (1) makes the following generalized requirement pertaining to the subgrade soils below a proposed landfill: The landfill subgrade material shall be free of organic material and consist of bedrock, on-site soils, or any select fill with the structural ability to support the landfill maximum load with a factor of safety of two (2.0).

Tennessee: There are no known specific minimum design standards for deep-seated stability analyses for Class II or Class I facilities in Tennessee.

Federal: There are no federal minimum design standards for deep-seated stability analyses for CCP facilities.

Selected Design Standard

Static, deep-seated stability analyses shall be performed based on the following recommendations. Guidance for these types of analyses is provided in [NAVFAC, 1986] and [OEPA, 2004]. The recommendations presented herein are based in part on these documents. A deep-seated stability analysis should include the following:

- Critical slope geometries under interim and long term configurations. Interim conditions are defined as any configuration of the facility that is of a temporary nature (such as a temporary internal slope that exists at close of one cell and prior to start of construction of the adjacent cell) that will be present for a period of less than one year. A long term configuration refers to any configuration that is not classified as interim (for example, post-closure geometries, or semi-temporary conditions that may exist for longer than one year during construction).
- Cross-sections of the facility which represent critical (worst-case) geometries or configurations. Prior to beginning the stability calculations, a screening analysis should be performed in order to identify these critical cross-sections considering both long term and interim facility configurations (such as height of fill over existing grade, sideslope geometry, etc.), properties of the foundation materials (including strength and consistency of the foundation materials, depths to competent material, etc.), and groundwater conditions. It will likely be necessary to analyze more than one section in a given project if a single critical section cannot be identified by inspection.
- Both circular and block-type failure surface geometries, as well as geometries involving failures along internal slopes and along liners featuring geosynthetic interfaces. Failure surface geometries of a wide range of sizes should be considered in the analyses, in order to identify the critical failure surface geometry.

US EPA ARCHIVE DOCUMENT

- The results of the geotechnical investigation and laboratory testing should be utilized to establish shear strength and unit weight parameters for the foundation soils and to establish groundwater elevations for input into the stability analyses.
- All static slope stability analyses should be based on effective stress strength parameters. Peak shear strength parameters should be assigned to earthen materials. For analyses of block failure surfaces which pass through a multilayer base liner system, shear strengths representing the liner should correspond to the weakest interface within the liner system. The Geotechnical Engineer should consider the amount of displacement that is anticipated along the liner system during service when selecting shear strengths for use in the analyses. Low anticipated deformations warrant the use of peak shear strength parameters, while high anticipated deformations would warrant the use of residual shear strength parameters. If the specific materials that are to comprise the liner system are known at the time of the stability modeling, the strengths assigned should be based on the results of laboratory direct shear testing that are representative of the materials and interfaces to be used in the design. If the specific materials that are to comprise the liner system are unknown at the time of the stability modeling, the modeling should be used to establish an envelope of friction angle and adhesion combinations, which together satisfy the design criteria minimum safety factors (see below) for all design cases. This envelope should be made part of a performance specification for the liner materials in conjunction with similar requirements determined from the seismic deep-seated stability analyses (Section 1.4.2.2) and the shallow translational stability analyses (Section 1.4.4).
- Landfill construction involves incremental placement of fill materials over a relatively long period of time. Thus increases in porewater pressures induced by each increment of loading will generally have time to come to equilibrium with the applied loads, prior to application of the next increment – i.e., effective strength conditions will prevail in the foundation. However, soft, cohesive materials with low permeability that underlie the facility may not adequately drain in response to one load increment before the next increment is applied. Such layers may be prone to stability failures under undrained conditions. In such situations, stability analyses should include a check of slope stability utilizing total strength parameters for the soft, cohesive layers.

  The Geotechnical Engineer should determine if a facility is prone to failure in a total strength mode, on a case-by-case basis. The following guidance is provided for total strength stability analyses:

  - Total strength stability analyses should be considered if soil materials with undrained shear strength of less than 1,000 pounds per square foot (psf) and permeability of $1x10^{-7}$ cm/sec are present within or underneath the facility.
  - Implementation of total strength analyses must consider the estimated rate of fill placement within the facility, and the value of the undrained shear strength that is available in each soft, cohesive layer at any time during the landfilling process. The undrained shear strength of these materials will change over time, depending on the amount of consolidation that has occurred under the load that has already been applied. Analyses representing a number of points in time during the landfilling process will usually be required.

US EPA ARCHIVE DOCUMENT

- The rate of consolidation and the undrained shear strength available after a particular amount of consolidation has occurred under a given load should be determined based on laboratory consolidation and triaxial testing.

- A minimum factor of safety of 1.5 shall be used for static analysis of deep-seated stability for post-closure slopes or slopes that are not classified as interim slopes. Interim slopes shall be designed for a factor of safety of 1.3. These factors of safety pertain to both effective strength and total strength analyses. Facilities that do not meet these criteria will need to be redesigned. If the facility meets the criteria under effective strength analyses but not under total strength analyses, consideration should be given to incorporating controlled loading rates (staged construction), vertical subdrainage (wick drains) or other methods of ground improvement. These techniques should be clearly incorporated into the plans and specifications for the facility.

- At a minimum, all slope stability analyses should be performed using 2-dimensional limit equilibrium analysis based on the method of slices and capable of determining minimum (critical) factors of safety based on a search of a wide range of potential circular and block failure geometries. An analysis methodology satisfying both force and moment equilibrium and incorporating the effects of interslice forces should be utilized – such as Spencer's Method or the Morgenstern-Price Method. Hand calculations and computer analyses are acceptable for this purpose. In certain situations, finite element or finite difference methods may also be used in conjunction with the limit equilibrium procedures, at the discretion of the Geotechnical Engineer.

1.4.2.2 Deep Seated Failure Analysis (Seismic)

1.4.2.2.1 Design Seismic Event

Definition

The design seismic event is defined as the event producing the maximum horizontal acceleration at the base of the facility for which the facility and its components must be designed to remain stable. This maximum horizontal acceleration is used as input in evaluating the liquefaction potential and seismically influenced global stability analyses.

Minimum Design Standards

Alabama: There are no minimum design standards for the design seismic event for special waste facilities. For municipal solid waste facilities 335-13-4-.01 (1) (d) gives requirements that are essentially identical to those in Tennessee (see below).

Kentucky: There are no specific minimum design standards for special waste facilities. For contained waste facilities, 401 KAR 48:070 Section 3 states: At a new contained solid waste landfill unit located in a seismic impact zone, all containment structures, including liners, leachate collection systems, and surface water control systems shall be designed to resist the maximum anticipated horizontal acceleration in lithified material for the site.

Tennessee: 1200-1-7-.04(2)(v) states that Class I and II disposal facilities shall not be located in seismic impact zones, unless the owner or operator demonstrates that all containment structures, including liners, leachate collection systems, and surface water control systems, are designed to

US EPA ARCHIVE DOCUMENT

resist the maximum horizontal acceleration in lithified earth material for the site. The owner or operator must place the demonstration in the Narrative Description of the Facility and Operations Manual. The state provides the following definitions with regard to this regulation:

> "Seismic impact zone" means an area with a ten percent or greater probability that the maximum horizontal acceleration in lithified earth materials, expressed as a fraction of the earth's gravitational pull will exceed 0.10g in 250 years.
>
> "Maximum horizontal acceleration in lithified earth material" means the maximum expected horizontal acceleration depicted on a seismic hazard map, with a 90 percent or greater probability that the acceleration will not be exceeded in 250 years, or the maximum expected horizontal acceleration based on a site-specific seismic risk assessment.

Selected Design Standard

Seismic design of facilities shall be performed based on an event which produces a maximum horizontal acceleration in lithified earth materials with a probability of exceedance of 2% in 50 years (the design event). This corresponds to an event with a return period of approximately 2,500 years, and is approximately equivalent to the requirements for Class I and II facilities in Tennessee and MSW facilities in Alabama.

The following information should be obtained/established, prior to performing seismic stability analyses:

- The seismic site class should be determined using the NEHRP site class definitions and based on the results of the geotechnical investigation. Sites that include ponded or sluiced fly ash materials or fly ash materials below the water table should be given Site Class F designation, by inspection.
- The peak ground acceleration in lithified earth at the site and corresponding to the design event should be selected based on data from the United States Geological Survey's (USGS) Seismic Hazard maps (currently available online at http://gldims.cr.usgs.gov/nshmp2008/viewer.htm and/or from the data in TVA's region-specific study performed by AMEC GeoMatrix.

  The peak acceleration obtained as above is referred to herein as PGA_{rock}.

- Most sites will include overburden soils and geomaterials lying above bedrock. In these cases, the parameter PGA_{rock} should be corrected to account for the characteristics of the overburden (i.e., potential for amplification of the rock motions through the overburden materials), to obtain the peak horizontal acceleration at the ground surface at the site (i.e., at the base of the facility), referred to herein as PGA_{design}.. The parameter PGA_{design} will be used as input in several of the seismic stability analyses described in the subsequent sections.

US EPA ARCHIVE DOCUMENT

The following minimum level of effort should be implemented to determine PGA_{design}::

For sites classified as Class A or B (rock sites), PGA_{rock} can be used as PGA_{design}. A site-specific response analysis (as described below) may also be used to determine PGA_{design}

Sites for which PGA_{rock} is less than 0.20g and the seismic site classification is C through E, PGA_{design} may be determined by multiplying PGA_{rock} by published amplification ratios to account for amplification of rock motions through the soil profile at the site. References containing amplification ratios include [Idriss, 1991]. A site-specific response analysis (as described below) may also be used to determine PGA_{design}.

Sites that have PGA_{rock} greater than 0.20g and are designated Site Classes C through E, or Class F sites with any value of PGA_{rock} warrant further study to obtain PGA_{design}. A site-specific response analysis is recommended for such sites. The site response analysis should include a seismological evaluation to establish the design moment magnitude, M_{design} (as described below) and to develop time histories at bedrock that are representative of the design event. The response analysis should also include a 1-D nonlinear or equivalent-linear ground response analyses which accounts for propagation of the rock motions through the soil column and to the ground surface. This analysis should be used to establish $PGA_{design.}$

- The mean moment magnitude for the event at the location of the site should be selected using USGS deaggregation data (currently available online at http://eqint.cr.usgs.gov/deaggint/2008/) and/or from the data in TVA's region-specific study performed by AMEC GeoMatrix (most current study at the time of writing of this document is dated 2004).

  The moment magnitude obtained in this fashion is referred to herein as M_{design}. This moment magnitude should be used as input to the seismic stability analyses described in subsequent sections.

1.4.2.2.2 Liquefaction Analysis

Definition

Liquefaction is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading. Liquefaction generally occurs in saturated fine to medium sands, silts, or similar cohesionless soils, in response to increases in pore pressures and associated loss of shear strength created by seismically induced ground motion. Liquefaction analyses are performed with the objectives of evaluating the potential for liquefaction of each subsurface deposit underlying a facility, and if liquefaction potential exists, predicting the extent of deformation that may occur due to liquefaction. This section describes minimum design guidelines for liquefaction analyses.

US EPA ARCHIVE DOCUMENT

Minimum Design Standards

Alabama: There are no minimum design standards for liquefaction analyses for CCP facilities. For municipal solid waste facilities, regulations which are indirectly related to liquefaction include: 335-13-4-.01 (1) (d) which presents requirements for facilities located in seismic impact zones. The requirements are essentially identical to those in Tennessee (see above).

Kentucky: There are no minimum design standards for liquefaction analysis for special waste facilities and contained waste facilities.

Regulations which are indirectly related to liquefaction include:

> 401 KAR 48:070 Section 3 states: At a new contained solid waste landfill unit located in a seismic impact zone, all containment structures, including liners, leachate collection systems, and surface water control systems shall be designed to resist the maximum anticipated horizontal acceleration in lithified material for the site.
>
> 401 KAR 48:070 Section 4 for contained facilities which require that the facility design address stability of the facility components if constructing in unstable areas.

Tennessee: Tennessee addresses liquefaction analyses in a document published by the Division of Solid Waste Management and entitled "Earthquake Evaluation Guidance Document". This guidance references a procedure for evaluating liquefaction potential of foundation soils presented in [Seed and Idriss, 1971] (see references below).

Regulations which are indirectly related to liquefaction include:

> 1200-1-7-.04(2)(v) Class I and II disposal facilities shall not be located in seismic impact zones, unless the owner or operator demonstrates that all containment structures, including liners, leachate collection systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site. The owner or operator must place the demonstration in the Narrative Description of the Facility and Operations Manual. The state provides the following definition with regard to this regulation:
>
> "Seismic impact zone" means an area with a ten percent or greater probability that the maximum horizontal acceleration in lithified earth materials, expressed as a fraction of the earth's gravitational pull will exceed 0.10g in 250 years.
>
> "Maximum horizontal acceleration in lithified earth material" means the maximum expected horizontal acceleration depicted on a seismic hazard map, with a 90 percent or greater probability that the acceleration will not be exceeded in 250 years, or the maximum expected horizontal acceleration based on a site-specific seismic risk assessment.

Selected Design Standard

- Soil units defined by the geotechnical investigation should be separated into materials that may subject to classical liquefaction and those that may be subject to softening and substantial reduction in strength during and after earthquake shaking. Separate evaluations will be required for each type of material.

US EPA ARCHIVE DOCUMENT

Soil layers susceptible to classical liquefaction should be identified as follows: Layers classified as sands or gravels, per USCS (all S- and G- classes); Layers classified as silts or clays (M- and C- classes) that have plasticity index less than 7. These soil layers are defined as Category 1 materials herein.

Natural silt, clay, or organic soils (M- and C- and O- classes) that have plasticity index greater than or equal to 7 though not susceptible to classic liquefaction, may be prone to softening under cyclic loads. These soil layers are defined as Category 2 materials herein.

Soils not fitting within the definitions of Categories 1 and 2 as defined above are considered special soils and are defined as Category 3 materials. Category 3 materials include fly ash and other CCPs, or other unnatural soil materials. Category 3 soils may be prone to classic liquefaction or cyclic softening, but their liquefaction (or softening) behavior is not well established in the literature and needs to be specifically addressed by the Geotechnical Engineer.

Evaluation of Soils With Potential For Classical Liquefaction (Category 1)

- All soil layers that fall under Category 1 as described above should be evaluated for the potential for liquefaction.
- Analysis of liquefaction potential should be performed in general accordance with the methods given in [You'd et al., 2001]. This methodology is an update to the methodology presented in the Tennessee guidance document referred above. This is a semi-empirical procedure which relates the soil's resistance to liquefaction based on typical field data that is obtained during the geotechnical investigation (SPT N values, CPT resistances, etc). The liquefaction analysis shall have the design seismic event (defined in Section 1.4.2.2.1) as its basis. Material properties for each layer used as input to the analyses should be established based on the results of the geotechnical investigation and site-specific laboratory testing.
- The result of the liquefaction analyses will be a factor of safety against liquefaction, where the factor of safety is defined as follows:

$$FS_{liq} = \frac{CRR}{CSR}$$

Where CRR = Cyclic Resistance Ratio, corresponding to the design event

CSR = Cyclic Stress Ratio, corresponding to the design event.

The factor of safety against liquefaction of each subsurface layer within Category 1 should be determined.

- Interpretation of the computed factors of safety is performed as part of the seismic slope stability analyses and seismic deformation analyses, as defined in Sections 1.4.2.2.3 and 1.4.2.2.4 below.
- If appropriate and at the discretion of the Geotechnical Engineer, the cyclic resistance of soils may be determined through the use of project-specific laboratory testing. Project-specific laboratory testing is recommended to establish cyclic resistance of highly

US EPA ARCHIVE DOCUMENT

sensitive clay soils falling within Category 2 (sensitivity greater than 6) or soils which reach peak undrained strength at low strains (less than 5% strain).

Evaluation of Soils With Potential For Cyclic Softening (Category 2)

- All soil layers that fall under Category 2 as described above should be evaluated for the potential for softening under cyclic loading.
- Analysis of the potential for cyclic softening should be evaluated according to [Idriss and Boulanger, 2008]. This is a semi-empirical procedure which relates the soil's resistance to cyclic softening based on data that is obtained during the geotechnical investigation (undrained shear strength and overconsolidation ratio). The analysis should have the design seismic event (defined in Section 1.4.2.2.1) as its basis. Material properties for each layer used as input to the analyses should be established based on the results of the geotechnical investigation and site-specific laboratory testing.
- The result of the liquefaction analyses will be a factor of safety against cyclic softening for each layer within Category 2. The factor of safety is defined as follows:

$$FS_{cs} = \frac{CRR}{CSR}$$

Where CRR = Cyclic Resistance Ratio, corresponding to the design event

CSR = Cyclic Stress Ratio, corresponding to the design event.

- Interpretation of the computed factors of safety is performed as part of the seismic slope stability analyses and seismic deformation analyses, as defined in Sections 1.4.2.2.3 and 1.4.2.2.4 below.
- If appropriate and at the discretion of the Geotechnical Engineer, the cyclic resistance of soils may be determined through the use of project-specific laboratory testing.

Evaluation of Special Soils (Category 3)

- Since the seismic response of Category 3 materials may differ from that of natural or typical soils, the methods for determining the potential for liquefaction or cyclic softening recommended for Category 1 and 2 soils above may or may not be applicable. Analysis of liquefaction or softening potential of Category 3 materials should be at the discretion of the Geotechnical Engineer and should be based on material and site-specific laboratory testing data, and on the design event. The Geotechnical Engineer should provide detailed description, assumptions, and supporting data to substantiate the methods that are selected to analyze Category 3 materials.

1.4.2.2.3 Seismic Slope Stability Analysis

Definition

Deep seated stability analyses evaluate the potential for mass slope instabilities, as defined above in Section 1.4.2.1.

Analyses under seismic conditions evaluate the potential for deep-seated instabilities under the action of the design seismic event and after the end of this event. This analysis also seeks to

US EPA ARCHIVE DOCUMENT

estimate the amount of permanent deformation that may occur within the facility due to the seismic action, so that facility components can be appropriately designed.

Minimum Design Standards

Alabama: Regulations and guidance pertaining to seismic stability analyses are as given for the liquefaction analyses, see Section 1.4.2.2.2.

Kentucky: Regulations and guidance pertaining to seismic stability analyses are as given for the liquefaction analyses, see Section 1.4.2.2.2.

Tennessee: Regulations and guidance pertaining to seismic stability analyses are as given for the liquefaction analyses, see Section 1.4.2.2.2.

Selected Design Standard

Seismic slope stability evaluations should, at a minimum, include the following:

1. Determination of critical cross-sections and failure geometries for analysis.
2. An interpretation of the results of the liquefaction analyses (described in Section 1.4.2.2.2).
3. Evaluation of post-earthquake shear strengths.
4. Post-earthquake residual conditions slope stability analyses.

Recommendations and guidelines for each of these components are as follows:

Determination of Critical Cross-Sections and Failure Geometries For Analysis

- The recommendations related to selection of critical cross-sections, failure surface geometries to be considered and slope stability analysis methods given in Section 1.4.2.1 for static stability analyses may also be applied to the seismic stability analyses. At a minimum, cross-sections and failure geometries considered in the seismic stability analyses should include all post-closure configurations and all configurations not classified as interim conditions, as defined for the static stability analyses (Section 1.4.2.1).

Interpretation of Liquefaction Analysis

- Prior to performing slope stability and deformation analyses, the liquefaction analyses presented in Section 1.4.2.2.2 should be performed, to identify those deposits within and underneath the facility that are prone to liquefaction or significant strength loss during the design seismic event. As described previously, the results of the analyses of 1.4.2.2.2 are the factors of safety against liquefaction (FS_{liq})or against cyclic softening (FS_{cs}) for each major soil layer. The potential for liquefaction of each layer should be interpreted based on the computed factors of safety, as follows:

  For soils susceptible to classical liquefaction (Category 1 as defined in Section 1.4.2.2.2):

  $FS_{liq} <= 1.10$ Assume layer liquefies under design event

  $1.1 < FS_{liq} <= 1.4$ Assume partial liquefaction and strength loss in layer

  $FS_{liq} > 1.4$ Assume no liquefaction

US EPA ARCHIVE DOCUMENT

For soils susceptible to cyclic softening (Category 2 as defined in Section 1.4.2.2.2):

FS_{cs} <= 1.4	Assume cyclic softening occurs as a result of design event.
FS_{cs} > 1.4	Assume no cyclic softening occurs.

Evaluation of Post-Earthquake Shear Strengths

The shear strength of soils that have liquefied or softened and liner and cap components that have displaced substantially in response to the design earthquake may be substantially smaller than the static shear strength of these materials.

The first estimate of post-earthquake shear strengths should be evaluated as follows. It may be appropriate in some cases to establish post-earthquake shear strengths in a manner other than that presented below. In such a case, the Geotechnical Engineer shall provide thorough written documentation of the procedures and methodology used to establish the post-earthquake shear strengths assigned.

- For soils susceptible to classical liquefaction (Category 1 as defined in Section 1.4.2.2.2), shear strengths should be taken no higher than as follows:

If FS_{liq}<=1.10	Assume layer is liquefied following the design event and assign post-liquefaction residual strength to the layer.
If 1.1<FS_{liq}<=1.4	Determine the strength for the layer by interpolation based on the computed safety factor against liquefaction. Interpolate between the post-liquefaction residual strength at FS = 1.10, and the full peak strength of the material at FS = 1.40.
If FS_{liq}> 1.4	Assign the full static drained strength of the layer for soils classified as sands or gravels (G- or S- classes); Assign the full static undrained strength for soils classified as silts or clays (M- and C- classes).

  The post-liquefaction residual strengths may be determined using published empirical methods, such as given in Seed, et. al, (2003) and Idriss and Boulanger, (2008) or using project-specific laboratory testing performed under the supervision of the Geotechnical Engineer.

- For soils susceptible to softening under cyclic loads (Category 2 as defined in Section 1.4.2.2.2), shear strengths should be taken as follows:

FS_{cs} <= 1.4	Assign shear strength equal to 80% of the peak undrained shear strength under static conditions.
FS_{cs} > 1.4	Assign shear strength equal to 100% of the peak undrained shear strength under static conditions.
Special Cases	For highly sensitive clay soils falling within Category 2 (sensitivity of 6 or greater) or soils which reach peak undrained strength at low strains, FS_{cs} may be interpreted as given above, but it is recommended that the post-

US EPA ARCHIVE DOCUMENT

earthquake shear strength be established using project-specific laboratory testing.

- For special soils (Category 3), post-earthquake shear strengths should be established by material and site-specific laboratory testing, and at the discretion of the Geotechnical Engineer.
- For liner interfaces containing geosynthetics, assign residual shear strengths. Residual strength properties used in the analyses should correspond to the weakest interface within the liner system. If the specific materials that are to comprise the liner system are known at the time of the stability modeling, the strengths assigned should be based on the results of laboratory direct shear testing on the materials and interfaces to be used in the design. If the specific materials that are to comprise the liner system are unknown at the time of the analysis, the analysis should be used to establish an envelope of friction angle and adhesion combinations, which together satisfy the design criteria minimum safety factors (see below) for all design cases. This envelope should be made part of a performance specification for the liner materials, in conjunction with similar requirements determined from the static deep-seated stability analyses (Section 1.4.2.1) and the shallow translational stability analyses (Section 1.4.4).

Post-Earthquake Residual Conditions Slope Stability Analysis

- This analysis consists of limit-equilibrium slope stability evaluations under post-earthquake conditions. It analyzes the susceptibility of the facility to slope failures occurring after the end of the design seismic event. The analysis should be performed using the post-earthquake shear strengths established as recommended above. Separate analyses should be performed for each critical cross-section and failure surface geometry. Acceptable limit equilibrium analysis procedures for this analysis are the same as those presented for the static stability analyses (Section 1.4.2.1).

  If the factor of safety at any cross-section under this analysis is less than 1.0, the facility is prone to large scale failure after the end of the seismic event, and redesign of the facility is warranted.

1.4.2.2.4 Seismic Deformation Analyses

Definition

Seismic deformation analyses are used to estimate the amount of permanent deformations (horizontal and vertical) that may occur within the facility in response to the design seismic event, as defined above in Section 1.4.2.2.1. The results of this analysis are used to evaluate the potential for damage of facility components due to seismically induced deformations.

Minimum Design Standards

Alabama: Regulations and guidance pertaining to seismic stability analyses are as given for the liquefaction analyses, see Section 1.4.2.2.2.

Kentucky: Regulations and guidance pertaining to seismic stability analyses are as given for the liquefaction analyses, see Section 1.4.2.2.2.

US EPA ARCHIVE DOCUMENT

Tennessee: Regulations and guidance pertaining to seismic stability analyses are as given for the liquefaction analyses, see Section 1.4.2.2.2.

Selected Design Standard

At a minimum, seismic deformation analyses shall include the following:

1. Newmark-type Sliding Block Analysis to determine accumulated displacements that occur during earthquake shaking.
2. For sites at which liquefaction is predicted in Category 1 soils (as defined in Section 1.4.2.2.2) and that also include incised by stream channels or other open face channels, or that are sites constructed on sloping ground, an evaluation of lateral spreading-type displacements should be performed.
3. An analysis of vertical deformations (settlements) induced by earthquake shaking.

Displacements Occurring During Earthquake (Newmark-Type Sliding Block Analyses)

Displacements accumulating during earthquake shaking should be performed using Newmark-type Sliding Block Analyses that explicitly accounts for propagation and amplification of the acceleration at the ground surface at the base of the facility (PGA_{design}), through the CCP mass above. The analysis should include the following:

- A separate Newmark Sliding Block analysis should be performed for each individual critical cross-section and failure geometry considered in the Post-Earthquake Residual Conditions Slope Stability Analysis (see Section 1.4.2.2.3 above).
- The analysis should be implemented such that an estimated magnitude of displacement within the CCP mass, along liner and cap systems, and at the location of any major facility components (underdrainage, leachate collection piping, etc) are obtained.
- Newmark Sliding Block analyses of each cross-section and failure geometry should begin by performing a pseudostatic limit equilibrium slope stability analysis, to establish the yield acceleration, k_y – defined as the magnitude of the pseudostatic seismic coefficient required to obtain a factor of safety of 1.0. Limit equilibrium analyses should be performed using the same methods and assumptions on shear strength as were used for the Post-Earthquake Residual Conditions Slope Stability Analyses (see Section 1.4.2.2.3).
- Using k_y determined as above, and using the geometric and stratigraphic configuration of the cross-section, perform a Newmark-type analysis to obtain seismically induced displacements.

  For Sites classified as Seismic Site Class A and B, or for sites classified as Site Classes C through E that have maximum horizontal acceleration (MHA) < 0.20g (as determined in Section 1.4.2.2.1), the sliding block analysis may be implemented using a simplified procedure such as given in Makdisi and Seed (1978) or Bray, (2007).

  Per the requirements of Section 1.4.2.2.1, a site response analysis will have been performed for sites classified as Site Class F, or for sites classified as C through E that have MHA>0.20g. The site response analysis would include development of site-specific earthquake time histories. For such sites, the Newmark-type sliding block calculations should be performed based on these time histories.

US EPA ARCHIVE DOCUMENT

- More rigorous methods such as finite element or finite difference methods may be utilized to estimate accumulated displacements during earthquake shaking, if warranted and at the discretion of the Geotechnical Engineer.

Deformations Occurring Due to Liquefaction-Induced Lateral Spreads

If the results of the liquefaction analysis of Section 1.4.2.2.2 indicate a factor of safety against classical liquefaction (FS_{liq}) less than 1.10 for any Category 1 soil layers, an evaluation of liquefaction-induced lateral spread displacements should be performed. Lateral spread displacements should be evaluated for each pertinent critical cross-section.

Lateral spread displacements may be computed using semi-empirical procedures, such as given in Bartlett and Youd (1995), and updated in Youd, et. al (2002), or by finite element or finite difference methods, at the discretion of the Geotechnical Engineer.

Earthquake-Induced Vertical Deformations (Settlement)

The buildup and subsequent dissipation of pore pressures induced by earthquake shaking may result in volumetric strains and corresponding settlements of unsaturated and saturated sand soils. The potential and magnitude for such settlements should be evaluated as part of the seismic deformation analyses. Earthquake-induced settlement analyses should be based on published and widely accepted empirical procedures, such as Tokimatsu and Seed, (1987).

Evaluation of earthquake-induced settlements should be performed for each individual critical section considered in the seismic deformation analyses. Additional analyses should also be performed to estimate settlements underneath subdrainage infrastructure, such as leachate collection piping and other facility components that may be sensitive to settlement. .

Deformation Performance Criteria

- *Displacement Criteria:* If applicable, displacement estimates from the Newmark-type Sliding Block analysis should be added to the estimates of lateral spread displacements. The resulting total displacement should then be compared to the following allowable displacements given in Table 1.4.2-1 from Kavazanjian (1999):

Table 1.4.2-1
Allowable Displacement

Component	Allowable Calculated Displacement	Comment
Liner System	150 to 300 mm	Actual expected deformation is very small.
Cover System	300 mm to 1 m	Damage is repairable.
Waste Mass	1 m	For displacement not impacting cover or liner.
Roadways, Embankments	1 m	Conventional geotechnical criteria.
Surface Water Controls	1 m	Conventional geotechnical criteria.
Gas Collection System	No Limit	Breakage common under normal operating conditions.

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If the predicted displacements exceed the allowable values, redesign of the facility is warranted.

The values given in Table 1.4.2-1 correspond to allowable displacements within the CCP mass and for materials and systems typically included in the design of landfill systems. For sites that include other structures or systems not covered by the above table, the Geotechnical Engineer should select appropriate displacement tolerances for these structures or systems.

- *Settlement Criteria:* Differential settlements under the liner and leachate collection systems will tend to reduce the constructed slope of these systems, and can affect their capacity to drain and manage leachate. The facility should be designed such that the minimum post-settlement slopes of the liner between leachate collection pipes is 2% and that the minimum post-settlement slope of leachate collection piping is 0.5 to 1 %. Differential settlements of piping should be limited to allowable values as established by the pipe manufacturer.

  Differential settlement of the liner system may induce strains in the geosynthetic components. The results of the settlement analysis should be used to estimate the maximum strain in the liner geosynthetic components. The facility should be designed to limit the strain to 5%. The strain across two points on the liner system that settle differentially with respect to each other may be computed using the following equation:

  $$E_T(\%) = \frac{L_f - L_0}{L_0} * 100$$ where,

  E_T = Tensile strain

  L_f = Original distance separating two location points

  L_0 = Final distance separating the same two points after settlement is complete

1.4.3 Settlement Analysis and Liner Strain Evaluation

Definition

Settlement analyses predict total and differential subsurface deformations underneath critical elements of the facility, such as the liner system, leachate collection system, and cap system. Settlements are induced by the loads imposed by the facility, the self weight of facility components and the CCP materials, acting on the foundation soils below the facility. Settlement analyses are used to evaluate the effects of the subsurface deformations on the integrity and operation of the facility components and to design appropriate strategies to mitigate these effects in order to maintain serviceability and stability of the facility.

Minimum Design Standards

Alabama: There are no minimum design standards for settlement analyses for CCP facilities in Alabama. For municipal solid waste facilities, settlement concerns are generally addressed in 335-13-4-.01 (1) (5) (i), which requires that the design of the facility address on-site or local soil conditions that may result in significant differential settling.

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Kentucky: There are no minimum design standards for settlement analysis for special waste facilities in Kentucky. There are also no specific standards for settlement analyses for contained waste facilities. However, 401 KAR 48:070 Section 4 (1) for contained facilities generally addresses settlement concerns by requiring that the facility design address on-site or local soil conditions that may result in significant differential settling.

Tennessee: For Class II facilities, 1200-1-7-.04(2)(w)(1) generally addresses settlement concerns by requiring that the facility design demonstrate that measures have been incorporated to ensure the integrity of the facility under the effects of on-site or local soil conditions that may result in significant differential settling. This demonstration is to be made in the design narrative for the facility which is submitted as part of the facility permit application. For Class II facilities, 1200-1-7-.04(4)(a)(1)(iii) states that the liner system be "Placed upon a foundation or base capable of providing support to the liner and resistance to pressure gradients above and below the liner to prevent failure of the liner due to settlement, compression, or uplift". 1200-1-7-.04(4)(a)(4)(iv) makes an equivalent statement for the cap system.

Federal: There are no federal minimum design standards for settlement analyses for CCP facilities.

Selected Design Standard

Settlement analyses shall be performed as part of the facility design and shall be documented for inclusion with the design narrative for the facility. The settlement analyses should include the following components:

- The geotechnical investigation should include laboratory testing that establishes the compressibility characteristics of all critical subsurface layers that are expected to undergo settlements under the facility. At a minimum, the laboratory testing plan should include one-dimensional consolidation tests per ASTM D 2435. A minimum of two tests per critical layer should be specified, to establish a range of values for the compressibility parameters in each layer.
- Settlement analyses should be focused on critical cross-sections of the facility, such as those sections featuring the highest fill, largest variations in the heights of fill, and underneath subdrainage infrastructure, such as leachate collection piping. The settlement analyses should generally be based on the maximum proposed grades for the facility.
- Settlement Analysis Methods: Settlement analyses for normally or over-consolidated fine-grained soils may be based on classical methods as presented in McCarthy, (1998). Settlement analyses for coarse-grained materials may be performed using the Hough Method, see FHWA, (2002). In lieu of, or complementary to, the classical analyses presented above, more rigorous analyses of settlement, such as by finite element or finite difference techniques, may also be performed, if warranted. Constitutive models and corresponding input parameters should be selected based on the characteristics of the materials being analyzed for settlement, and should be based on appropriate laboratory testing.

  Settlement analyses should also include an evaluation of secondary compression and creep of critical layers, as determined appropriate by the Geotechnical Engineer.

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- Because typical facilities will feature large area fills, the load of the facility will generally be transmitted to significant depths below the landfill base. The settlement analysis should account for compression of all soil layers within the subsurface profile to the top of competent bedrock. Settlement of competent bedrock materials can generally be neglected, unless determined otherwise by the Geotechnical Engineer.
- *Seismically Induced Settlements:* Settlement occurs when liquefaction and attendant pore pressure dissipation causes densification of the liquefied layer. The magnitude of settlement (volumetric strain) can be estimated from charts developed in Tokimatsu and Seed (1987) based on the average cyclic shear stress ratio induced by the earthquake and the density of the soil in question. Settlement may result in damage to the liner and/or internal drainage systems. The effects of seismically induced settlements should be considered in the design, at the discretion of the Geotechnical Engineer.
- *Areas of Karstic Terrain and Former Mining Activity:* Where a facility is to be constructed on karstic terrain or former mined areas, appropriate field investigations should be made to identify any underground voids that may exist (geophysical surveys, etc). Near-surface voids or open sinkholes, should be appropriately treated and filled to prevent subsidence underneath the constructed facility. If deep voids are present, further study of the overlying materials should be performed to evaluate the potential for subsidence and its effects on the facility.
- Results of the settlement analyses should be used to estimate differential settlements underneath the liner system collection piping and other components prone to settlement-related distress. The facility should be designed to limit total and differential settlements under these components to allowable values as established by the manufacturer.
- Settlements under the facility will occur after construction of the base liner and leachate collection systems, and during/after filling operations. Differential settlements under the liner and leachate collection systems will tend to reduce the constructed slope of these systems, and can affect their capacity to drain and manage leachate. The facility should be designed such that the minimum post-settlement slopes of the liner between leachate collection pipes is 2% and that the minimum post-settlement slope of leachate collection piping is 0.5 to 1 %. Differential settlements of piping should be limited to allowable values as established by the pipe manufacturer.
- Differential settlement of the liner system may induce strains in the geosynthetic components. The results of the settlement analysis should be used to estimate the maximum strain in the liner geosynthetic components. The facility should be designed to limit the strain to 5%. The strain across two points on the liner system that settle differentially with respect to each other may be computed using the following equation:

$$E_T(\%) = \frac{L_f - L_0}{L_0} * 100 \quad \text{where,}$$

E_T = Tensile strain

L_f = Original distance separating two location points

L_0 = Final distance separating the same two points after settlement is complete

1.4.4 Shallow Translational Failure Analysis

Definition

Shallow translational failures are failures involving sloped interfaces within the cap or liner systems constructed near the surface of a proposed facility. Generally, failures involving interfaces that are within 10 ft of the surface of a permanent or interim slope may be considered shallow translational failures. Critical interfaces for shallow translational failures typically include interfaces involving geosynthetic to soil contact or geosynthetic to geosynthetic contact. Forces driving shallow translational failures include the gravity load (weight) of the cap or liner system (i.e., the component of the weight aligned parallel to the interface slope), seismic forces, and hydrostatic forces due to subsurface water collecting within the interfaces. Forces resisting shallow translational failures include friction and adhesion between the materials at the critical interfaces, and passive earth forces at the base of a run of the cap or liner system. Shallow translational stability analyses quantify the driving and resisting forces along each critical interface, and establish the factor of safety against failure, defined as the ratio of the resisting forces to the driving forces.

Minimum Design Standards

Alabama: There are no specific minimum design standards for shallow translational stability analyses for CCP or MSW facilities. Regulations which are indirectly related to shallow translational stability include: 335-13-4-.20(2)(c)(2), which requires that final slopes for MSW landfill covers not exceed 25%.

Kentucky: There are no specific minimum design standards for shallow translational stability analyses for special waste facilities. For contained waste facilities, general requirements for structural stability of cap and liner systems are given in 401 KAR 48:080 Section 10, which states the following: "The design engineer shall analyze the structural integrity of the site, the subbase, each component of the composite liner, each component of the final cover, the composite liner system and the final cap as a system. Modifications to the design shall be provided where necessary, to achieve a minimum factor of safety of two (2) for the subbase, one and one-fourth (1.25) for the structural design of the facility liner components, and one and one-half (1.5) for the final cover system. Synthetic liner material and structural synthetic materials shall be designed for a maximum elongation of ten (10) percent."

The requirement does not present any specific analysis methodologies or assumptions to be used in evaluating the factors of safety.

Tennessee: There are no specific minimum design standards for shallow translational stability analyses for Class I or II landfills. Regulations which are indirectly related to shallow translational stability include: 1200-1-7-.04(4)(a)(1) which requires that slopes for landfill liners not exceed 25%.

Tennessee provides guidance for seismic stability design of landfill covers in a document published by the Division of Solid Waste Management and entitled "Earthquake Evaluation Guidance Document". This guidance presents a procedure for evaluating seismic displacements of landfill caps, based on that given in [Makidisi and Seed, 1978].

US EPA ARCHIVE DOCUMENT

Federal: There are no specific federal minimum design standards for shallow translational stability analyses for CCP facilities.

Selected Design Standard

Final design of cap and liner materials shall conform to state standards on maximum slopes. Shallow translational stability analyses shall be performed for each critical interface within the final cap system, and should be considered for interim configurations of the liner system (i.e., prior to CCP placement or during the early stages of CCP placement) where liner slopes exceed 10%. The analyses should be performed for the most steeply sloped sections of the proposed cap or liner.

If the specific materials to be used for the cap and liner system components are known during the design phase, a program of direct shear testing should be implemented on all critical interfaces of the design. The resulting shear strength parameters should be used as input within the shallow translational stability analyses.

If the specific materials to be used in construction of the liner and cap system are not known during the design phase, the shallow translational stability analyses should be utilized to develop an envelope of friction angle and adhesion combinations, which together satisfy the minimum factors of safety presented below, for all design cases. This envelope should be made part of a performance specification for the liner and cap materials.

As a minimum design standard, the following conditions should be considered in the shallow translational stability analyses:

1) Static, Drained Conditions: This condition consists of the typical, service conditions to be experienced at the facility. This analysis does not account for the effects of seepage or groundwater, nor those of seismic events.

2) Static, Saturated Conditions: This condition adds the effects of seepage forces on the critical interfaces to the assumptions for the static drained conditions described above. Seepage forces may build within the cap system if the drainage layer becomes clogged or its capacity is exceeded during a high intensity storm event.

3) Seismic Conditions: This condition should consist of the basic conditions assumed for the static, drained analysis, with the addition of a seismic factor corresponding to the design seismic event, as described in Section 1.4.2.2.1. The seismic factor used as input into the analysis should correspond to the ground acceleration at the level of the system being analyzed (cap or liner), after accounting for amplification of ground motions through the materials underlying the system.

4) Static, Residual Strength Conditions: This condition analyzes the susceptibility of the facility to a large scale shallow translational failure, occurring after an unanticipated, short duration event that produces large displacements within the cap system (such as an earthquake). Though the critical interface may survive the initial event, the displacements that are experienced may exceed the peak

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strength of the interface and create a lower strength, residual strength condition afterward. If the residual strength is too low, large scale failure may result after the end of the initial event.

5) Other Conditions: Other project-specific conditions should be considered at the discretion of the Geotechnical Engineer. These may include conditions incorporating surcharges such as construction equipment, temporary slopes or configurations that may exist at some point in the construction or service of the facility, or special conditions such as gas pressures acting on the cap or liner systems.

The analyses should be based on limit equilibrium methodologies, using a slope stability analysis computer program or using suitable closed form solutions such as those presented in the Ohio EPA's "Geotechnical and Stability Analyses for Ohio Waste Containment Facilities" (2004), or in Soong, and Koerner (1996).

In general, peak shear strength parameters may be utilized as input into the shallow translational stability analyses with exception of the following: the static, residual strength condition, and liquefaction at the toe of slope, for which residual strength parameters should be utilized. In addition, the tensile capacity of geosynthetics should be neglected in analyses of shallow translational stability.

The buildup of head at interfaces involving the drainage layer component of cap or liner systems should be limited to the thickness of the drainage layer or less, in order to minimize the effects of hydrostatic uplift forces at this interface for the static, saturated analysis condition. It is recommended that the design be based on procedures given in OEPA (2004).

Recommended minimum factors of safety for each condition are as follows:

1) Static, Drained Conditions: 1.5
2) Static, Saturated Conditions: 1.1
3) Seismic Conditions: >1.0
4) Static, Residual Strength Condition: 1.1
5) Other Conditions: Project-specific and determined by the Geotechnical Engineer.

- These minimum factors of safety assume that strength parameters for the critical interfaces have been established using rigorous, project-specific laboratory testing, and that the analysis is for a typical CCP facility. Uncertainties in strength characteristics of the critical interfaces or in the use or configuration of the facility, or other special project considerations may dictate the use of higher factors of safety in design.
- If the factor of safety under seismic conditions (computed using the limit equilibrium methods described above) is less than 1.0 for the permanent cap system, an analysis of permanent, seismically induced deformations of the cap should be made. As a minimum design standard, a Newmark-type Sliding Block analysis, as described in Section 1.4.2.2.3 should be utilized to estimate deformations. The facility should be designed such that the predicted deformation under the design seismic event is no greater than

US EPA ARCHIVE DOCUMENT

listed in Table .1.4.2.1, and/or not in excess of allowable deformations as established by the manufacturer.

1.4.5 Anchor Trench Analysis

Definition

The terminus of geosynthetic components of liner and cap systems consists of a horizontal runout length and vertical embedment into an excavated anchor trench. The dimensions and configuration of this anchor trench is based on the tensile forces applied to the geosynthetic materials during temporary loading conditions in conjunction with the allowable tensile stresses for these materials.

Minimum Design Standards

Alabama: There are no known minimum design standards for the design of anchor trenches in Alabama.

Kentucky: There are no known minimum design standards for the design of anchor trenches for special waste facilities or contained waste facilities in Kentucky.

Tennessee: There are no known minimum design standards for the design of anchor trenches for Class I or Class II facilities in Tennessee.

Federal: There are no minimum federal design standards for the design of anchor trenches for CCP facilities.

Selected Design Standard

Anchorage shall be designed for a worst-case temporary scenario occurring during construction, such as a condition that may exist while earthmoving equipment is placing cover soil over geosynthetic components of the facility. The loading applied to the geosynthetic components should consider the weight of the soil present over these components, as well as the surcharge loads applied by the construction equipment.

Anchor trench design should be in general accordance with the methodology given in Qian, Koerner, and Gray (2002).

The resistance provided by runout length and anchor trench combinations should not be so great as to induce yield or tensile failure of the geosynthetic material prior to pullout. It is recommended that the resistance provided not exceed 2/3 of the geosynthetic material's yield strength (if the material has a definable yield strength) or ½ of the material's tensile strength.

Regardless of the results of analysis, a minimal anchor trench or combination runout section and anchor trench should be specified in the facility design, to provide resistance against unforeseen loadings such as wind. The minimal design should develop a stress in the geoynthetics that is no greater than ½ of the material's tensile strength.

US EPA ARCHIVE DOCUMENT

1.5 LEACHATE MANAGEMENT SYSTEM

This section presents engineering design and performance standards associated with the effective management of leachate. The leachate management system includes components for collection and extraction of leachate from the landfill, as well as conveyance, storage, and treatment of the leachate. Design standards are provided in this section to estimate leachate generation, design leachate collection system piping and drainage layers, properly size leachate extraction and conveyance systems, and provide adequate leachate storage facilities.

1.5.1 Leachate Generation Calculations

Definition

Leachate generation calculations are conducted to estimate the volume of leachate that will be produced during various phases of the landfill life. The calculations are used for design of the leachate management system, including leachate collection pipe sizing, pipe spacing, leachate collection layer design, leachate extraction system design (pump sizing or gravity outlets), and leachate storage facility volumes.

Minimum Design Standards

Alabama: There are no minimum design standards for leachate generation calculations for CCP waste facilities. There are also no minimum design standards for leachate generation calculations for municipal solid waste (MSW), industrial waste (IW), and construction and demolition debris (C&DD) landfills.

Kentucky: There are no minimum design standards for leachate generation calculations for special waste facilities. There are also no minimum design standards for leachate generation calculations for contained landfills (i.e., MSW).

Tennessee: The minimum design standard for Class II disposal facilities are specified in TDEC 1200-1-7-.04(7). These standards specify that the leachate generation calculations are to be based on the infiltration volume of the 25-year 24-hour storm through the intermediate cover.

Selected Design Standard

The standard to be used for calculating leachate generation shall be based on the Tennessee regulation (TDEC 1200-1-7-.04(7)). More specifically, the Hydrologic Evaluation of Landfill Performance (HELP) model shall be used to calculate leachate generation. The HELP model was developed by the U.S. Army Engineer Waterways Experiment Station under a cooperative agreement with the U.S. Environmental Protection Agency (EPA) and is recommended by the USEPA, widely accepted in industry, and required by many States for evaluating waste management facilities. The HELP model is a computer program that computes estimates of water balances for land disposal systems. As of the date of this Document, the most recent model version is Version 3.07, which can be downloaded free on the Internet at the following website: http://el.erdc.usace.army.mil/products.cfm?Topic=model&Type=landfill.

US EPA ARCHIVE DOCUMENT

At a minimum, the following scenarios should be analyzed by the HELP model to determine leachate generation rates:

- Scenario 1 – Initial Conditions: This scenario should assume one 10-ft lift of CCP material has been placed in the landfill, with 12-inches of intermediate cover soils placed over the CCP material. This scenario provides the peak expected leachate generation rate for the facility, which is typically utilized for sizing leachate extraction and conveyance systems (Section 1.5.4) and leachate storage facilities (Section 1.5.3). In addition, leachate collection pipe spacing and minimum required leachate collection layer permeability should be designed using this HELP model analysis (Section 1.5.2). Site specific design for the minimum liner system grades and maximum leachate flow length to the leachate collection piping should be utilized in the HELP model.

- Scenario 2 - Intermediate Conditions: This scenario assumes half of the permitted CCP height has been placed in the landfill. The CCP material is assumed to be covered with 12-inches of intermediate cover soils. This scenario provides the average leachate generation rates over the active life of the landfill.

- Scenario 3 - Closed Conditions: This scenario assumes the full height of CCP has been placed in the landfill and the final closure cap system has been constructed. This scenario estimates leachate generation rates during the post-closure period and provides cap system infiltration estimates for use in the design of cap system drainage layers.

Each scenario should be simulated by the HELP model for a minimum thirty (30) year time period to verify that all scenarios are modeled for all reasonably expected climactic conditions. Synthetic weather and solar radiation data may be generated using the HELP model for the specific project location. However, the designer must verify that synthetically generated rainfall data provides a peak daily rainfall amount greater than or equal to the 25-year, 24-hour storm event. Current weather data, such as the National Oceanic and Atmospheric Administration (NOAA) shall be referenced to determine the 25-year, 24-hour storm event. If the 25-year, 24-hour storm event is not provided, the designer should manually input this storm event into the HELP model simulation.

As the operation of the first phase of the landfill progresses, actual site-specific leachate generation data may be available based on volumes collected in the leachate management system. If this data is available, the design of the future leachate management system should be evaluated and compared to the HELP model results. Using site specific data as opposed to data obtained from the HELP model will likely require regulatory approval prior to acceptance of any design modifications.

1.5.2 Pipe Sizing and Spacing and Leachate Drainage Layer

Definition

Calculations for establishing the design requirements of the leachate collection drainage layer and the sizing and spacing of leachate collection piping are required to evaluate whether the leachate collection system is adequately designed to maintain leachate depth (head) above the

US EPA ARCHIVE DOCUMENT

liner system below the regulatory-required depth. The maximum depth of leachate head above the liner system is limited to 12-inches per federal regulations 40CFR258 Subtitle D. The leachate collection system, which includes the collection piping and drainage layer, is used to collect the leachate produced in the landfill to prevent the buildup of leachate head on the liner, and to drain leachate to a leachate extraction system. Leachate collection piping is typically constructed of a system of perforated PVC or HDPE pipe. The leachate drainage layer can be constructed of natural material (sand or gravel), geosynthetic materials (geonet or geocomposite synthetic drainage layers), or a combination of natural and geosynthetic materials. Geocomposite drainage layers can accommodate significantly larger flow rates compared with natural drainage layers.

Minimum Design Standards

Alabama: There are no minimum design standards for size and spacing of leachate collection piping and the leachate drainage layer calculations for CCP facilities. There are, however, minimum design standards for MSW, IW, and C&DD landfills as specified in ADEM 335-13-4-.18, which states that the leachate collection system shall be designed and constructed to maintain less than 1-foot of leachate head over the liner.

Kentucky: There are no minimum design standards for size and spacing of leachate collection piping or the leachate drainage layer calculations for special waste facilities. There are, however, minimum component requirements for contained landfills (MSW) as specified in 401 KAR 48:080(6)(4a), which states that a 12 inch drainage layer with a minimum permeability of 1×10^{-2} cm/sec or a layer of equivalent performance shall be designed and constructed to maintain less than one foot of leachate head over the liner. A filter fabric is required above the drainage layer. The slope of leachate collection piping shall be no less than 1% along the main leachate collection line, and no greater than 50% at any point. All leachate collection pipes shall be designed and constructed to a minimum slope of 1%, and no greater than 50% at any point. Main leachate collection lines shall have a minimum diameter of eight (8) inches, and lateral leachate collection pipes shall have a minimum diameter of four (4) inches.

Tennessee: The minimum design standards for size and spacing of leachate collection piping and the leachate drainage layer calculations for Class II disposal facilities are specified in TDEC 1200-1-7-.04(7)(i) and (ii). This regulation states that the leachate collection system must be designed, constructed, operated, and maintained such that the leachate depth over the liner does not exceed 1-foot of leachate head over the liner as calculated referencing the infiltration volume of the 25-year 24-hour storm through the intermediate cover. The regulation further states that leachate interception surfaces and associated piping must be designed, constructed, operated, and maintained to function without clogging throughout the scheduled post-closure care period.

Selected Design Standard

In all cases, the leachate collection system shall be designed to maintain less than 1-foot of leachate head over the liner. The decision on which type of leachate drainage layer to use, either natural materials, geosynthetic materials, or a combination of natural and geosynthetic materials is largely based on material availability and their comparative cost and shall be evaluated on a case-by-case basis.

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Natural Drainage Layer Design: For natural (sand or gravel) leachate collection layers, the minimum spacing of leachate collection piping and minimum required permeability for the leachate collection drainage layer should be evaluated using the HELP model (Scenario 1 as described in Section 1.5.1). The maximum pipe spacing (leachate flow length along the liner system to the leachate collection piping) and minimum required permeability of the granular drainage layer should be designed such that the peak head on the liner system provided in the HELP model output is less than 12 inches. Site specific design for the minimum liner system grades and maximum leachate flow length to the leachate collection piping should be utilized in the HELP model. All natural drainage layers shall include a non-woven geotextile, or graded granular filter, above the leachate drainage layer.

Geosynthetic Drainage Layer Design: The HELP model does not accurately calculate the head on the liner system when modeling geosynthetic drainage layers. The standard method for designing geosynthetic drainage layers, commonly known as the Giroud equation, is provided in Giroud et al. (2000). The Giroud equation calculates the ultimate transmissivity of drainage layers taking into account reduction factors for intrusion, creep, chemical clogging, and biological clogging. Geosynthetic drainage layers should generally be designed to carry the peak leachate inflow rate from the HELP model (Scenario 1 as described in Section 1.5.1) entirely within the geocomposite drainage layer (leachate head should not exceed the thickness of the geocomposite drainage layer). Site specific design for the minimum liner system grades and maximum leachate flow length to the leachate collection piping should be utilized when evaluating geocomposite drainage layers using the Giroud equation.

Natural and Geosynthetic Drainage Layer Design: If the proposed leachate collection system consists of a geosynthetic drainage layer with an overlying natural drainage material, it is recommended that the Giroud equation be used to verify that the peak leachate inflow rate from the HELP model (Scenario 1 as described in Section 1.5.1) can be carried entirely within the geocomposite drainage layer. If leachate cannot be maintained within the thickness of the specified geocomposite, another, larger geocomposite should be specified or additional calculations should be performed to show that the geocomposite combined with the overlying natural drainage layer is adequate to manage the leachate under the peak conditions while maintaining less than 1-foot of head over the liner system using conservative, but reasonable assumptions. The methodology for designing a leachate collection system consisting of a geosynthetic drainage layer and a natural drainage layer is described in Giroud et al. (2004).

Pipe Sizing: The Mannings equation shall be used to verify the size of leachate collection piping using the minimum leachate collection pipe grades and peak daily leachate generation rate from the HELP model (Scenario 1 as described in Section 1.5.1). The Mannings equation is an empirical equation that applies to uniform flow in open channels and pipes and is a function of the velocity, flow area and slope of these systems. This methodology is supported by Quain, Koerner and Gray (2002), and Chevron Phillips Chemical Company (1985).

Pipe Slope: Unless specific state requirements are more stringent, the facility should be designed such that the leachate collection piping has a minimum grade of at least 0.5%, after including the effects of estimated facility settlements. Where state requirements are more stringent, those requirements should be incorporated into the design.

US EPA ARCHIVE DOCUMENT

Additional design considerations for the leachate collection system include the following:

- Designed to function without clogging throughout the scheduled post-closure care period. Clogging shall be considered for both collection pipe perforations as well as clogging (also referred to as "blinding") of the geotextile component of a geosynthetic drainage layer. To minimize clogging of pipe perforations, the size of the perforation needs to be considered when selecting a granular material to surround the collection piping. To minimize clogging of the geotextile component of a geosynthetic drainage layer, the particle size/gradation of the overlying buffer material or CCP must be determined to be compatible with the apparent opening size (AOS) of the geotextile. Design methodology for this is provided in Quain, Koerner and Gray (2002). Alternatively, a graded granular filter (sand, aggregate, etc.) can be used in place of the geotextile to minimize clogging.

- Configure the leachate collection piping system to allow internal inspection, cleaning and maintenance.

1.5.3 Leachate Storage Facility Sizing

Definition

Leachate storage facilities, whether a leachate storage tank, or a leachate storage pond, must be sized to accommodate storage of the anticipated leachate generated at the landfill for an established time period. Sizing of leachate storage facilities is typically conducted using leachate generation calculations to estimate the volume of leachate that will be produced during various phases of the landfill life (as previously discussed in Section 1.5.1). The capacity of the leachate storage facility typically varies from site-to-site, since it is dependent on the type of treatment facilities that are available and the maximum allowable discharge rate to the treatment facility. This is further discussed in the Leachate Loadout, Treatment and Disposal section (Section 1.5.7) of this Document.

Minimum Design Standards

Alabama: There are no minimum design standards for leachate storage facility sizing for CCP facilities. There are also no minimum design standards for leachate storage facility sizing for MSW, IW, and C&DD landfills.

Kentucky: There are no minimum design standards for leachate storage facility sizing for special waste facilities. There are, however, minimum design standards for contained landfills (MSW) as specified in 401 KAR 48:080(6)(4f), which states the leachate collection tanks shall be a minimum of 1,000 gallons and that additional capacity shall be provided to store leachate for a minimum of 15 days production at peak production rates during operation and closure.

Tennessee: The minimum design standard for leachate storage facility sizing for Class II disposal facilities are specified in TDEC 1200-1-7-.04(7)(iii). This regulation states that leachate collection reservoirs must have sufficient capacity to store the volume of leachate expected to be generated in 30 days, or other adequate provisions approved by the Commissioner.

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Selected Design Standard

As previously stated, the capacity of the holding tank will depend on the type of treatment facilities that are available and the maximum allowable discharge rate to the treatment facility. If leachate is to be recirculated onto the landfill, a smaller storage volume may be adequate since storage will be retained within the CCP material. For the purposes of this Document, leachate recirculation refers to the reintroduction of collected leachate into the CCP mass (within the lined areas of the landfill) for purposes of dust control and to aid in material compaction. To a limited extent in CCP landfills, leachate recirculation also allows for temporary storage of leachate within the landfill, thereby delaying overall leachate generation rate into the leachate collection system at the bottom of the landfill. If leachate will be hauled by tanker truck from the storage facility, sufficient volume should be provided to allow for tanker truck down-time, long weekends or allowable discharge rates at the treatment plant. If leachate is to be treated on-site prior to disposal, the volume of the storage facility will be dependent on the flow limitations of the treatment system. If leachate will be conveyed to a connection to a sanitary sewer line, the storage facility size will be governed by the allowable discharge rate to the treatment plant. Note that discharge to a local sewer would need to be coordinated with the appropriate agencies for their specific requirements. Pretreatment and flow control may be necessary prior to its discharge.

Regardless of the multiple scenarios that are typically site-specific, the design standard shall include first calculating leachate generation rates using the HELP model as discussed previously in the Leachate Generation Calculations section (Section 1.5.1). Based on the leachate generation rates, the minimum storage volume for the leachate storage facility shall be one (1) to three (3) days of leachate production during the peak daily leachate production period as specified by the design engineer and recommended in the following reference: "Integrated Solid Waste Management, Engineering Principles and Management Issues", G. Tchobanolglous, H. Theisen, S. Vigil, 1993, pp. 439-440. As stated previously, site specific means of leachate treatment and disposal shall be considered when determining the leachate storage requirements of a facility. If leachate for the facility is being hauled by truck for disposal, three days of peak leachate production is recommended to allow for leachate build-up during such events as long weekends, temporary reductions in hauling capacity due to mechanic problems, weather, etc., and to allow for adequate storage in the event that a significant rainfall event comes while the leachate tanks or ponds have not been fully emptied.

If the facility drains to a gravity sewer or gravity drains to the plant's treatment system, then reduced storage requirements (one day of peak leachate production) may be reasonable for the site.

Additional considerations for the leachate storage facility include the following:

- If the leachate storage facility is a pond, it shall be constructed with a bottom composite (compacted clay and geosynthetic membrane) liner system and groundwater monitoring wells to enable monitoring of any potential leaks through the liner system.
- If the leachate storage facility is a storage tank, whether above ground or below ground, the tank shall be constructed of materials compatible (nonreactive) with the chemicals in the leachate. Storage tanks are typically constructed of specialized plastics, fiberglass, or metal

with specially treated internal coatings or glass lining. Leachate storage tanks can be single- or double-walled, however double-walled tanks are preferred because of the added safety.

- The leachate storage facility should have a reliable and convenient means of detecting the volume of collected leachate in the facility and of sampling such leachate.
- It is standard industry practice and therefore a design standard requirement of this document that a secondary containment system be constructed around the leachate storage facility in the event of leakage or catastrophic failure of the system. In accordance with the federal secondary containment requirements for oil storage (40 CFR 112.8(c)(2) - Spill Prevention, Control, and Countermeasure Plan Requirements), the secondary containment system shall be constructed to provide containment for the entire capacity of the largest single container and sufficient freeboard to contain precipitation. For leachate storage tanks, the typical industry standard volume provided by the secondary containment system is 110% of the total volume contained in the largest storage tank within the leachate storage facility. It is therefore required that the volume of the secondary containment system be designed to provide the larger of either the entire capacity of the largest single container and sufficient freeboard to contain precipitation or 110% of the total volume contained in the largest storage tank within the leachate storage facility.
- The leachate storage facility shall include an early warning high-level alarm (audible and visual) and automatic shut-down of the supply forcemain or pumps to prevent leachate overflows.

1.5.4 Leachate Extraction System Sizing

Definition

The leachate extraction system includes the means to withdraw or convey leachate collected within the limits of the landfill/CCP boundary to the external leachate conveyance system, which ultimately conveys the leachate to the storage system typically located outside the limits of the landfill/CCP boundary. Leachate extraction systems, in general, consist of either gravity flow extraction systems or pumped extraction systems. In either system, the leachate is collected via the leachate collection system (collection piping and drainage layer) and conveyed to the leachate collection sump(s), which are located in low points within the landfill cell liner system. Gravity extraction systems convey the leachate by gravity from the leachate collection sump, through the liner system of the sideslope berm or bottom sump of the landfill, to the external leachate conveyance system and storage system. Pumped extraction systems similarly collect the leachate in a collection sump. However, pumped extraction systems use a pumping system (typically a side-slope riser pumping system) to lift the leachate from the collection sump into the external leachate conveyance system and storage system. Appropriate sizing of the leachate extraction system is important to avoid leachate backups in the landfill cell and to maintain less than one foot of leachate head over the liner system.

US EPA ARCHIVE DOCUMENT

Minimum Design Standards

Alabama: There are no minimum design standards for leachate extraction system sizing for CCP facilities. There are also no minimum design standards for leachate extraction system sizing for MSW, IW, and C&DD landfills.

Kentucky: There are no minimum design standards for leachate extraction system sizing for special waste facilities. There are also no minimum design standards for leachate extraction system sizing for contained landfills (MSW).

Tennessee: There are no minimum design standards for leachate extraction system sizing for Class II disposal facilities. There are also no minimum design standards for leachate extraction system sizing for Class I disposal facilities (i.e., MSW).

Selected Design Standard

The leachate extraction system shall be sized based on the peak daily leachate generation rates calculated using the HELP Model (Scenario 1 as described in Section 1.5.1). The peak leachate generation rate shall be calculated based on the landfill area draining to each leachate collection sump. Based on the leachate generation rates anticipated in each collection sump, the extraction system sizing shall be calculated.

For a gravity flow extraction system, the size of the leachate extraction pipe shall be determined using the daily maximum rate of leachate generation to the collection sump from the HELP model in conjunction with the Manning's equation. As a separate note, for gravity flow extraction systems where the pipe penetrates the liner system of the sideslope berm or bottom sump of the landfill, great care must be taken to ensure that the seal where the pipe penetrates the landfill liner is sound.

For a pumped extraction system, the leachate extraction pumps shall be sized to ensure removal of leachate at the daily maximum rate of leachate generation from the collection sump according to the HELP model. The pumps shall have sufficient operating head to lift the leachate from the base of the leachate collection sump to the access port. Typically, submersible pumps are utilized in pumped extraction systems. The pump "off" level switches shall be set at an appropriate elevation to maintain a sufficient leachate level covering the submersible pump to prevent the pump from burning out. The pump "on" switch shall be set at an appropriate elevation to activate the lead pump to allow for efficient pumping cycles (typically 10 to 12 minutes between cycles). The pump cycle is dependent on the flow capacity of the pump, the peak leachate flow rate into the sump, and the leachate storage volume of the depressed sump area where the pumps are located. This methodology is supported Quain, Koerner and Gray (2002).

It is recommended that a two pump system, operating in a lead-lag configuration be utilized for each side slope riser pumping system. A two pump system provides the ability for intermittent pumping during varied low and medium flow conditions, while also providing for a back-up during pump maintenance and dividing the wear between the two pumps. In addition, in situations where high rates of leachate are being produced, both pumps can operate simultaneously to extract leachate. It is further recommended that a pumped extraction system

US EPA ARCHIVE DOCUMENT

be provided with a visual alarm system at the pump panel to alert operators when the pumping system is not operational or leachate levels are higher than the pump set points. It is recommended that an auto dialer function be utilized to alert operations personnel, a site supervisor at the landfill, or appropriate personnel in a control room at the power plant. The pumps should also be controlled automatically by an automatic shut-off relay from the leachate storage facility prior to the storage facility experiencing overflow levels.

1.5.5 Pipe Strength Calculations

Definition

Leachate collection piping internal to the landfill cells and conveyance piping external to the landfill cells are subject to both dynamic and static loadings capable of damaging (crushing) the piping systems. Significant static and/or dynamic loading may also cause pipe deflection resulting in changes in the underlying pipe slope, as well as its alignment and elevation, and therefore affect the function of the piping system. Static loadings on the piping system are produced from deposited CCP material and/or soil overburden above the piping system. Dynamic loadings on the piping system are produced from equipment or vehicles traversing the area above the piping system. Pipe strength and deflection calculations should verify that the pipe material type, burial depth, and bedding material are adequate to prevent pipe crushing and/or significant deflection.

Minimum Design Standards

Alabama: There are no minimum design standards to withstand for pipe strength to withstand crushing and deflection for CCP facilities. There are also no minimum design standards for pipe strength to withstand crushing and deflection for MSW, IW, and C&DD landfills.

Kentucky: There are no minimum design standards for pipe strength to withstand crushing and deflection for special waste facilities. There are, however, minimum design requirements for contained landfills (MSW) as specified in 401 KAR 48:080(6)(4a), which states that piping shall be designed to withstand static and dynamic loads that may be encountered.

Tennessee: The minimum design standard for pipe strength to withstand crushing and deflection for Class II disposal facilities are specified in TDEC 1200-1-7-.04(5)(i)(II). This regulation states that the leachate collection and removal system be constructed of materials that are of sufficient strength and thickness to prevent collapse under the pressures exerted by overlying CCPs, cover materials and by any equipment used at the facility.

Selected Design Standard

Pipe strength and deflection calculations shall be conducted for both static and dynamic conditions. Pipe strength calculations shall include resistance to pipe deflection and critical buckling pressure.

It is recommended that pipe deflection for the static case be calculated using the Modified Iowa Formula. It is further recommended that the static pipe strength analysis be conducted based on the external soil/CCP pressure and that the piping system's critical buckling pressure is taken

US EPA ARCHIVE DOCUMENT

into account. Deflection less than five percent (5%) is considered acceptable, as recommended by the Plastic Pipe Institute. Consideration must be given to perforated versus solid wall piping. A factor of safety greater than 2.0 shall be acceptable for pipe strength. The calculation methodology for both static pipe deflection and static pipe strength is supported by Quain, Koerner and Gray (2002).

It is recommended that pipe deflection and pipe strength for the dynamic case be calculated using the Boussinesq Point Load Equation and that the PLEXCALC II computer program be used (developed by Performance Pipe, a division of Chevron Phillips Chemical Company LP). The PLEXCALC II program is utilized to calculate factors of safety against wall crushing, wall buckling, and short-term and long-term deflection. This methodology is presented in the Performance Pipe Engineering Manual, Book 2, Chevron Phillips Chemical Company LP (2003). Consideration must be given to perforated versus solid wall piping. Deflection less than five percent (5%) is considered acceptable, as recommended by the Plastic Pipe Institute. A factor of safety greater than 2.0 shall be acceptable for pipe strength.

In addition to the requirements stated above, the soil frost depth shall be considered such that the top of the piping is at least as deep as the local frost depth. Frost depths can be found by consulting the local (typically County) building codes.

1.5.6 Leachate Conveyance

Definition

Leachate conveyance piping, also sometimes referred to as transmission piping, includes piping that transfers the leachate collected within the limits of the landfill/CCP boundary from the leachate extraction system to the leachate storage system located outside the limits of the landfill/CCP boundary. Leachate conveyance piping can function as a gravity flow pipe or as a force main (under pressure). Leachate conveyance piping is typically constructed of PVC or HDPE materials. This section will discuss the design standards to be used for these conveyance piping systems.

Minimum Design Standards

Alabama: There are no minimum design standards for leachate conveyance piping for CCP facilities. There are also no minimum design standards for leachate conveyance piping for MSW, IW, and C&DD landfills.

Kentucky: There are no minimum design standards for leachate conveyance piping for special waste facilities. There are also no minimum design standards for leachate conveyance piping for contained landfills (MSW).

Tennessee: There are no minimum design standards for leachate conveyance piping for Class II disposal facilities, other than requiring that the system be chemically resistant to the leachate expected to be generated, per TDEC 1200-1-7-.04(5)(i)(I). There are no further design standards for leachate conveyance piping for Class I disposal facilities (MSW).

US EPA ARCHIVE DOCUMENT

US EPA ARCHIVE DOCUMENT

Selected Design Standard

The decision whether the leachate conveyance piping will be a gravity flow system or a force main is largely dictated by the topography of the site perimeter, the location of the leachate storage or treatment system, as well as owner preference.

The design standard for gravity flow leachate conveyance piping is that it shall be sized to carry the anticipated peak daily flows from upgradient landfill cells based on the leachate generation calculations (HELP model Scenario 1, as described in Section 1.5.1). The size of gravity piping can be designed using the Manning's equation. It is recommended that manholes be included at approximately 500-foot intervals or other intervals as specified by local code, as well as in locations where significant changes in direction, both horizontal and vertical, to enable the gravity line segments to be isolated for repair and maintenance. If necessary, gravity flow leachate conveyance piping shall be equipped with air release valves and vacuum break valves or a combination air release/vacuum break valve in the appropriate locations to avoid air blockage and/or to provide a vacuum break. These typically will not be necessary if the conveyance piping is operating at a continuous downward slope with manholes, as referenced above, such that the manholes will also act to release air and provide a vacuum break. In addition, for gravity flow systems, consideration must be given to provide a means to stop the flow of leachate through the conveyance piping to the leachate storage facility in the event that the leachate storage facility is full or needs to be shut down for maintenance or other reasons. Flow is typically regulated through the use of manual or automatic valves located near the limits of the landfill/CCP boundary, where the conveyance piping connects to the leachate extraction system.

The design standard for a leachate conveyance force main shall be based on calculations conducted whereby the total system head loss is balanced with the operating range(s) of the pump(s). This methodology, which is outlined below, is supported by US Army Corps of Engineers Guidance Manual TM 5-814-2 (1985). The following methodology shall be used:

- Based on the leachate generation calculations (Section 1.5.1) and leachate extraction system sizing calculations conducted (Section 1.5.4), the anticipated leachate flow shall be established.
- An iterative approach using the Hazen-Williams formula with a variety of internal pipe diameters (4-inch, 6-inch, 8-inch, etc.) should be used to calculate the friction losses for each scenario. The Hazen-Williams formula utilizes roughness coefficients (C-values) which correspond to the force main piping material to be utilized. Alternately, head losses due to friction can be calculated using the Darcy formula. Friction factors used in the Darcy formula can be determined using the Moody friction factor curves.
- Minor losses through bends, fittings, and valves should be accounted for in the overall friction loss. This can be done using the equivalent length method or by calculating the head loss coefficient (K) for each bend or fitting. Typical values for each method are tabulated in the Cameron Hydraulics handbook for various pipe materials and pipe diameters. For the equivalent length method, the tabulated equivalent length for each fitting is added to the actual pipe length and substituted back into the Hazen-Williams or Darcy equations to calculate the overall friction loss. When using the head loss coefficient method, the K value is multiplied by the velocity head yielding the head loss

through that fitting in feet. Those losses are then added to the friction loss through the pipe.

- The velocity of the leachate through the leachate conveyance force main is affected by the friction factor and pipe diameter; therefore these variables must be balanced during the design of the leachate conveyance force main. Flow rate velocity shall be maintained at a minimum of two (2) feet/second to avoid settling of particulates and a maximum of ten (10) feet/second to avoid pipe scouring, as referenced in USACE TM 5-814-2 (1985).
- Static head, the difference in elevation between the pump suction level and the highest proposed point in the leachate conveyance force main, should be calculated. The total dynamic head (TDH), the summation of the friction loss and the static head, should be calculated. The Hazen-Williams calculation provides results for different friction head losses at varying flow rates, while the static head loss is independent of the flow rate.
- Using the design flow rate and the total dynamic head, a pump size should be selected based on manufacturer-specified pump curves. This process is an iterative approach to select the leachate conveyance force main size to balance overall friction loss through the pipe while maintaining operations within design range.
- If multiple pumps are included in the design, as is typical with a multi-cell landfill configuration, each different pumping scenario shall be considered in sizing the pumps and the leachate conveyance force main.
- The velocity of the leachate through the leachate conveyance force main is affected by the friction factor and pipe diameter; therefore these variables must be balanced during the design of the leachate conveyance force main.
- It is recommended that cleanouts be included in the leachate conveyance force main in areas of significant low spots in elevation along the force main profile. Shut-off valves (gate valves or plug valves) should be included at approximately 1000-foot intervals, or other interval as specified by local code, to enable leachate conveyance force main segments to be isolated for repair and maintenance.
- Leachate conveyance force mains should be equipped with air release valves and vacuum break valves or a combination air release/vacuum break valves in the appropriate locations to avoid air blockage and/or to provide a vacuum break.
- It is recommended that a double-walled conveyance pipe should be used whenever leachate is transported outside the limits of the lined landfill area. This applies to gravity flow and force main systems. Use of double-walled leachate conveyance piping is an industry standard, is required in various states, as well as recommended in Bolton, (1995). Although it is recommended that double-contained piping be utilized, the designer may wish to further discuss the alternate use of single-wall piping with the appropriate regulatory authority.

US EPA ARCHIVE DOCUMENT

1.5.7 Leachate Loadout, Treatment, and Disposal

Definition

The landfill leachate loadout, treatment, and disposal system includes the means by which the leachate is loaded or extracted from the leachate storage system into a final treatment and disposal system. Leachate loadout areas may consist of a tanker-truck loading area if discharging leachate at an off-site treatment plant or using trucks for leachate recirculation. Leachate recirculation is defined in Section 1.5.3. The loadout area may also consist of a monitored connection or pump station to convey the leachate into an on-site treatment system, for direct discharge to a sanitary sewer, or for direct discharge to a receiving surface water body under an NPDES permit.

Leachate treatment and disposal system options depend on site-specific conditions and may include the following: on-site treatment and discharge to surface water in compliance with a site-specific NPDES permit, direct discharge to a sanitary sewer, truck hauling to a wastewater treatment plant, leachate recirculation into the landfill, or a combination of these approaches. An additional approach that is commonly used by CCP landfills is to return the leachate to the powerplant for use as plant process water.

Minimum Design Standards

Alabama: There are no minimum design standards for leachate loadout, treatment and disposal for CCP facilities. There are also no minimum design standards for leachate loadout, treatment and disposal for MSW, IW, and C&DD landfills.

Kentucky: There are no minimum design standards for leachate loadout, treatment and disposal for special waste facilities. There are, however, minimum component requirements for leachate loadout, treatment, and disposal for contained landfills (MSW) as specified in 401 KAR 48:080(6)(4g). This regulation generally states that when leachate is discharged to a sediment basin structure, or leachate is treated using an on-site wastewater treatment plant, or other method of discharge is proposed, a KPDES permit shall reflect this provision. When an off-site wastewater treatment plant is used, written documentation shall be provided showing the acceptance of the leachate, including the criteria for disposal at the wastewater treatment plant. The regulation also states that the system shall have a method to measure the quantity of leachate managed at the site.

Tennessee: The minimum design standard for leachate loadout, treatment, and disposal for Class II disposal facilities are specified in TDEC 1200-1-7-.04(4)(a).8. This regulation generally states that collected leachate must be managed in accordance with any other applicable state and local regulations, that the leachate must be sampled and analyzed, and that leachate recirculation into the emplaced waste is acceptable.

Selected Design Standard

The selection of the leachate loadout, treatment, and disposal system is based on site-specific conditions. All potential loadout, treatment, and disposal options should be considered and evaluated on a site-specific basis. The evaluation shall include, but not be limited to, the

US EPA ARCHIVE DOCUMENT

system's technical feasibility, permitting requirements, and relative costs. TVA should make a decision regarding the treatment and disposal methodology best suited for the specific site based on this evaluation.

If discharging to a sanitary sewer or wastewater treatment plant, plant capacity and the potential chemical overload of the treatment plant must be considered. Coordination with the wastewater treatment plant is imperative, including anticipated flows and chemical constituents, which will need to be agreed upon prior to allowing this discharge. Additional discharge permits will most likely be required, governed by state and possibly county-specific regulations. Prior to discharge of the leachate to the sewer or treatment plant, flow control or pretreatment may be required. Other concerns that the sewer or treatment plants may have include pH, corrosivity, odor, retention time, and potential for chemical reaction with sewer system components, among others.

In all cases, the leachate loadout, treatment, and disposal system shall have a method to measure the quantity of leachate managed at the site and the leachate must be sampled and analyzed in accordance with the requirements of the site-specific discharge permits. In addition, the leachate loadout area shall be contained within a secondary containment area to avoid releases while transferring leachate from the conveyance system to the loadout system. Design of the leachate loadout system shall also take into account any applicable Occupational Safety and Health Administration (OSHA) regulations, as well as methods to enable the system to be user-friendly for operators, leachate truck haulers, etc.

1.6 SURFACE WATER RUN-OFF AND EROSION CONTROL

This section discusses the engineering design and performance standards associated with the management of surface water and sediment and erosion control measures. Factors considered with respect to surface water run-off include the design of surface water management structures and sediment basins to convey and manage expected stormwater flows from the landfill.

Factors considered with respect to erosion control measures include minimizing the volume of anticipated soil loss and providing sediment basins and BMPs to manage erosion during construction and development of the landfill.

1.6.1 Soil Erosion Calculations

Definition

Soil erosion calculations for landfill design and for pond closures (in above-grade facilities where the CCPs are emplaced above the perimeter berm, i.e., wet-stacking necessitating a sloped closure system) are performed for three essential reasons. The first reason is to estimate the amount of soil loss from the upper soil component of a landfill closure cap system. A second reason is to understand the quantity of sediment that may accumulate in a sediment basin to estimate maintenance and sediment removal activities. The third reason is to provide erosion and sediment controls construction periods at the landfill and to manage sediment and erosion from soil stockpiles used in the operation of the landfill.

US EPA ARCHIVE DOCUMENT

Soil erosion can become a significant problem on landfill embankments and closure cap systems. If significant soil erosion occurs, the structural integrity of an embankment may become compromised. In addition, soil erosion in the upper soil component of a closure cap system may cause the underlying closure cap layers (clay or geosynthetics) to become uncovered or exposed which may result in slope instabilities and potential releases of CCPs to the environment. In areas where only soil cover is placed above the CCP material, erosion may cause the CCP to become uncovered in areas, resulting in exposed material, additional leachate production, and significant maintenance issues.

Minimum Design Standards

Alabama: There are no minimum design standards for soil erosion calculations for CCP facilities. There are, however, minimum design standards for MSW, IW, and C&DD landfills as specified in ADEM 335-13-4-.20, which states that slopes longer than 25 feet shall require horizontal terraces, of sufficient width for equipment operation, for every 20 feet rise in elevation or utilize other approved erosion control measures.

The Alabama Soil and Water Conservation Committee (SWCC) published a guidance manual, *The Alabama Handbook for Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas* (2003). This guidance manual, while not having the force of law, provides guidance for preventing or minimized the related problems of erosion, sediment and stormwater. It provides a basis for developing sound plans and implementing appropriate measures (BMPs). This handbook is available via the internet at: http://swcc.alabama.gov/pdf/Handbooks&Guides/ASWCC_June_2003_Alabama_Handbook_Construction_E&S_Control.pdf

In addition, the National Resource Conservation Service (NRCS) has state-specific and county-specific technical guides, referred to as Field Office Technical Guides (FOTGs). The FOTGs contain technical information provided by the Alabama NRCS about the conservation of soil, water, air, and related plant and animal resources. The FOTGs are localized so that they apply specifically to the geographic area for which they are prepared. The FOTGs specific to Alabama are available via the Internet at: http://efotg.nrcs.usda.gov/efotg_locator.aspx?map=AL. Referring to Section IV - Practice Standards and Specifications, provides specific details, specifications, detailed requirements and practices for erosion and sediment control, as well as other practices specific to the state. The design guidance provided in these documents are used as the basis for developing sound erosion and sediment control practices for inclusion in the site-specific SWPPP, which is a regulatory requirement of the general construction permit under the state's NPDES program administered by ADEM.

Kentucky: There are no minimum design standards for soil erosion calculations for special waste facilities. There are, however, minimum design standards for surface water calculations for contained landfills (MSW) as specified in 401 KAR 48:080. This regulation generally requires berms of at least one foot in height or two feet in width be placed at various specified intervals for varying slopes to prevent erosion.

Currently there are no state or county-specific technical guidance manuals published for Kentucky. However, the NRCS has state-specific and county-specific technical guides, referred

US EPA ARCHIVE DOCUMENT

to as Field Office Technical Guides (FOTGs). The FOTGs contain technical information provided by the Kentucky NRCS about the conservation of soil, water, air, and related plant and animal resources. The FOTGs are localized so that they apply specifically to the geographic area for which they are prepared. The FOTGs specific to Kentucky are available via the Internet at: http://efotg.nrcs.usda.gov/efotg_locator.aspx?map=KY. Referring to Section IV - Practice Standards and Specifications, provides specific details, specifications, detailed requirements and practices for erosion and sediment control, as well as other practices specific to the state. The design guidance provided in this document can be used as the basis for developing sound erosion and sediment control practices for inclusion in the site-specific stormwater pollution prevention plan (SWPPP), which is a regulatory requirement of the general construction permit under the state's KPDES program.

Tennessee: The minimum design standard for soil erosion calculations for Class II disposal facilities are specified in TDEC 1200-1-7-.04(8)(c).4. This regulation does not provide a specific design standard for these calculations, rather provides general requirements that the final surface of the disposal facility or disposal facility parcel shall be graded and/or provided with drainage facilities in a manner that:

- Minimizes erosion of cover material (e.g., no steep slopes);

- In order to minimize soil erosion, as soon as practicable after final grading, the operator shall take steps as necessary to establish a protective vegetative cover of acceptable grasses over disturbed areas of the site. These steps shall include seeding, mulching, and any necessary fertilization at a minimum, and may include additional activities such as sodding of steeper slopes and drainage ways if such are necessary.

- In addition to the drainage and grading requirements and vegetative cover requirements, the operator shall take other measures as may be necessary to minimize and control erosion and sedimentation (e.g., soil stabilization, sediment ponds) at the site.

TDEC has published the *Tennessee Erosion and Sediment Control Handbook*, (2002), which is designed to provide information to planners, developers, engineers, and contractors on the proper selection, installation, and maintenance of BMPs. The handbook is intended for use during the design and construction of projects that require erosion and sediment controls to protect waters of the state. This handbook is available via the internet at:

http://www.state.tn.us/environment/wpc/sed_ero_controlhandbook/eschandbook.pdf

In addition, the NRCS has state-specific and county-specific technical guides, referred to as Field Office Technical Guides (FOTGs). The FOTGs contain technical information provided by the Tennessee NRCS about the conservation of soil, water, air, and related plant and animal resources. The FOTGs are localized so that they apply specifically to the geographic area for which they are prepared. The FOTGs specific to Tennessee are available via the Internet at: http://efotg.nrcs.usda.gov/efotg_locator.aspx?map=TN. Referring to Section IV - Practice Standards and Specifications, provides specific details, specifications, detailed requirements and practices for erosion and sediment control, as well as other practices specific to the state. The design guidance provided in these documents are used as the basis for developing sound erosion

US EPA ARCHIVE DOCUMENT

and sediment control practices for inclusion in the site-specific SWPPP, which is a regulatory requirement of the general construction permit under the state's NPDES program administered by TDEC.

Selected Design Standard

Cap System Erosion Control: The Revised Universal Soil Loss Equation (RUSLE) shall be used to predict maximum soil loss from the final cap system. The RUSLE was developed by the USDA Agricultural Research Service and takes into account all the factors known to affect rainfall erosion, including climate, soil, topography and vegetation. The allowable soil loss from the cap system, based on calculations with the RUSLE, shall be 2 tons per acre per year. It is recommended that benches or terraces be designed on the cap system to limit erosion such that it is below the maximum soil loss requirement of 2 tons per acre per year. Factors that will affect the erosion rate from the cap system include the slope of the cap system, the drainage length between benches or terraces, and the properties of the cap system cover soil and vegetation. In states that provide specific guidance on the spacing and design of cap system erosion control benches or berms, the specific state design standards shall be followed, and RUSLE shall be used to confirm adequate erosion control is provided. The surface water design criteria for benches on the cap system are discussed in the Surface Water Calculations Section (Section 1.6.2). The 2 ton/acre/year allowable erosion value is provided by the USDA and many state regulations. This is also referenced in Section 11.4 of Qian, Koerner, and Gray (2002).

Erosion Control During Construction: Sediment and erosion control BMPs shall be provided during all construction activities, and to manage sediment from the landfill area and stockpiles during operations. The methodology or BMPs used to provide erosion control shall be based on the details and specifications provided on a state and county basis as presented in the NRCS FOTGs as well as any state or county technical guidance manuals or handbooks specified for the specific states as referenced in the Minimum Design Standards paragraphs of this section.

1.6.2 Surface Water Calculations

Definition

Surface water runoff (non-contact water) that has not come in contact with the CCP materials must be planned for by developing surface water management controls within the site. Sloped areas within the landfill will cause large volumes and higher peak runoff flows from the site than would occur naturally. The surface water runoff should be directed into channels that are capable of carrying most storm loads without overflowing or flooding adjacent areas. Generally, surface water drainage structures are designed based on calculations that take into account a specific design storm event for the particular region. Diversion channels consisting of benches, berms or swales on the side slopes of the landfill are used to intercept runoff before it has a chance to accumulate, flood areas and/or cut erosion gullies. Structures such as riprap letdowns or pipe slope drains are also used to convey runoff (surface water) from the diversion channels to the toe of slope of the landfill where a perimeter surface water collection channel is typically located. The perimeter surface water collection channels convey the surface water to the sedimentation basins to retain any collected sediment prior to discharge of the flow, as controlled by the site-specific NPDES permit.

US EPA ARCHIVE DOCUMENT

Minimum Design Standards

Alabama: There are no minimum design standards for surface water calculations for CCP facilities. There are, however, minimum design standards for surface water calculations for MSW, ILF, C&DD landfills as specified in 335-13-4-.17 - Drainage. This regulation generally requires a run-on and run-off control system for the active and/or closed portions of the landfill during a 24-hour, 25-year storm event. Additionally, on-site drainage structures are required to carry incident precipitation from the disposal site so as to minimize the generation of leachate, erosion and sedimentation.

In addition, as discussed in more detail in the Minimum Design Standards section for design of erosion and sediment controls, the *Alabama Handbook for Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas* (2003), provides guidance for preventing or minimizing the related problems of erosion, sediment and stormwater. Also, the NRCS FOTGs for Alabama provide BMPs for surface water management.

Kentucky: There are no minimum design standards for surface water calculations for special waste facilities. There are, however, minimum design standards for surface water calculations for contained landfills (MSW) as specified in 401 KAR 48:080. This regulation states that surface run-on and run-off ditches pass the 100-year, 24-hour, 100-year storm event, and that all designs shall be verified by the unit hydrograph method of calculation unless another method is approved. Also, the NRCS FOTGs for Kentucky provide BMPs for surface water management.

Tennessee: The minimum design standard for surface water calculations for Class II disposal facilities are specified in TDEC 1200-1-7-.04(2)(i). This regulation generally requires that the run-on and run-off control system be designed, constructed, operated, and maintained such that it is capable of preventing flow onto the active portion of the facility for all flow up to and including peak discharge from a 24-hour, 25-year storm.

In addition, as discussed in more detail in the Soil Erosion Calculations Section (Section 1.6.1), the *Tennessee Erosion and Sediment Control Handbook* (2002), provides information on the proper selection, installation, and maintenance of BMPs. Also, the NRCS FOTGs for Tennessee provide BMPs for surface water management.

Selected Design Standard

Surface water calculations shall be prepared using the Soil Conservation Service (SCS) method (previously by the USDA Soil Conservation Service, now by the Natural Resources Conservation Service (NRCS)). This includes using rainfall frequency curves provided by the NRCS and precipitation data provided by National Oceanic and Atmospheric Administration (NOAA) for the specific region where designing. The calculated surface water runoff will be based on peak flow SCS method, which entails conducting hydraulic modeling using Technical Release 55 (TR-55) method. The TR-55 method was developed by the USDA NRCS Conservation Engineering Division. TR-55 presents procedures to calculate storm runoff volume, peak rate of discharge, hydrographs, and storage volumes required for stormwater reservoirs. TR-55 can be used in conjunction with the Mannings equation to determine the design dimensions, maximum water elevations and flow velocities of the conveyance structure;

US EPA ARCHIVE DOCUMENT

including open channels, culverts, letdowns, etc. Numerous computer software applications are available that assist in surface water calculations using these methods.

All surface water drainage structures, including channels, culverts, and benches on the landfill cap system shall be designed to carry expected flows based on the 24-hour, 25-year storm event for the particular region. This is supported by the Tennessee Class II disposal facility regulations (TDEC 1200-1-7-.04(2)(i)), as well as Federal regulations 40CFR258 Subtitle D. If specific state requirements provide more stringent design standards, the state design requirements shall be followed.

In addition, it shall be required (and is in accordance with Federal regulations 40CFR258 Subtitle D), that run-on controls be utilized to control drainage into and out of the landfill working face (area where CCP is currently being placed). Run-off from the active and/or closed portions of the landfill unit must also be controlled and shall be conveyed to a settling basin or other sedimentation control structure in accordance with the state's NPDES program

The following additional design elements shall be considered during the design of the surface water system:

- Velocity of the surface water flow through the surface water drainage structures must be calculated as referenced above. The velocity must be coordinated with the lining material used for the structure. For example, grass-lined channels are sufficient for low velocity flows, erosion control matting (ECM) or turf reinforced matrix (TRM) is used for medium to high velocity flows, while riprap and hard-liner materials (i.e., concrete, grouted riprap, and fabriform) are used for very high velocities. Specific velocity limits for each of these materials can be obtained in the NRCS FOTGs, state-specific design guidance manuals and/or from the material vendors.

- The dimensions and slope of the surface water drainage structure, as well as the maximum design water level in the structure, must be calculated as referenced above. The dimension of the conveyance structure must also allow for a free-board distance above the maximum design water level. A minimum of 6-inches of freeboard is recommended, or as otherwise specified in the NRCS FOTGs. A minimum grade of 2% shall be used for benches or berms conveying surface water off of the landfill cap system. A 0.5% minimum grade is required for perimeter surface water structures outside of the landfill cap system. However, shallow slopes shall be implemented at the discretion of the design engineer. Slopes of 1% or greater are desirable to achieve maintained positive drainage and provide surface water structures that are constructible.

- Energy dissipation devices, such as riprap protection placed at inlets and outlets to culverts, channels or letdowns; check-dams within channels; gabion baskets or other structural reinforcement at sharp channel bends; etc. must be considered to avoid excessive erosion and scouring.

- The use of non-mechanical gravity-flow surface water conveyance structures is most desirable and recommended. Mechanical conveyance (pumping) of surface water is not

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typically desired, and in some cases not permitted by the regulatory authorities. This is because since the control and conveyance of surface water that is generated by storm events at quantities, durations and times that cannot be well predicted, a system reliant on a mechanical means (pumps) that are susceptible to malfunction and power outages would not be reliable.

Lastly, the methodology or BMPs used to meet the referenced surface water management requirements shall be based on the details and specifications provided on a state and county basis as presented in the NRCS FOTGs as well as the technical guidance manuals or handbooks specified for the specific states as referenced in the Minimum Design Standards paragraphs of this section.

1.6.3 Sedimentation Basin Design

Definition

A sediment basin is a temporary holding pond created by construction of a barrier across a drainageway or by excavating a basin or by a combination of both. The purpose of the sediment basin is to detain sediment-laden runoff from disturbed areas in storage long enough for most of the sediment to settle out of the runoff and be deposited in the basin. This practice applies where erosion control measures are insufficient to prevent excessive off-site sedimentation. The design of the sediment basin is largely based on the storage volume and detention time required to settle out the sediment from the runoff. Also taken into consideration is the amount of sedimentation anticipated during the construction activities.

Minimum Design Standards

Alabama: There are no minimum design standards for sedimentation basin analysis for CCP facilities. There are, however, general requirements for sedimentation basins for MSW, ILF, C&DD landfills as specified in 335-13-4-.17(3). This regulation generally requires that run-off from the active and/or closed portions of the landfill unit must be handled in accordance with NPDES and shall be routed to a settling basin or other sedimentation control structure.

In addition, as discussed in more detail in the Soil Erosion Calculations Section (Section 1.6.1), the *Alabama Handbook for Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas* (2003) provides guidance for preventing or minimizing the related problems of erosion, sediment and stormwater. Specifically, the design of sediment basins is presented on page 295 in the Handbook. Also, the NRCS FOTGs for Alabama provide details on the design of sediment basins.

Kentucky: There are no minimum design standards for sedimentation basin analysis for special waste facilities. There are, however, minimum design standards for sedimentation basin analysis for contained landfills (MSW) as specified in 401 KAR 48:070(2)(4). This regulation generally requires that sediment basins be designed such that the storage volume and principal spillway do not cause the emergency spillway to discharge during the 25-year, 24-hour storm event. Additional requirements include that the emergency spillway is capable of passing a 100 year, 24

US EPA ARCHIVE DOCUMENT

hour storm even with no flow overtopping the structure and that the minimum sediment storage volume be provided for 1-year prior to requiring maintenance to restore the design volume.

Also, the NRCS FOTGs for Kentucky provide details on the design of sediment basins.

Tennessee: The minimum design standard for sedimentation basin analysis for Class II disposal facilities is specified in TDEC 1200-1-7-.04(2)(i). This regulation generally requires the following:

- Sediment basins associated with run-on and run-off control systems must be designed to detain at least the water volume resulting from a 24-hour, 25-year storm and to divert through emergency spillways at least the peak flow resulting from a 24-hour, 100-year storm.
- Sedimentation basins must be emptied or otherwise managed expeditiously after storms to maintain design capacity of the system.
- Run-on and run-off must be managed separately from leachate unless otherwise approved by the Commissioner.

In addition, as discussed in more detail in the Soil Erosion Calculations Section (Section 1.6.1), the *Tennessee Erosion and Sediment Control Handbook* (2002) provides information on the proper selection, installation, and maintenance of BMPs, including sedimentation basins. Specifically, the design of sediment basins is presented on page SB-1 in the Handbook.

Also, the NRCS FOTGs for Tennessee provide details on the design of sediment basins.

Selected Design Standard

The design standard, which is also the Industry standard, is to calculate the surface water runoff based on the peak flow SCS method. For conducting a sediment basin design, this entails conducting hydraulic modeling using TR-55 and Technical Release 20 (TR-20) methods. TR-55 has already been discussed in the Surface Water Calculations section. TR-20 was developed by the USDA NRCS Hydrology Branch and is now available as a WinTR20 version. TR-20 provides a hydrologic analysis of a watershed under present conditions. Output consists of peaks and/or storm hydrographs. Subarea surface runoff hydrographs are developed from storm rainfall using the hydrograph, drainage areas, times of concentration, and SCS runoff curve numbers. Instructions to develop, route, add, store, divert, or divide hydrographs are established to convey floodwater from the headwaters to the watershed outlet or in this case a sedimentation basin. Other computer programs, for example HydroCAD, can be used to incorporate TR-55 and TR-20 modeling.

The hydraulic modeling shall be conducted to design the sediment basin to detain at least the surface water volume resulting from a 24-hour, 25-year storm and to divert through the emergency spillways at least the peak flow resulting from a 24-hour, 100-year storm.

US EPA ARCHIVE DOCUMENT

Additional and specific design features for sediment basins, including inlet and outlet structures, shall be based on the state-specific and county-specific requirements (if applicable). Relevant details provided in the NRCS FOTGs may also be used for design guidance.

1.7 IMPOUNDMENT SYSTEMS

As stated in the Ash Impoundments Section (Section 1.1.2), ash impoundment can be defined as a collection of water, wastewater, or liquid-borne materials that is contained by an artificial structure (dam), into which CCPs are placed for disposal or temporary storage. This section discusses seepage analysis design standards for impoundments.

1.7.1 Seepage Analysis

Definition

The movement of water through and underneath impoundments for water or saturated earthen materials is referred to as seepage. All such structures are subject to seepage through and underneath the impoundment and foundation. Seepage can impact the operation and stability of impoundment systems, and seepage control is generally necessary in these systems to prevent excessive uplift pressures, instability of slopes, piping through the impoundment and foundation, and erosion of material by loss into open joints or pore spaces in the foundation.

Minimum Design Standards

Alabama: There are currently no specific minimum design standards for seepage analyses of impoundment systems at waste facilities. However, dam safety regulations are in the process of being developed.

Kentucky: There are no specific minimum design standards for seepage analyses of impoundment systems at waste facilities.

Kentucky regulates dams and dam safety in the state via 401 KAR 4:030, and provides design requirements in Division of Water Engineering Memorandum No. 5 (2-1-75). There are no specific requirements for seepage analyses in these documents.

Tennessee: There are no specific minimum design standards for seepage analyses of impoundment systems at waste facilities.

However, Tennessee provides general requirements for seepage in its dam safety regulations, Chapter 1200-5-7, "Rules and Regulations Applied to the Safe Dams Act of 1973". Requirements related to seepage in this document include:

> 1200-5-7-.07 (5) (d) states: "All dams shall be designed and constructed to prevent the development of instability due to excessive seepage forces, uplift forces, or loss of materials in the impoundment, abutments, spillway areas, or foundation. Seepage analysis for design and inspection during construction shall be in sufficient detail to prevent the occurrence of critical seepage gradients. All dams permanently impounding water shall be constructed with an impoundment toe drain with drain pipes installed to discharge the seepage."

US EPA ARCHIVE DOCUMENT

1200-5-7-.08 (5) (f) requires that seepage control features be presented in the plans for dams projects.

Selected Design Standard

Appropriate seepage analyses shall be performed to estimate the quantity of seepage both through and underneath the impoundment, and to estimate the distribution of pore pressures and seepage velocities within and beneath the impoundment

Seepage analyses should be implemented using appropriate 2-D or 3-D finite element or finite difference computer programs or by manual techniques such as drawing flow nets. It is recommended that seepage analyses be performed by computer analysis. The analyses should be performed by a qualified Geotechnical Engineer or Hydrogeologist with substantial experience in seepage modeling.

Input parameters to the seepage analysis, such as hydraulic conductivity of impoundment and foundation materials, should be based on the judgment of an experienced Geotechnical Engineer or Hydrogeologist and on the results of field and laboratory testing performed as part of the geotechnical investigation and judged to be sufficient by the qualified and experienced Geotechnical Engineer responsible for the analysis.

The following points shall be considered for seepage underneath and through impoundments and associated embankments:

- Impoundments and associated embankments shall be configured to mitigate all detrimental effects of seepage underneath and through the structure, such as excessive uplift pressures, piping (internal erosion), instability, sloughing, or removal of material by dissolution or erosion.
- All potential modes of piping and/or uplift for a given embankment should be considered in the evaluation on a case by case basis. These modes include , but are not necessarily limited to the following:

  (1) Upward seepage emerging at the downstream or exterior toe.

  (2) Heave where a low-permeability layer overlies a more pervious stratum.

  (3) Seepage emerging along the downstream or exterior slope or along some other sloping downstream surface.
- The evaluation of piping potential for the assessment or design of an impoundment or associated embankment should be conducted by an experienced engineer and consider several factors including, but not limited to: subsurface conditions and soil types/characteristics, embankment geometry and materials zoning, and exit gradients. There is not a universally accepted criteria for assessing the factor of safety against piping, and piping should be considered on a case by case basis.
- For piping mode (1), upward seepage emerging at the downstream or exterior toe, the critical gradient factor of safety method is commonly used whereby the factor of safety is computed as follows:

US EPA ARCHIVE DOCUMENT

$$FS_{eg} = \frac{i_{cr}}{i_e}$$ where,

FS_{eg} = factor of safety for vertical exit gradient

i_e = maximum exit gradient computed from seepage modeling

i_c = critical gradient causing flotation of the impoundment foundation material.

$$i_c = \frac{G_s - 1}{1 + e}$$

G_s = specific gravity of foundation soils

e = void ratio of foundation soils

USACE Engineering Manual EM 1110-2-1901 (1993, p. 4-24 states: "Investigators have recommended ranges for factor of safety for escape gradient, FSg = i_{cr}/i_e from 1.5 and [sic] 15, depending on knowledge of soil and possible seepage conditions. Generally, factors of safety in the range of 4-5 (Harr 1962 and 1977) or 2.3-3 (Cedergren 1977) have been proposed." The U.S. Bureau of Reclamation (USBR) Design Standard No. 13, Chapter 8, page 8.8 provides more specific guidance as follows: "The heterogeneity of soils in nature, the progressive and hidden deterioration that can occur from seepage flow, and the sparse data base from appropriate testing justify a conservative safety factor (SF=4) for design of exit gradient protective features."

Selection of an acceptable minimum FSeg against piping should be on a case by case basis at the discretion of an experienced engineer. Based on the literature, a FS_{eg} greater than or equal to 4 should provide reasonable guidance as an acceptable minimum factor of safety for this piping mode, providing it is used in conjunction with other factors and sound engineering judgment. Subsurface conditions and piping susceptibility of soils must be taken into account when assessing the adequate factor of safety. Highly piping susceptible soils such as nonplastic cohesionless silts and fine sands, poorly graded, or gap-graded soils, and dispersive clays may justify a higher minimum factor of safety. Conversely if the engineer has adequate and reliable information that the soils and site conditions have a low susceptibility to piping, the engineer may decide to accept a lower FSeg value

- For piping mode (2), heave where a low-permeability layer overlies a more pervious stratum, the Natural Resources Conservation Service (NRCS) Soil Mechanics Note No. 7 (1979) states: :"To be safe against uplift or blowout a suitable factor of safety is needed. A value of 1.5 to 2 is suggested with the higher values to be used on the

US EPA ARCHIVE DOCUMENT

more piping prone materials. Using an effective stress analysis the FS_h value should be computed as:

$$FS_h = \frac{Z\gamma b}{(ho - htw)\gamma w}$$ where,

FS_h = factor of safety against heave

Z = thickness of upper low-permeability layer

γb = buoyant unit weight of soil

ho = piezometric head in lower pervious layer at toe

htw = tailwater head at toe

- For piping mode (3), seepage emerging along the downstream or exterior slope or along some other sloping downstream surface, there is no generally accepted guidance for evaluating the minimum factor of safety against piping and each case must be evaluated individually by an experienced engineer..
- Where the above factor of safety cannot be met, engineering features (e.g. filters) meant specifically to control exit gradients and their detrimental effects shall be incorporated into the impoundment design. The state of the practice for design of new water retention embankments includes such engineering features in any embankment which woulf jeopardize human life it if was to fail.

The following points shall be considered for seepage underneath and through impoundments and associated embankments:

- The impoundment/embankment cross-section shall not be configured such that the predicted phreatic surface exits at a point along the downstream slope of the impoundment in an uncontrolled manner. The design of all impoundments should include pervious drainage features (such as toe drains, inclined or chimney drains, rockfill toes, etc.) meant specifically to collect and manage seepage flows passing through the impoundment structure's cross-section and discharge these flows safely away from the structure. Appropriate filtering media or features shall be provided between the embankment fill materials and/or foundation materials, and the pervious drainage features to preclude piping or migration (internal erosion) of soil particles.
- Design of seepage control features and filter media should be in general accordance with recommendations given USACE EM 1110-2-2300 General Design and Construction Considerations for Earth and Rock-fill Dams, Appendix B, (2004) or other similar references. Filter media shall be designed such that all interfaces between embankment soils, foundation materials, filter material, and drain material across which water can flow are filter-compatible. Discharge capacities of seepage control features should be established based on the maximum steady-state seepage rates estimated in the analysis

US EPA ARCHIVE DOCUMENT

multiplied times a factor of safety, tyoically between 3 and 1o, depending on the confidence in the analysis..

- In addition to pervious seepage control features, planning and design of all impoundments/embankments should also formally consider the applicability of using impervious structures meant to control, limit, or direct seepage – such as cores, cut-off walls, grout curtains, etc. The results of these considerations should be reported in the design documentation.

1.7.2 Impoundment Stability Analysis

1.7.2.1 Deep-Seated Stability Analyses (Static)

Definition

The definition of deep-seated stability analyses as it applies to impoundment systems is generally the same as for landfill systems, as described in the Slope Stability Analysis Section (Section 1.4.2).

Minimum Design Standards

There are no specific minimum design standards for static stability analyses of impoundment systems at waste facilities in any of the three states. Related guidance and regulations for landfill facilities were described in the Slope Stability Analysis Section (Section 1.4.2), and could generally also apply to impoundment systems.

Some states have dam safety regulations, which may also apply to impoundment systems. Requirements are summarized below:

Alabama: Alabama currently does not have any dam safety regulations. However, regulations are in the process of being developed.

Kentucky: Kentucky regulates dams and dam safety in the state via 401 KAR 4:030, and provides design requirements in Division of Water Engineering Memorandum No. 5 (2-1-75). There are no specific requirements for stability analyses in these documents.

Tennessee: Tennessee provides its dam safety regulations in Chapter 1200-5-7, "Rules and Regulations Applied to the Safe Dams Act of 1973". There are no specific requirements for stability analyses for new dams in this document. For existing dams 1200-5-7-.06 (1) states: "All dams shall be stable. There shall not be excessive cracks, sloughing, seepage or other signs of instability or deterioration. In cases where the stability of the dam is questionable, it shall be the responsibility of the owner to either demonstrate to the Commissioner that the dam is stable or drain the reservoir and remedy the unstable condition prior to refilling the reservoir."

Selected Design Standard

Due to the nature of their construction and use, the stability of impoundment systems should be evaluated using analysis cases and criteria that are accepted for the design of dams. For static stability evaluations of impoundment systems, methodologies, guidance on use of total or effective strength parameters, and minimum factors of safety for design given in the USACE EM-1110-2-1902 (2003) should be utilized.

US EPA ARCHIVE DOCUMENT

The stability analysis cases to be considered should include, at a minimum:

1. Construction and End of Construction Conditions: This analysis considers the newly completed impoundment and/or one or more critical configurations of the impoundment during construction. The analysis assumes that the impoundment has been constructed such that pore pressures in low permeability materials both within and underneath the impoundment have not had sufficient time to come to equilibrium. Thus, drained (effective) strengths are input into this analysis for free-draining materials (typically those with permeability of $1x10^{-4}$ cm/sec or higher) and undrained (total) strengths are input into this analysis for materials that drain slowly (typically those with permeability less than $1x10^{-7}$ cm/sec). For materials with permeability intermediate to these values, analysis should be implemented using both undrained and drained strengths, and the more conservative assumption should be used for design. The effects of staged impoundment construction or any other special drainage enhancing features (such as wick drains) should be included in this analysis, as and if appropriate.

2. Long-Term Stability Conditions: This analysis considers the impoundment under normal operating conditions that will exist a sufficient length of time after construction. It is assumed that pore pressures have had sufficient time to reach equilibrium both within and underneath the impoundment and steady-state seepage and/or hydrostatic conditions corresponding to the normal operating pool have developed. The steady state seepage and pore water pressure distributions determined in the Seepage Analysis Section (Section 1.7.1) should be used in this analysis. Drained (effective) shear strengths should be used for all materials.

3. Sudden Drawdown Conditions: This analysis considers a condition in which the impoundment has been under a long term stability condition for some time and then is subjected to a sudden lowering of the pool elevation. This condition represents a critical scenario for the upstream slope of the impoundment, due to loss of stabilizing hydrostatic pressure from the reservoir that has been drawn down. It should be assumed that drawdown is very fast, and no drainage occurs in materials with low permeability. Materials with values of permeability greater than $1x10^{-4}$ cm/s can be assumed to drain during drawdown, and drained strengths are used for these materials. The three-stage methods and procedures given in Appendix G of USACE EM-110-2-1902 should be used for implementing stability analyses for this condition.

   If the impoundment structure being designed is to be situated adjacent to or in close proximity to another impoundment, consideration shall also be given to the effects of sudden drawdown of the adjacent impoundment on the structure being designed.

US EPA ARCHIVE DOCUMENT

Table 1.7.2-1 (reproduced from EM-1110-2-1902, 2003) provides additional summary information for the static stability analysis conditions described above.

Table 1.7.2-1
Cases to be analyzed for static stability of impoundment systems
(Taken from USACE EM-1110-2-1902 "Slope Stability", Table 2-1, 2003)

Shear Strengths and Pore Pressures for Static Design Conditions

Design Condition	Shear Strength	Pore Water Pressure
During Construction and End-of-Construction	Free draining soils – use drained shear strengths related to effective stresses[1]	Free draining soils – Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations if there is no flow, or using steady seepage analysis techniques (flow nets or finite element analyses).
	Low-permeability soils – use undrained strengths related to total stresses[2]	Low-permeability soils – Total stresses are used; pore water pressures are set to zero in the slope stability computations.
Steady-State Seepage Conditions	Use drained shear strengths related to effective stresses.	Pore water pressures from field measurements, hydrostatic pressure computations for no-flow conditions, or steady seepage analysis techniques (flow nets, finite element analyses, or finite difference analyses).
Sudden Drawdown Conditions	Free draining soils – use drained shear strengths related to effective stresses.	Free draining soils – First-stage computations (before drawdown) – steady seepage pore pressures as for steady seepage condition. Second- and third-stage computations (after drawdown) – pore water pressures estimated using same techniques as for steady seepage, except with lowered water level.
	Low-permeability soils – Three-stage computations: First stage--use drained shear strength related to effective stresses; second stage--use undrained shear strengths related to consolidation pressures from the first stage; third stage--use drained strengths related to effective stresses, or undrained strengths related to consolidation pressures from the first stage, depending on which strength is lower – this will vary along the assumed shear surface.	Low-permeability soils – First-stage computations--steady-state seepage pore pressures as described for steady seepage condition. Second-stage computations – total stresses are used; pore water pressures are set to zero. Third-stage computations -- same pore pressures as free draining soils if drained strengths are used; pore water pressures are set to zero where undrained strengths are used.

US EPA ARCHIVE DOCUMENT

Minimum Factors of Safety: Impoundment systems should be designed to meet the factors of safety given in Table 1.7.2-2 below.

Table 1.7.2-2

Minimum Factors of Safety for Static Stability Analyses of Cases for Impoundment Systems.

(Taken from USACE EM-1110-2-1902 "Slope Stability", Table 3-1, 2003)

Minimum Required Factors of Safety: New Earth and Rock-Fill Dams

Analysis Condition[1]	Required Minimum Factor of Safety	Slope
End-of-Construction (including staged construction)[2]	1.3	Upstream and Downstream
Long-term (Steady seepage, maximum storage pool, spillway crest or top of gates)	1.5	Downstream
Maximum surcharge pool[3]	1.4	Downstream
Rapid drawdown	1.1-1.3[4,5]	Upstream

[1] For earthquake loading, see ER 1110-2-1806 for guidance. An Engineer Circular, "Dynamic Analysis of Embankment Dams," is still in preparation.

[2] For embankments over 50 feet high on soft foundations and for embankments that will be subjected to pool loading during construction, a higher minimum end-of-construction factor of safety may be appropriate.

[3] Pool thrust from maximum surcharge level. Pore pressures are usually taken as those developed under steady-state seepage at maximum storage pool. However, for pervious foundations with no positive cutoff steady-state seepage may develop under maximum surcharge pool.

[4] Factor of safety (FS) to be used with improved method of analysis described in Appendix G.

[5] FS = 1.1 applies to drawdown from maximum surcharge pool; FS = 1.3 applies to drawdown from maximum storage pool.

For dams used in pump storage schemes or similar applications where rapid drawdown is a routine operating condition, higher factors of safety, e.g., 1.4-1.5, are appropriate. If consequences of an upstream failure are great, such as blockage of the outlet works resulting in a potential catastrophic failure, higher factors of safety should be considered.

The standards related to selection of material parameters, critical cross-sections, and slope stability analysis methods given in Section 1.4.2 for static stability analyses of landfill systems also apply to static stability analysis for impoundment systems. Additional considerations for impoundment systems are listed below

- Failure surface geometries considered in the analysis should include toe, basal geometries passing through the impoundment foundation and block failure geometries. Consideration should be given to preferential sliding planes such as along drainage layers or along interfaces anticipated to have weaker shear strength than the general impoundment fill.
- Earthen structural components of impoundment systems (such as perimeter dikes) are generally expected to be of lower height and extent than earthen components of landfills. Therefore, the properties of the foundation materials underneath impoundments may have greater influence on stability analyses. Careful consideration of variations of the foundation materials along the alignment of impoundments should be included when selecting critical cross-sections for stability analysis.
- Total and effective stress shear strength properties for stability analyses of impoundment systems should be established based on site specific field and laboratory testing, as described in the Site Investigation Section (Section 1.3.1).

US EPA ARCHIVE DOCUMENT

1.7.2.2 Seismic Stability Analyses

Definition

Seismic stability analysis for impoundment systems includes evaluation of liquefaction potential and seismic deep-seated stability. These terms were defined previously for landfill systems in Sections 1.4.2.2, and these definitions also generally carry over to impoundment systems.

Minimum Design Standards

There are no specific minimum design standards for seismic stability analyses of impoundment systems at waste facilities in any of the three states. Related guidance and regulations for landfill facilities were described in Section 1.4.2.2, and could generally apply to impoundment systems.

Some states have dam safety regulations, which may also apply to impoundment systems. Requirements are summarized below:

Alabama: Alabama currently does not have any dam safety regulations. However, regulations are in the process of being developed.

Kentucky: Kentucky regulates dams and dam safety in the state via 401 KAR 4:030, and provides design requirements in Division of Water Engineering Memorandum No. 5 (2-1-75). There are no specific requirements for seismic stability analyses in these documents.

Tennessee: Tennessee provides its dam safety regulations in Chapter 1200-5-7, "Rules and Regulations Applied to the Safe Dams Act of 1973". There are no specific requirements for seismic stability analyses for new dams in this document.

Regarding seismic design of dams, 1200-5-7-.07 (1) (b) states: "All structures other than Category 3 dams constructed before 2008 shall be designed to withstand seismic accelerations of the following intensities: Zone 1 = 0.025g, Zone 2 = 0.05g, Zone 3 = 0.15g. Zones refer to *Geologic Hazards Map of Tennessee* (Miller, 1978). All dams constructed during or after 2008 shall be designed to withstand the peak ground acceleration for an earthquake with a 10% probability of exceedance in 50 years as determined by the United States Geological Survey at the time the construction permit is issued. A different peak ground acceleration may be used if site specific studies using accepted engineering practices determine that a different value is appropriate.

The above magnitude of peak ground acceleration corresponds to an event with an approximate 475-year return period. This is a substantially smaller event than the 2,400-yr return period event that the state requires for design of new landfill facilities, and much lower than the 3,000 to 10,000 year return periods typically used for dams (,more specifically, high hazard dams).

Selected Design Standard

The design seismic event for impoundment systems shall be the same as that described in Section 1.4.2.2.1 for landfill systems and consistent with the design structure.

Liquefaction analyses as described in Section 1.4.2.2.2 for landfill systems should also be performed for impoundment systems.

US EPA ARCHIVE DOCUMENT

The seismic slope stability analyses and seismic deformation analyses presented in Section 1.4.2.2.3 for landfill systems are also applicable to impoundment systems. The seismic analyses should be performed assuming that the fluid level existing within the impoundment at the time of the design seismic event corresponds to the coincident pool elevation, defined as the elevation at which the impounded fluid is expected to be at or below for half of the time during each year.

1.7.3 Settlement Evaluations

Definition

Settlement analysis consists of predicting of total and differential subsurface deformations underneath impoundments and critical structures associated with impoundments (such as drainage features, seepage collection features, etc). Settlements are induced primarily by the self weight of the impoundment and the materials retained by the impoundment.

Minimum Design Standards

There are no specific minimum design standards for settlement analyses of impoundment systems at CCP facilities in any of the three states. Related guidance and regulations described in Section 1.4.3, under landfill systems may generally apply to impoundment systems as well.

Similarly, there are no minimum design standards for settlement analysis in any of the three states within their dam safety regulations.

Selected Design Standard

The recommended methodologies for settlement analyses given in Section 1.4.3 for landfill systems shall also apply to impoundment systems. For impoundment systems, the following additional guidelines are applicable:

- Impoundments shall be designed such that all freeboard requirements are met assuming the maximum computed static settlements. The design should be implemented such that freeboard of at least 2 ft is predicted after including seismic deformations corresponding to the design seismic event.
- Analyses focused specifically on determining settlement profiles underneath drainage structures, piping, spillways and outlet works, and other structures should be performed, and these components should be designed to accommodate the estimated settlements.

1.8 SPILLWAY / DRAINAGE SYSTEMS

Definition

A spillway is an open or closed channel, or both, used to convey water from a reservoir. It may contain gates, either manually or automatically controlled, to regulate the discharge of water. A principal spillway is the first discharge system designed to begin operation after the normal design storage capacity is exceeded. An emergency spillway is a discharge system designed to operate at an elevation above the principal spillway to safely convey discharges that exceed the principal spillway's capacity without jeopardizing the safety of the dam.

US EPA ARCHIVE DOCUMENT

Minimum Design Standards

State and federal regulations for minimum spillway design flow requirements are given in Table 1.9-1.

Table 1.9-1
Comparison of State and Federal Regulations: Spillway Design Flow Requirements

Hazard Category of Dam	Alabama	Kentucky	Tennessee[2]	Federal
High Hazard	N/A	Class C Dams: $P_C = PMP$[1]	Small Dams: 1/2 PMP Intermediate Size Dams: PMP Large Dams: PMP	High Hazard Potential[3]
Moderate Hazard	N/A	Class B Dams: $P_B = P_{100} + 0.40 \times (PMP - P_{100})$[1]	Small Dams: 1/3 PMP Intermediate Size Dams: 1/2 PMP Large Dams: PMP	Significant Hazard Potential[3]
Low Hazard	N/A	Class A Dams: $P_A = P_{100} + 0.12 \times (PMP - P_{100})$ [1]	Small Dams: 100-Year Flood Intermediate Size Dams: 1/3 PMP Large Dams: 1/2 PMP	Low Hazard Potential[3]

Note 1: Criteria are for Freeboard Hydrograph. P denotes 6-hour design precipitation, P_{100} refers to 6-hour, 100-year precipitation, and PMP represents 6-hour Probable Maximum Precipitation.
Note 2: Tennessee dam size classifications:

Category	Storage (acre-feet)	Height (ft.)
Small	30 – 999	20 - 40
Intermediate	1000 – 50,000	41 - 100
Large	50,000+	100+

Note 3: For Federal inflow criteria refer to Federal Guidelines for Dam Safety: Selecting and Accommodating Inflow Design Floods for Dams, U.S. Department of Homeland Security, Federal Emergency Management Agency, October 1998, Reprinted April 2004

US EPA ARCHIVE DOCUMENT

US EPA ARCHIVE DOCUMENT

Selected Design Standard

Flood and freeboard spillway criteria are provided in Table 1.9-2. The remainder of the section presents additional spillway design standards.

Table 1.9-2
TVA Spillway Criteria for Wet Ash Disposal Impoundments

Federal Hazard Category	Spillway Design Flood[1,2]	Minimum Freeboard During Normal Operating Conditions	
		Dam[2]	Upground Reservoir
High	100% Probable maximum flood (PMF) unless a flood inflow condition is identified such that a failure at that flow or larger flows (up to the PMF) will no longer result in unacceptable additional consequences.	Sufficient freeboard shall be provided to prevent overtopping of the top of the dam the design flood and factors such as wind, wave action, ice, ..etc. reservoir.	5 feet
Significant			5 feet
Low			3feet

Note 1: Hydrologic and hydraulic calculations must prove that the combined spillway capacity is sufficient to safely pass the inflow design flood without overtopping the dam. The inflow hydrograph shall be calculated as the flow resulting from the design flood storm event falling on the watershed upstream of the impoundment and the impoundment itself during a 6-hour period.
Note 2: For inflow design flood and freeboard requirements use procedures outlined in Federal Guidelines for Dam Safety: Selecting and Accommodating Inflow Design Floods for Dams, U.S. Department of Homeland Security, Federal Emergency Management Agency, October 1998, Reprinted April 2004.

Spillway structures shall consist of reinforced concrete or steel construction with proper foundations, engineered to withstand with a minimum Factor of Safety of 2.0 for all wind, water, ice, debris and other loads that can reasonably be expected to impinge on the structure during its service life.

Spillways should consist of a single outlet structure serving as both principal spillway and emergency spillway, a principal spillway and a dedicated emergency spillway, or two or more spillways having combined flow capacity sufficient to safely pass the design flood. Spillways consisting of a poorly founded vertical or near vertical outlet pipe with an open end at the desired pool level should be avoided unless they can satisfy all required design criteria.

Outlet works design should adhere to currently accepted engineering practice, and should consider design issues including but not limited to:

- Settlement along conduit profile
- Pipe joint requirements (pipe joint separation)
- Seepage control associated with conduits
- Proper application of spillway materials deemed appropriate with respect to embankment depth, foundation conditions, etc.
- Pipe bedding, cradling and/or encasement
- Energy dissipation and erosion control
- Corrosion protection for ductile iron or other metal pipe

All spillway structures shall be equipped with a means to shut off flow, such as by emplacing stop-logs, or by actuating a gate or valve. Such control features should be located at or near the upstream end of the discharge pipe, near the spillway structure, so that pressurizing the outlet pipe is avoided when valve is closed.

Principal spillways shall be equipped with trash racks designed and built to provide positive protection from clogging of the spillway at any point. The average velocity of flow through a clean trash rack shall not exceed 2 ft/sec with the water elevation in the reservoir 5 feet above the top of the trash rack or at the crest of the emergency spillway, whichever is lower. Velocity will be computed on the basis of net area of opening through the rack.

All spillway structures shall be equipped with personnel access devices meeting pertinent OSHA safety requirements. These may consist of fixed or floating walkways and platforms capable of safely holding a minimum of 1,000 pounds of personnel and equipment without sinking or otherwise becoming unstable. Walkway loadings shall be incorporated into the design of the structure. Guardrails shall be provided to prevent falls into the ash impoundment or the spillway structure.

Whenever possible, discharge conduits should be founded in natural foundation soil at the base of the impoundment dam/dike or at an abutment, and not through the embankment itself. If the conduit must be placed through the dam/dike embankment the design shall include seepage control features such as cutoffs, collars and filter drains. Discharge conduits may not be made of corrugated metal pipe.

Chutes or channels conveying water from the outlet end of discharge conduits should be located in natural ground to the extent possible. Chutes and outlet channels located on the dam embankment should include robust erosion control features such as filters and riprap between them and the surface of the dam to prevent or minimize erosion due to water leakage or spillover.

Pipe conduits shall be of such design as to safely support the total external loads and shall convey flow without rupture or leakage. All pipes shall have the ability to resist corrosion from surrounding soils and impounded materials based on current acceptable testing standards. All pipe conduits shall convey flow at the maximum design velocity without damage to the interior surface.

Up-ground or off stream reservoirs shall be equipped with a passive overflow prevention device that limits the maximum water level by discharging excess water. It shall have sufficient flow capacity to exceed by a minimum of 10% the sum of all pumped inflows plus precipitation runoff to the reservoir. Upground or off stream reservoirs shall be equipped with water level monitoring instruments. If practical, instruments that can be read from a remote location, such as a control panel at the power plant, are recommended. At a minimum, upground reservoirs should be equipped with a staff gage that can be read conveniently from locations readily accessible to power plant personnel.

US EPA ARCHIVE DOCUMENT

1.9 ASH STORAGE POND AND LINER DESIGN CRITERIA

This section describes ash storage ponds and liner materials. Selected design standards are presented for dike construction, vertical separation distance, liner systems, protective drainage cover, liner penetrations, discharge structures, splitter dikes, pond sizing and future dam raising, as they relate to the of design ash storage ponds and liners.

When designing new dams, dikes and appurtenances for wet ash disposal facilities, the designer should refer to published design manuals such as the US Army Corps of Engineers EM-series manuals including EM-1110-2-2300, *Engineering and Design - General Design and Construction Considerations for Earth and Rock-Fill Dams* (2004), and related manuals.

Definition

An ash storage pond is defined as an impoundment formed by earthen dams/dikes built above the pre-existing ground surface, excavation into the existing ground surface, or a combination thereof that is used for temporary or permanent storage of wet-sluiced coal combustion products.

The liner is defined as an impervious or low-permeability layer of soil or man-made geomembrane that separates the saturated wet-sluiced coal combustion products and transport water from the soils and groundwater below and outside of the ash storage pond.

1.9.1 Recommendations for Dike Construction

Construction of the dikes should proceed as zoned earthen embankments, with a low permeability (clay) layer comprising the upstream or shoreline side. Material behind and below the impervious material should be selected to meet filter criteria with respect to the low permeability liner material. On the downstream toe of the dike, a graded filter and toe drain to control seepage is recommended. Filter criteria and filter drain design should adhere to USACE EM 1110-2-2300, Appendix B, Filter Design (2004). The slopes should be no steeper than 3H:1V, providing a factor of safety consistent with the parameters presented in this Document.

The crest of the dike should be wide enough to accommodate a two-way road, corresponding to a 25 to 30 ft wide crest. The road should include geotextile in its base and at least 6 to 9 inches of aggregate (“road stone”) so that medium weight service trucks can use the road without damage.

The impervious core of the dikes, built on the upstream slope, should be thicker at the bottom and thinner at the top, with a minimum of 3-foot thickness at the top. Thickness at the bottom should be determined by geotechnical engineering analysis, but in general the clay thickness should not be less than 1/3 the depth of the water to be stored above.

1.9.2 Vertical Separation Distance above Groundwater

A geologic buffer should be present between the base of the ash pond and the underlying groundwater system. In lieu of geologic buffers to maintain a groundwater separation distance, artificial groundwater controls via gravity draining underdrain systems, capable of lowering the groundwater to create separation between the liner and the seasonal high ground water table may

US EPA ARCHIVE DOCUMENT

be proposed. Such an underdrain system would need to be monitored, sampled, and tested as part of the facility monitoring program, similar to groundwater or surface water monitoring.

1.9.3 Liner System

A composite liner system should be present along the base of the sideslopes of the ash pond. The composite system should be comprised of a synthetic geomembrane in direct contact with a low permeability soil. More specifically, an impervious clay layer (permeability less than 1 X 10^{-7} cm/sec) should be constructed at the bottom of the pond such that the thickness is a minimum of 3 feet, or 1/6 the depth of the water to be stored above it. This ratio can be adjusted/refined using geotechnical engineering analysis. Overlying the clay liner, an HDPE, 60 mil geomembrane (or approved equivalent is recommended based on adequate design and testing for stability during construction and operation.

In general, the bottom of the pond should promote positive drainage to the outlet structure. Pond bottom slopes should be 2% minimum in the direction of liquid flow, with minimum 0.5% slopes in the direction of drainage pipes placed at regular intervals within the protective cover layer above the liner (discussed below).

1.9.4 Protective Drainage Cover

Overlying the composite liner, a protective cover system should be installed to protect the liner during construction and operation as well as facilitate a drainage path below the deposited ash. A drainage blanket of fine to medium sand (or possibly bottom ash/boiler slag) meeting filter criteria with respect to the fly ash, can be constructed at the bottom of the pond, just above the impervious blanket. Slotted or perforated drainage pipe can be embedded in it to drain transport water from the fly ash. The following are generally criteria to achieve this component:

- Protective cover at the pond bottom should be at least 24 inches in thickness, and comprised of a two-layered system.

- The bottom layer should be at least 12 inches thick. The bottom layer of the protective cover should be overlain by a continuously sewn geotextile having an AOS compatible with the upper component of the protective cover.

- The upper layer of the protective cover should be at least 12 inches in thickness. The upper layer of the protective cover should be capable of resisting the erosion force of sluice water as the pond is initially filled. Bottom ash may be used for either component of the protective cover layer, and may be placed at a greater thickness if desired to provide adequate erosion resistance to sluice water anticipated at initial pond filling.

- Protective cover on the pond slopes should satisfy the requirements listed for the pond bottom with the following exceptions:
  - The shoreline of the pond should have erosion protection such as crushed stone or riprap. Size requirements can be determined using wave-fetch design methods.

US EPA ARCHIVE DOCUMENT

The erosion protection material should be bedded on granular filter material or a geotextile fabric.

- In areas where future bottom ash removal can be reasonably anticipated the minimum 24-inch thick protective cover should be overlain with riprap to serve as a physical barrier to excavating equipment. A geotextile separation layer may be needed beneath the rip rap layer.
- Beneath any permanent inlet pipe, having its discharge end lying above any portion of the pond slope, a minimum rip rap thickness of 24 inches of suitably sized rock extending the entire slope length, and a minimum distance of 50 feet out onto the pond bottom is recommended. A geotextile separation layer may be needed beneath the rip rap layer. Alternatively, a concrete slab, consisting of at least a 4-inch thick mesh reinforce concrete slab, having the same surface dimensions recommended for the riprap armoring layer, may be substituted for the riprap.
- All pond slopes should satisfy the shallow translational stability analysis (veneer stability calculation) for the selected liner and protective systems as described elsewhere in this document.

1.9.5 Liner Penetrations

Concrete outfall structures with stop log adjustable outlets should be used in regulating water levels in the pond. Geosynthetic liner materials should be mechanically fastened to the concrete surface using stainless steel flat bars with anchor bolts spaced at 12 inch centers. Alternatively, specially designed HDPE embedment strips may be cast in the concrete surface onto which the HDPE liner component may be continuously welded. No pipe penetrations of the liner are anticipated.

1.9.6 Pond Discharge Structure

The pond discharge should be comprised of a long shallow trough that drains to a central discharge structure. The trough should be designed to reduce flow velocities within the pond and reduce the occurrence of concentrated flow ("short-circuiting"). The main discharge structure should be equipped with water level control facilities such as stop-logs so that the pond water level can be raised as it fills with ash over time. While no specific recommendation is provided for the configuration and/or material selection as this is an element of design, due to problems with corrosion and design life, corrugated metal pipe is not recommended.

1.9.7 Splitter Dikes

If desired, splitter dikes can be constructed within the pond using bottom ash, boiler slag or earth materials to partition the pond into two or more sections. Each section should have sufficient surface area to satisfy settling requirements for the coal combustion product to be disposed of in the pond.

In the event that different water elevations are to be considered for either side of the splitter dike, the dike should be designed prior to construction.

US EPA ARCHIVE DOCUMENT

1.9.8 Pond Size/Configuration

The surface area of the pond should be large in comparison to its depth. Settling equations should be used for pond sizing, using particle size and specific gravity values that are representative of the coal combustion product to be disposed of.

1.9.9 Future Dam Raisings

The potential impacts of future embankment raisings should be considered when planning the geotechnical aspects of ash pond design, such as predicted foundation settlement, slope stability, embankment zoning and geometry, and pipe material selection. When the possibility of future raisings cannot be ruled out, the design of new ash disposal ponds should be sufficiently conservative to support future raising(s).

1.10 VISIBILITY

While there are no specific State or Federal regulations regarding visibility, the following discussion is provided as a recommendation, especially when facilities are sited in locations where they will operate within clear view of residential areas.

It is standard procedure to locate landfills in areas where they are less visible to the surrounding public. Landfills must be designed and situated in accordance with the applicable siting regulations, which typically require a specified buffer distance between the limits of CCP and the property line, as well as from surrounding domiciles. The following is recommended to minimize visibility to the surrounding public:

- Visibility from outside the landfill property in the direction of all sides of the landfill should be obstructed using either constructed berm, landscaping or tall vegetation, to the extent possible;
- Haul roads, stockpiles, operational areas and staging areas should be screened either using a constructed berm, landscaping or tall vegetation, to the extent possible;
- Evaluations such as a line-of-sight study or balloon test should be conducted in the earliest stages of site planning.

Note that it may not be possible to completely obstruct the view of the landfill, especially as the landfill increases in height. The surrounding topography and existing vegetation can sometimes be used to the designer's advantage. Three-dimensional line-of-sight evaluations can be conducted to determine at what point or phase in the landfill life it will become visible to the surrounding community. Other methods, such as balloon testing, may be conducted to determine at what elevation the landfill will be visible to the surrounding community. In some cases, the ultimate height of the landfill will need to be negotiated with the surrounding community, depending on the level of community opposition to the project.

US EPA ARCHIVE DOCUMENT

1.11 ACCESS, PERIMETER, HAUL, & SERVICE ROADS

General Definitions

There are four primary types of roadways that are used in CCP Facilities

The following text defines the design requirements for Access, Perimeter, Haul, and Service Roads as they pertain to CCP facilities.

1.11.1 Access Roads

Definition

Access roads are roads that are used to enter the facility, generally from a public road.

Minimum Design Standards

State/Local: There is no specific state standards in Alabama, Kentucky or Tennessee associated with private roads. Contact the local authorities for specific requirements that may need to be incorporated to the design.

The design standards listed below are based upon the Mine Safety and Health Administration (MSHA) Haul Road Inspection Manual (HRIM) 1999, United States Department of Labor, Mine Safety and Health Administration and Design of Surface Mine Haulage Roads. References to the respective State's Department of Transportation (DOT) design guidance for pavements are included as well.

Selected Design Standard

Road design is based upon factors that must be determined for the specifics of each individual facility. These factors include the following items:

- Vehicle Types-Vehicles that may be utilizing the road should be determined prior to the commencement of design.
- Gross Vehicle Weight (GVW)-GVW shall be established prior to design.
- Traffic Volumes-Projected traffic volumes should be established prior to design commencement.
- Design Speed –Typical Access Road design speeds range from 10 mph to 30 mph. A design speed should be selected and used for design iterations, and adjusted as necessary.
- Road Surface-The desired road surface (pavement, aggregate, etc.) should be established so that a friction factor can be selected for design.
- Road Widths – Haul roads widths should be 3.5 x widest vehicle width for a 2-lane road. See MSHA (HRIM), for recommended road widths based upon the number of traffic lanes and vehicle widths. Additional pavement widening may required around sharp curves. See MSHA (HRIM), Table 15 for widening based upon the curve radius dimension and the GVW.

US EPA ARCHIVE DOCUMENT

Once these criteria are established, the following criteria shall be observed:

- Stopping Sight Distance (SSD) – The SSD may vary between 50 ft to 1700 ft based upon design speed, GVW, road surface friction factor and gradient.
- Vertical Alignment - Maximum grade is restricted to 10% with grades to 15% permitted only for short distances. There are no minimum grade restrictions.
- Horizontal Alignment- The following guidelines pertain to horizontal alignments:
    - Minimum horizontal curve is 130 ft (20mph design speed)
    - Minimum horizontal curve is 300 ft (30mph design speed)
    - Super-elevation is required for curves with a radius less than or equal to 1000 ft.
    - Intersection Sight Distance varies between 225' to 375' depending upon design speeds ranging from 15 mph to 30mph.
- Pavement Design – There are no minimum pavement design standards. State specific DOT design guidelines should be utilized. Soil types should be evaluated and considered for pavement design.
- Cross Section – Pavement cross slope to range between 2% and 4%.
- Drainage Design Criteria
    - Roadside Drainage Ditches – 10-year storm for flow capacity of V-ditch.
    - Roadway Culverts – No minimum requirements are listed for culvert design. Use local DOT guidelines or 10-year storm design, whichever is greater.
    - Culvert Outlet Protection – recommended with no standard listed. Use State specific DOT guidelines.
- Runaway Vehicle Provision – No Minimum Requirements
- Guardrail or Earthen Berms-Required for 2:1 foreslope on adjacent ditches or a fill height of 10 feet or greater.
- Storm Water Pollution Prevention Plan (SWP3) – Specific State and Local requirements for construction and post- construction storm water management should be incorporated into the design.
- Right-of-Way Permits-A permit for constructing an entrance within the public right-of-way may be required. Coordinate with the State specific DOT or locality for more information and for more information regarding this permit.
- Signage-Signs indicating speed, direction of movement (right-of-way), use of headlights, railroad crossings, and hazardous conditions must be readily visible and legible.
- Clearance from overhead power lines- Overhead high-voltage power lines are required to be installed no less than 15 feet above grade. Booms and masts of equipment shall not be operated within 10 feet of an energized overhead power line. For more information see chapter 12 of the HRIM.

US EPA ARCHIVE DOCUMENT

1.11.2 Perimeter Roads

Definition

Perimeter roads are roads to facilitate the operation of the facility, located around and outside of the limits of CCP placement.

Minimum Design Standards

State/Local: There is no specific state standards in Alabama, Kentucky or Tennessee associated with perimeter roads.

Selected Design Standard

Perimeter Road design is based upon factors that must be determined for the specifics of each individual facility. To some degree, the perimeter road design will be governed by the landfill design standards. See Section 1.10.1 "Recommendations for Dike Construction" for further detail. Additionally the following guidelines are recommended for good engineering practice.

- Vehicle Types-Generally these facilities are designed for medium-weight service trucks however be sure to confirm this with the operations personnel.
- Traffic Volumes-Projected traffic volumes should be established prior to design commencement. These roads are generally very low volume.
- Road Surface and Composition- These roads are generally aggregate, varying in thickness from 6 to 12 inches. A 6 inch base material should underlay the aggregate, with a non-woven geotextile placed in between the two layers (10 oz. is typical). All materials should be well compacted according to specification. Soil types should be evaluated and considered for pavement design.
- Road Widths – Perimeter roads typically allow for 2-way traffic, however, one-way traffic may be permitted if frequently spaced turn-arounds are provided and space is of concern. Each lane should be no less than 12 feet wide.
- Cross Section – Pavement cross slope to range between 2% and 4%.
- Drainage Design Criteria-Drainage criteria should match that for the landfill surface drainage requirements.
- Guardrail or Earthen Berms-Required for 2:1 foreslope on adjacent ditches or a fill height of 10 feet or greater.
- Clearance from overhead power lines- Overhead high-voltage power lines are required to be installed no less than 15 feet above grade. Booms and masts of equipment shall not be operated within 10 feet of an energized overhead power line. For more information see chapter 12 of the HRIM.

US EPA ARCHIVE DOCUMENT

1.11.3 Haul Roads

Definition

Haul Roads are roads that are designed to accommodate large trucks with a payload at a design speed with consistent daily (hourly) traffic. Haul roads are used within the footprint of a landfill, used for delivery of material to the working face. These roads are not for public use.

Minimum Design Standards

State/Local: There is no specific state standards in Alabama, Kentucky or Tennessee associated with private haul roads. They are generally the responsibility of the landfill operator to design, build, and maintain.

Selected Design Standard

The Haul Roads used within the footprint of the landfill are the responsibility of the Operator to design, build, and maintain. The design may be based upon the minimum design standards in accordance with MSHA. The Operator may be required to submit the typical Haul Road section to TVA for evaluation, however, ultimately, it is their responsibility to ensure its integrity and functionality. The Haul Road design should be based on the following criteria at a minimum:

- Vehicle Types
- Gross Vehicle Weight (GVW)
- Traffic Projections
- Desired Design Speed
- Road surface-Soil types should be evaluated and considered for pavement design.

Vehicle types, GVW including payloads and traffic projections should be selected to optimize hauling efficiency. Road surface design should consider costs, dust control, vehicle tires and friction factors.

1.11.4 Service Roads

Definition

Service Roads are roads that allow maintenance vehicles to access certain facilities on a periodic basis. Service Roads are not intended to carry payloads within the facility.

Minimum Design Standards

State/Local: There are no specific minimum design standards for private service road designs in any of the three states.

Selected Design Standards

Road design is based upon factors that must be determined for the specifics of each individual facility. These factors include the following items:

US EPA ARCHIVE DOCUMENT

- Vehicle Types-Service Roads shall be designed for maintenance vehicles (pickup trucks, gators, etc.)
- GVW-Gross Vehicle Weight shall be established prior to design.
- Traffic Volumes-Projected traffic volumes should be established prior to design commencement. Volumes on Service Roads are assumed to be extremely low.
- Design Speed –Typical Service Road design speeds range from 10 mph to 30 mph. A design speed should be selected and used for design iterations, and adjusted as necessary.
- Road Surface-The desired road surface (pavement, aggregate, etc.) should be established so that a friction factor can be selected for design. Generally, Service Roads are aggregate, dirt, or reinforced grass.
- Service Road Widths – One lane service roads shall be 9 feet wide (min), or equal to the maximum vehicle width. Two lane service roads shall be 18 feet wide (min).

Once these criteria are established, the following criteria shall be observed:

- Stopping Sight Distance (SSD) – The SSD may vary between 50 ft to 1700 ft based upon design speed, GVW, road surface friction factor and gradient.
- Vertical Alignment - Maximum grade should be restricted to 10% with grades up to 15% only for short distances. There are no minimum grade restrictions.
- Horizontal Alignment- The following guidelines pertain to horizontal alignments:
    - Minimum horizontal curve is 50 feet.
    - Super-elevation is not required.
    - Minimum Intersection Sight Distance varies between 225' to 375' depending upon design speeds ranging from 15 mph to 30mph. These values should be compared to the State DOT standards and designed based upon the most stringent requirements.
- Pavement Design – There are no minimum requirements for pavement design. A geotechnical evaluation of the pavement subgrade is recommended to determine the bearing capacity referred to as the CBR. The pavement section should be a minimum section based on soil conditions (CBR value), groundwater, and level of maintenance desired by TVA.
- Cross Section and Drainage Design – Allow for positive drainage.
- Guardrail or Earthen Berms-Required for 2:1 foreslope on adjacent ditches or a fill height of 10 feet or greater.
- Storm Water Pollution Prevention Plan (SWP3) – Specific State and Local requirements for construction and post- construction storm water management should be incorporated into the design. If the road is hard-surfaced, it will be considered
- Signage-Signs indicating speed, direction of movement (right-of-way), use of headlights, railroad crossings, and hazardous conditions must be readily visible and legible.
- Clearance from overhead power lines- Overhead high-voltage power lines are required to be installed no less than 15 feet above grade. Booms and masts of equipment shall not be

US EPA ARCHIVE DOCUMENT

operated within 10 feet of an energized overhead power line. For more information see chapter 12 of the HRIM.

1.12 SURVEYING

Definition

Land surveying is the detailed study or inspection, as by gathering information through observations, measurements in the field, verbal or written inquiry, or research of legal instruments, and data analysis in the support of planning, designing, and establishing of property boundaries. It involves the re-establishment of cadastral surveys and land boundaries based on documents of record and historical evidence, as well as certifying surveys (as required by statute or local ordinance) of subdivision plats/maps, registered land surveys, judicial surveys, and space delineation. Land surveying can include associated services such as mapping and related data accumulation, construction layout surveys, precision measurements of length, angle, elevation, area, and volume, as well as horizontal and vertical control surveys, and the analysis and utilization of land survey data. Expectations prior to commencement of surveying are established in regards to accuracy, datum, field equipment, hardware and software and other related requirements. These expectations can vary from project to project, state to state or within surveying sub disciplines.

Surveying and Mapping for Design and Plan Development

Field survey data shall be used to create mapping or supplement existing mapping. When applicable, determinations regarding level of supplemental survey shall be established prior to project start. Criteria shall be determined prior to map creation regarding the level of detail required. Detail shall meet or exceed minimum criteria requirements. Supervisory control of all survey and map efforts shall be by a registered professional surveyor in the State in which surveys are performed and maps created. Quality assurance / quality control measures shall be in place to ensure proper map development procedures are in use.

Key Components of Surveying Control

Key components of surveying are the establishment of control in which all future survey data will be based. Control points shall be durable in the environment they are set and uniquely and easily identified. Control points shall have specific northing, easting and elevation information attributes. From these control points, data is gathered and a data base is created. Supervisory control of all survey efforts shall be by a registered professional surveyor in the State in which surveys are performed. Quality assurance / quality control measures shall be in place to ensure proper field and office procedures.

Minimum Surveying Standards

Alabama: As indicated by statute, or as established prior to project start.

Kentucky: As indicated by statute or as established prior to project start.

Tennessee: As indicated by statute or as established prior to project start.

US EPA ARCHIVE DOCUMENT

Accuracy Requirements & Measurement Specifications

Every determination of distance shall be made either directly or indirectly in such a manner that the linear error in the distance between any two points (not necessarily adjacent points) shall not exceed the reported distance divided by ten thousand (allowable linear error = reported distance divided by ten thousand) and every angular measurement shall be made in such a manner that the allowable (directional) error, in radians, shall not exceed the allowable linear error divided by the reported distance (allowable (directional) error = allowable linear error divided by reported distance). When the reported distance is less than two hundred feet, the linear error shall not exceed 0.02 feet. The reported distance is the distance established by the survey. The lengths and directions of the lines shall be specified so that the mathematical error in closure of the property boundary does not exceed 0.02 feet in latitudes and 0.02 feet in departure. Surveys performed using metric measurements shall utilize the metric equivalents based upon the U.S. survey foot conversion factor.

US EPA ARCHIVE DOCUMENT

2.1 BASIS OF DESIGN REPORT

2.1.1 Definition

The Basis of Design (BOD) Report shall be developed for each specific design related project and is intended to supplement the Construction Documents. The BOD report is intended to document the design criteria and minimum standards utilized for each specific project. This guidance document shall be utilized as the initial basis for the BOD but contains only the specific standards and assumptions related to each specific project. The BOD report shall also provide relevant calculations utilized in the design.

The BOD report shall at a minimum include the following:

1. Project Description – defining the scope of work, the location and types of the facilities, purpose or intentions of the proposed improvements and the general design criteria established (i.e. TVA Guidance Document, Federal Guidelines, State or Local Guidelines)

2. Existing Site Conditions – description of the existing site and facility(s) conditions involved in the project and the associated problems necessitating the required improvements.

3. Proposed Improvements – description of the proposed improvements to each facility within the project.

4. Design Criteria – listing of the established design criteria and assumptions utilized in the design of the proposed improvements. The list should be separated into facility types or design components (i.e. Stability Analysis, Surface Water Run-off and Erosion Control, etc.) and identify the sources of each criteria listed.

5. Design Exceptions or deviations from the recommended minimum standards. This should also include an explanation or justification of the deviations from the minimum standards. Note: These should be identified as early as possible or red flagged and discussed with TVA prior to design development.

6. Design Narratives – a description or brief narrative of each design related component including the following as they pertain to the proposed improvements:

    a. Stability Analysis

    b. Leachate Management System

    c. Surface Water Run-off and Erosion Control

    d. Embankment systems

    e. Spillway / Drainage Systems

US EPA ARCHIVE DOCUMENT

f. Haul and Service Roads

g. Special Factors

Note: Spcific calculations should be referenced and included in Appendices.

7. Construction Cost Estimate and Schedule

8. Permits – A listing of the required design and construction permits necessary for the project completion.

9. Construction or Implementation Plan - a brief description of the major construction components and a brief implementation plan identifying services or facilities that are required to be maintained during construction. Also include a list of long lead time procurement items.

10. Operational features - a brief of the operating and maintenance requirements of the proposed facilities.

11. Appendix – Supplemental Reports (i.e. Geotechnical Report), Detailed Calculations or modeling output, Catalogue Cuts of proposed equipment or materials specified in the design and pertinent correspondence with TVA or regulatory agencies.

2.2 CALCULATION DOCUMENTATION

The following section describes the preparation and documentation of calculations. All calculations shall be prepared and submitted to TVA using the following guidelines:

- General Information - Be neat and legible. Use a standard computation pad if performing hand-calculations and include a calculation cover sheet and table of contents. A heading should be included on all pages to reflect project, consultant name, name of person performing calculation, reviewer's name, and date. All calculations should be grouped together for various portions of a project and submitted electronically to TVA.
- Objectives - Briefly state the description and purpose of the calculation and where the result is used (e.g., drawings, specifications, equipment structure). Use sketches for clarity.
- References – List the references (reports, technical papers, codes, textbooks, etc.) on which the calculation(s) is based.
- Inputs - Define all parameters used in the calculation. This includes items such as geometry, material properties, and symbols not commonly used. Give references for values, properties and coefficients used.
- Assumptions - List all pertinent engineering assumptions made, and the basis for these assumptions.

US EPA ARCHIVE DOCUMENT

- Methodology – Provide a narrative explaining the methodology used to implement the calculation. Show sources or derivations of equations not commonly used. Give references for methods applied. Include the name, source and revision of any technical software used to assist in performing the calculation. State the criteria for judging results, including regulatory approaches.

- Results – Provide all results pertinent to the calculation. Explain the results in a clear manner. Compare results to minimum acceptable standards, codes or commonly applied design practice.

- Conclusions – Provide a narrative interpreting the results with respect to their end-use. All recommendations concluded from the calculations should be stated in a separate report.

2.3 TRIGGER POINT MEMORANDUM

Definition

Trigger Points are design elements considered critical to a facility's integrity. Any element or specific design criteria of an engineering project considered critical to the stability or function of the facility shall be identified as a trigger point. The following section outlines a TVA strategy to identify and provide the extra attention required to manage trigger points at all TVA CCP facilities.

Selected Standard

For all major capital projects at CCP facilities, including all major permitting and construction projects, the design engineer shall develop and submit a trigger point evaluation document to TVA in the form of a brief technical memorandum. At a minimum, the trigger point memorandum shall include:

1. An introduction of the project;
2. The definition and identification of trigger points specific to the project; and,
3. Recommendations for each identified trigger point. Recommendations shall include:
    - Additional measures that may be taken to verify construction in accordance with the design specifications;
    - Recommendations to be incorporated into the inspection or maintenance programs of each identified trigger point;
    - Identification of potential problems for each trigger point; and,
    - Development of an action plan to mitigate the trigger point if potential problems are identified.

When necessary, figures or specific references to the design documents (drawings or specifications) shall be provided to aid in the identification and discussion of each trigger point. The trigger point memorandum shall be provided to TVA at the conclusion of the engineering design process and be revisited at the close of permitting and construction activities. A meeting should be held between TVA and the design engineer to discuss and clarify the findings of the trigger point memorandum as needed.

US EPA ARCHIVE DOCUMENT

Typical trigger points for CCP facilities include, but are not limited to:

- **Material strength or construction requirements**, such as compaction or minimum material interface shear values required for the stability of the facility.
- **Critical design assumptions** that should be confirmed during construction or revisited in future design projects.
- **Systems designed to stabilize the facility**, including wick drains or underdrain systems which must function properly over the life of the facility.
- **Design elements that may be impacted by potential settlement**, such as liner and leachate collection systems, final cap systems, and structural foundations.
- **Specific areas of concern**, such as spillways or critical embankment areas.
- **Design elements requiring critical maintenance or inspection**, such as piping systems, pumping systems, embankments, or spillways critical for the facility to function as designed.

2.4 PERMIT REGULATORY REQUIREMENTS SUMMARY

A Permit Regulatory Requirements Summary shall be provided on a permit-by-permit basis by the Engineer preparing the specific permit. The Permit Regulatory Requirements Summary shall consist of the following at a minimum:

- a compilation table or summary of the key regulatory requirements by the permit,
- a description of inspections/monitoring requirements,
- a description of documentation and reporting requirements, and
- a reporting schedule.

This Permit Regulatory Requirements Summary shall be prepared by the design engineer upon receipt of the final permit. The Permit Regulatory Requirements Summary is provided for internal use by TVA and shall be regularly reviewed and updated.

2.5 OTHER RELATED DOCUMENTATION

In the preparation of design and permit documents, additional requirements are necessary. As such, the following additional plans, calculations, and reports shall be required:

1. For all permits, the following plans shall be prepared and submitted to TVA as required by regulations or in addition to:
    a. QA/QC Plan: A plan documenting the quality control items and acceptable values.

US EPA ARCHIVE DOCUMENT

b. Waste Characterization Plan: A plan documenting the chemical characteristics of the CCP materials included in the permit as well as testing procedures and frequency.

c. Operations Plan: A plan documenting the key aspects of the operation of the facility such that the operation remains in compliance with the permit.

d. Groundwater Monitoring Plan: A plan documenting the method, frequency, data collection and processing, reporting, etc. related to groundwater quality assessment. This plan shall be prepared with the Environmental Support Group.

2. For all construction documents, the following documents shall be prepared and submitted to TVA as required by regulations, contract, or in addition to:

a. CQA requirements: Within the technical specifications for the project, construction quality assurance testing requirements shall be provided with the understanding that these items will be monitored and recorded during the construction.

US EPA ARCHIVE DOCUMENT

This section lists the requirements of the regulations. In general, the specific requirements are not identified; and a reference to the full text of the regulation is presented instead

3.1 ENGINEERING CERTIFICATION

This section defines the certification requirements for dry ash landfills and wet ash ponds for each of the three states where TVA facilities are located. During the design, permitting, construction and operation of both permitted ash landfills and ash impoundments there are specific instances whereby a state licensed engineer is required to provide written certifications. This section provides information relative to what certifications are required in each of the states.

3.1.1 Dry Ash Landfills

Minimum Design Standards

Alabama: General design standards for solid waste disposal facilities are contained in the ADEM administrative code §335-13-4-.11 that references §§335-13-4-.12 through 335-13-4-.20 as well as §335-13-5-.02(1). These regulations require that all plans, specifications, operational procedures, letters of final construction certification and other technical data for the construction and operation of a permitted facility be prepared by a professional engineer registered in the State of Alabama. The seal or signature and registration number of the design engineer shall be affixed to the plans, specifications and reports. All legal property descriptions and survey plats shall be by a registered land surveyor (RLS) with the seal or signature and registration number of the land surveyor affixed. Landfill closure must be certified by a registered professional engineer or registered land surveyor. Alabama currently does not require CCP to be disposed on in permitted landfills. Therefore, there are no specific disposal requirements other than what is deemed necessary by the agency on a case-by-case basis. Therefore, the above certifications apply if CCP is disposed of in a permitted landfill.

Kentucky: The landfill permitting law is found in the Kentucky Revised Statutes (KRS) Chapter 224 and the regulations are in 401 Kentucky Administrative Regulations (KAR) Chapters 30, 40, 45, 47, 48, and 49. Generally the landfill application for contained landfills, construction/demolition debris landfills, and residual landfills is in three phases: the Notice of Intent, the Administrative Application, and the Technical Application; however, in Kentucky, CCP is considered a special waste. The special waste landfill application is a single phase with its own public notice requirements. The special waste requirements are found in 401 KAR Chapter 45. This regulation requires that all plans, reports, phasing, closure and post-closure plans be prepared by a professional engineer registered in the State of Kentucky.

Tennessee: Under the TDEC rules, Chapter 1200-1-7-.04 provides specific requirements for Class I, II, III and IV disposal facilities. A professional engineer registered in the State of Tennessee must plan, design, and inspect the construction of any of these facilities; also a registered professional engineer must assist in the start-up and outlining of correct operating procedures for any new or altered facility. In Tennessee, CCP's must be disposed of in either a

US EPA ARCHIVE DOCUMENT

Class I or Class II landfill. A landfill in Tennessee for which only CCP waste is disposed would be subject to the Class II standards.

Federal: 40 CFR 258 establishes the design criteria for new nonhazardous waste disposal facilities and lateral expansions of existing facilities. However, a plan approved by the division director of an approved state will meet the specifications of this rule. Since each of the three states is an approved state, there are no additional federal requirements.

Selected Standard

For all projects governed by regulations that require engineering certification, these activities shall be completed and submitted to the Agency accordingly.

For projects not governed by regulations that require engineering certification, these activities shall be completed and submitted to TVA for project documentation. In this way, the information shall be available for these projects upon request.

3.1.2 Wet Ash Ponds (Impoundments and Dams)

None of the states or the federal regulations specifically address engineering certification of non-hazardous waste impoundments. However, if the structure provides treatment prior to discharge through a permitted outfall, a professional engineer certification may be required. Additionally, if the structure has a dam, as defined by the applicable regulations (1200-5-7), certification would be required in each state. However, according to the TVA Dam Safety Program, state dam safety regulations do not apply. Rather, the TVA Dam Safety Program follows Federal programs in these regards. The following sections for each state are therefore provided for reference only,

Alabama: The ADEM Water Division – Industrial Section provides Construction Guidelines for Industrial Wastewater Impoundments (Rev. 03/00). These guidelines state that a professional engineer should supervise construction and verify that engineering requirements and best engineering construction practices are met during construction. The guidelines further state that after construction is complete, a professional engineer should submit a letter of certification stating that the impoundment was built in conformity with the engineering requirements listed above or alternate requirements determined acceptable by the Department, any stipulations specified by the Department, and the initial submittal. If the impoundment differs from the initial submittal, 'as built' drawings should be submitted.

Kentucky: Certification of KPDES permit applications and reports are defined in 401 KAR 5.060 Section 9. However, this certification is not required to be stamped by a professional engineer. Additionally, there are no specific engineering certification requirements for impoundments other than those required for dam safety. All plans and specifications submitted for consideration of a dam must bear the seal and signature of the responsible engineer as defined in KRS 322.010(2), except officers and employees of the United States government while engaged in engineering for the government. Each sheet of the drawings shall bear the seal and signature of the engineer or engineers responsible for its preparation (Title 401 Chapter 4, Section 030).

US EPA ARCHIVE DOCUMENT

Tennessee: Whenever any new construction or change in existing construction in a wastewater treatment facility is performed, a registered engineer must plan, design, and inspect the construction of such works (TN Rule 1200-4-2-.01). The certified plans must be submitted to TDEC as part of the application for a NPDES permit. Additionally, should the structure require a dam, as defined in TN Rule 1200-5-7-.02(10), a qualified professional engineer must prepare design and specification and supervise the construction. An application for an operating certificate must be submitted to TDEC.

Federal: Since all three of the states in which TVA has facilities are approved states under the NPDES program (40CFR123), no additional certification to the USEPA is required. The federal government does not have governing dam safety certifications.

3.2 BORROW SOURCE REQUIREMENTS

This section will present the applicable regulatory requirements for investigation and testing of soil borrow material sources and provide recommendations on the related procedures. The purpose of borrow material testing is to prequalify the proposed soils intended for use as natural liner or closure materials for landfill or pond constructions. Prequalification is conducted so that the engineer and/or construction contractor have advance knowledge of whether the planned borrow source materials will most likely meet the required design specifications for use as natural liner or closure materials. Potential borrow source materials should be inspected, sampled, and tested prior to their use under the direction of a qualified, geotechnical or landfill engineer. The following subsections discuss sampling, testing and stockpile management in relation to soil borrow source materials.

3.2.1 Sampling

Several methods exist for sampling proposed soil borrow sources. Samples can be obtained by drilling soil borings and recovering samples of the soil from the borings or excavating test pits into the borrow source and collecting bulk samples from the exposed cuts. Excavating test pits in the borrow source provides greater information about the variability of the soils and potential stratigraphic changes than viewing small soil samples recovered from soil borings and is therefore the preferred method.

Qualified field personnel, typically a representative of the design engineer firm or third party QA/QC firm, should be present during sampling to ensure the collection of high quality samples, and to provide an accurate representation of the soil conditions. Samples of approximately 50 pounds should be collected from specified depth intervals at each test pit location and stored in a clean 5-gallon sample bucket. Samples should be collected at specified depth intervals as appropriate to ensure complete coverage of the borrow source and to satisfy the sampling frequency. Small samples of the soil should be placed in a sealed plastic bag to determine the in-situ moisture content of the soil. The sealed plastic bag should be placed inside of the 5-gallon bucket for shipping. Samples should be shipped at the end of the sampling process to a qualified geotechnical laboratory for analysis.

US EPA ARCHIVE DOCUMENT

3.2.2 Testing

Samples taken from the borrow source should be tested to ensure conformance with specifications for parameters defined in the design documents such as percent fines and plasticity index. The soil samples will undergo preliminary testing referred to as index testing. Index or characterization testing will determine if the borrow source samples meet the minimum criteria to be considered as natural liner or closure materials for landfills or ponds. Index testing includes Moisture Content, Atterberg Limits, Particle Size Analysis, and Standard or Modified Proctor Tests. Hydraulic Conductivity testing is performed on borrow source materials that are determined to meet the minimum specifications through index testing. Hydraulic conductivity tests are performed on soils to verify that the potential soil liner or cap material can be compacted to achieve the required low hydraulic conductivity determined by the design engineer.

Minimum Design Standard

Alabama: There are no state regulatory requirements governing the frequency of testing of soil borrow sources for the state of Alabama.

Kentucky: There are no state regulatory requirements governing the frequency of testing of soil borrow sources for Special Waste Landfills. There are, however, minimum requirements for contained landfills (MSW) as specified in 401 KAR 48:80 Section 4, which provides the following testing frequencies for proposed natural liner and cap soils. Table 2.2.2-1 provides American Society for Testing and Materials (ASTM) specifications and recommended frequencies for soil borrow source material investigations in Kentucky.

Table 3.2.2-1

ASTM Specifications and Recommended Frequencies for Soil Borrow Source Material Investigations in Kentucky

Test	Frequency	ASTM Standard
Moisture Content	1 / 2,000 cubic yards	ASTM D 2216
Particle Size Analysis / Soil Classification	1 / 2,000 cubic yards	ASTM D 422
Atterberg Limits	1 / 2,000 cubic yards	ASTM D 4318
Standard or Modified Proctor Test	1 / 20,000 cubic yards	ASTM D 698 ASTM D 1557
Hydraulic Conductivity	1 / 20,000 cubic yards	ASTM D 5084

US EPA ARCHIVE DOCUMENT

Tennessee: There are no state regulatory requirements governing the testing of soil borrow sources for the state of Tennessee.

Selected Design Standard

Table 2.2.2-2 provides ASTM specifications and recommended frequencies for soil borrow source material investigations.

Table 3.2.2-2

ASTM Specifications and Recommended Frequencies for Soil Borrow Source Material Investigations

Test	Frequency	ASTM Standard
Moisture Content	1 / 2,000 cubic yards	ASTM D 2216
Particle Size Analysis / Soil Classification	1 / 2,000 cubic yards	ASTM D 422
Atterberg Limit Determination	1 / 2,000 cubic yards	ASTM D 4318
Standard Proctor Test or Modified Proctor Test	1 / 10,000 cubic yards	ASTM D 698 ASTM D 1557
Hydraulic Conductivity	1 / 10,000 cubic yards	ASTM D 5084

3.2.3 Stockpile Management

Soil borrow materials that have been pre-qualified via the testing and analysis program (and are thus determined to meet the criteria for soil liner or cap material) shall be excavated and stockpiled on-site. Stockpiles should be constructed in accordance with these recommended guidelines:

- Locate stockpiles in a strategic location to minimize haul distance and double handling;
- Do not stockpile soils in a location which will interfere with the natural drainage tendencies of the surrounding areas;
- Stockpile soils a minimum of 50-feet away from concentrated flows of stormwater, drainage courses, and inlets;
- Slope stockpile to promote drainage;
- Protect soil stockpiles from stormwater run-on with temporary perimeter barriers such as berms, dikes, silt fencing, or hay bales; and

US EPA ARCHIVE DOCUMENT

- Provide temporary seeding in accordance with erosion and sediment control regulations as discussed in the Soil Erosion Section (Section 1.6.1) of this Volume.

3.3 AGENCY NOTIFICATIONS AND RESTRICTIONS

Definition

Agency notifications by way of applications, correspondence, etc are required for permitted facilities in accordance with several state and federal regulations. This section summarizes general notification requirements for the states and government and then provides a reference to the notification requirements. Additionally, in order to do significant work affecting federally controlled properties (i.e. work in which federal funds are used, work requiring federally issued permits such as USA COE Section 10 and Section 404 permits or TVA Section 26a permits) a number of federal agencies must be notified. These agencies, notified as part of the NEPA process, are listed in the sections which follow.

3.3.1 Dry Ash Landfill

Alabama**:** The notification process for a permitted landfill in Alabama requires notification to ADEM through the application process (ADEM Rule 333-13-5). However, prior to ADEM's beginning review of a new permit application the local government with jurisdiction over the proposed landfill site must approve the location, after involving the public in the siting process (Code of Alabama 1975, §22-27-48). A pre-application conference with the technical staff of the Land Division and the applicant's representatives to discuss specific requirements for the application is recommended. Specific processing steps and time periods are provided so that the applicant can include these tasks in the overall planning of the project development. In some instances, all necessary media-specific permit applications can be processed on parallel tracks. Applications for construction or modification of an existing landfill should be made at least 180 days in advance of start of construction.

Since construction of a landfill by TVA would involve federally owned property or federal funding, an environmental assessment (EA) of the project would be reviewed through the NEPA process. Numerous state and federal agencies are included in the scoping and coordination of the NEPA process. In Alabama these include: Alabama Historical Commission, ADEM Division of Water Quality, Division of Solid Waste, Division of Air, Alabama Department of Conservation and Natural Resources, Division of Wildlife and Freshwater Fisheries, Alabama National Floodplain Insurance Program, as well as numerous other state, local, and federal agencies.

Kentucky: Prior to submitting an application for a special waste landfill permit, as required by 401 KAR 45:030, a meeting should be scheduled with the Division of Waste Management concerning the location of the site and to review the potential need of additional permits that might be necessary for the project, such as KPDES discharge permits, floodplain permit, transportation permit, etc. The division has a list of agencies and points of contacts for these other permits. Division staff is also available to discuss the permitting and public notification

US EPA ARCHIVE DOCUMENT

process. Once an application is deemed complete, the state's timeline for issuing a preliminary decision to deny the permit is 180 calendar days. If the decision is made to issue the permit, a draft permit is prepared and 30-day public notice is published. Depending on the comments received, the construction permit may be issued soon after the comment periods, or if more concerns are identified, it may take several months.

As with Alabama, construction of a landfill in Kentucky would involve federally owned property or federal funding, and an EA of the project would be reviewed through the NEPA process. Coordination would be required with the State Historic Preservation Officer, Kentucky Department of Environmental Protection (Air, Water and Solid Waste), State Fish and Game agencies, Forest Service, Economic and Community Development, Agriculture Transportation and others. Federal agency coordination is described below.

Tennessee**:** The notification process for a landfill in Tennessee requires notification to the Division of Solid Waste Management through the local field office. In accordance with TDEC Rule 1200-1-7-.02(2), a Part I and Part II application, along with the application fee, must be submitted before the application is deemed complete. The Part I application (Form CN-1036) which includes the name, address and phone numbers of the owner(s); proposed activities to be conducted at the facility; a statement regarding whether the facility is subject to local approval (TCA § 68-211-701) and county approval if necessary, a topographic map and disclosure statement. The Part II includes: a hydrogeologic assessment of the potential site; facility design plans and operations manual; financial assurance demonstrating the financial responsibility for closure and post-closure care; and other specific requirements for Class I, II, III, and IV disposal facilities.

After the Part I application is received and reviewed for completeness, a preliminary public notice is issued. The state archeologist reviews the site for the existence of burial grounds. The Part II items are then submitted to the Division and are reviewed by a committee. When all documents and approvals have been met, a second public notice of intent to issue a permit is issued. Public response to this second notice may generate another public notice to hold a public hearing. If requested, the Division may give notice of a public hearing concurrently with the second public notice of intent to issue a permit. After review of the public comments, a final public notice with the permit decision is issued. The entire permit process may take from 12 months or longer, depending on the type of facility, public interest, public hearings, revisions, appeals, and site preparation.

Since construction of the landfill would involve federally owned property or federal funding, an EA of the project would be reviewed through the NEPA process. Within the state of Tennessee there are a total of six departments that must be notified prior to construction if any of the following issue categories are pertinent to the NEPA review. These departments and their divisions that should be included in the review process if there are issues pertinent to that division include:

- TN Department of Environment and Conservation (TDEC) Divisions: Threatened and Endangered Species and Wild/Scenic Rivers; State Parks; Public Recreation; Water; Air;

US EPA ARCHIVE DOCUMENT

Surface/subsurface Drinking Water Sources; Septic Systems; Superfund; Underground Storage Tanks; Solid/Hazardous Waste; and Radiological Health.

- TDEC Tennessee Historical Commission
- Tennessee Department of Agriculture
- Tennessee Department of Transportation
- Tennessee Wildlife Resources Agency
- Tennessee Department of Economic and Community Development.

Federal**:** Construction of a landfill in any of the three states would involve federally owned property or federal funding; therefore an EA of the project would be reviewed through the NEPA process. Federal agencies that are included in the NEPA process include:

- US Fish and Wildlife Service
- US Forest Service
- Natural Resources Conservation Service
- Native American Tribes (United Southeastern Tribes – USET)
- US Army Corps of Engineers
- US Environmental Protection Agency
- Tennessee Valley Authority

3.3.2 Wet Ash Ponds (Impoundments and Dams)

According to the TVA Dam Safety Program, state dam safety regulations do not apply. Rather, the TVA Dam Safety Program follows Federal programs in these regards. The following sections for each state are therefore provided for reference only.

Alabama: ADEM regulations do not specifically address wet ash impoundments or dams. Prior to any construction of a new wet ash impoundment or modification to an existing impoundment whereby water may be discharged, an application to ADEM's Water Division must be made in accordance with rules provided in Chapters 335-6-6. The application must include technical components of the control structures, BMPs, etc.

Kentucky: The Kentucky DEP regulations do not specifically address wet ash impoundments. Prior to any construction of a new wet ash impoundment or modification to an existing impoundment whereby water may be discharged, an application to KDEP in accordance with 401 KAR 5:060 must be submitted and a permit issued. The technical and administrative

US EPA ARCHIVE DOCUMENT

requirements are similar to other states, such as Alabama (above) that have permit programs approved by the EPA, with the exception that the Dam Safety Section and Floodplain Compliance Section requirements for dam construction would be part of the submittal package. Regulations relating to dam safety are in Kentucky Administrative Regulations Chapter 4, Title 401, Water Resources.

Tennessee: TDEC regulations do not specifically address wet ash impoundments. However, prior to any construction of a new wet ash impoundment or modification to an existing impoundment whereby water may be discharged, an application to TDEC Division of Water Pollution Control must be made in accordance with TDEC Rule 1200-4-5-.05. The technical and administrative requirements are similar to other states, such as Alabama (above) that have permit programs approved by EPA.

Federal: Since all three of the states in which TVA has facilities are approved states under the NPDES program (40 CFR 122), no additional notification to USEPA are required. In the case of new land disturbances or federal properties, the NEPA notification for wet ash ponds is the same as for dry ash landfills (Section 3.3.1).

3.4 GENERAL CONSTRUCTION PERMITS (LOCAL, STATE, AND FEDERAL)

Definition

If a facility plans to clear, grade, or excavate an area of one or more acres, a stormwater construction permit is required by all states in the TVA region. This section outlines the general requirement necessary for obtaining such permits

3.4.1 Dry Ash Landfills

Alabama: The rules for storm water construction permits require an operator/owner to register construction activities and associated areas one acre or greater in size. Construction activities less than one acre in size that are part of or associated with a larger plan of development or sale that might eventually exceed one acre, must register. In addition, construction activities less than one acre in size that are determined by ADEM to have significant potential to cause or contribute to water quality impairment, may be required to register.

These rules require that a Construction Best Management Practices Plan (CBMPP), prepared by a qualified credentialed professional (QCP), and designed to minimize pollutant discharges in stormwater runoff to the maximum extent practicable during land disturbance activities, be fully implemented and effectively maintained. A CBMPP is required to be submitted with the request for registration for proposed discharges to a Tier 1 waterbody(s), proposed discharges to an Outstanding National Resource Water (ONRW) designated waterbody, and for projects involving waterbody relocation or significant alteration. NPDES registration coverage must be retained until all disturbed areas have been reclaimed and/or effective storm water quality remediation has been achieved.

US EPA ARCHIVE DOCUMENT

Kentucky: The storm water construction permit requires any agency, company or individual planning construction activity in Kentucky disturbing one acre or more to:

- Develop and implement a "Storm Water Best Management Practices Plan." Among the requirements of this plan is that there be provisions for inspections every seven days and after each rain of one-half inch or more.

- Submit a signed Notice of Intent (NOI) form to Kentucky Division of Water at least 7 day before construction begins (if filed electronically and 30 days if paper filled.

- Submit a copy of the NOI to the municipal operator of any municipal separate storm sewer system (MS4) the site discharges into. Note that a NOI is now not required on activities at a facility that is covered under an NPDES permit.

- Submit a signed Notice of Termination (NOT) form to the Kentucky Division of Water after construction activity has ceased and the site has been finally stabilized.

Tennessee: Applicants must submit the following information for a Storm Water Construction Permit:

- A completed and signed NOI for Construction Activity - Storm Water Discharges. The NOI must include a map on 8 ½ inch by 11 inch paper with boundaries 1-2 miles outside the site property with the site and construction area outlined and the receiving water or receiving storm sewer highlighted and identified. It is preferable for this map to be the appropriate portion of a USGS 7.5-minute quadrangle map.

- A site-specific SWPPP must be developed and submitted with the NOI. The SWPPP must be developed, implemented, and updated according to Part 3 of the Construction General Permit (CGP).

Federal: Construction permits follow state regulations in all states of the TVA Region. The NEPA process must be followed for any construction that involves significant work affecting federally controlled properties. Certain project activities can trigger the need for an EA under NEPA. Pursuant to the Council on Environmental Quality (CEQ) regulations (40 CFR 1500-1508) for implementing the procedural provisions of the National Environmental Policy Act (NEPA) (42 U.S.C. 4321 et seq.), an EA of potential environmental effects associated with federally funded projects is required. Other federal permits include Section 404 (for activities that impact waters of the United States) and/or Section 10 (for activities that impact navigation) with Joint Section 26a TVA permits. Where there are results of an EA warrant, an environmental impact statement (EIS) may be required. In industrial development type projects, an EA and possibly an EIS may be triggered by:

- Significant work affecting federally controlled properties. In Tennessee this is most often work on properties controlled by the Corps of Engineers or TVA;

- Work requiring federally issued permits such as Corps Section 10 and 404 permits or TVA Section 26a permits;

US EPA ARCHIVE DOCUMENT

- Project work for which federal funds are used.

See Section 3.3 for agencies which require notification. The lead agency is identified by the category of impact. If a 404 permit is required, the USACE is the lead reviewing agency. Following notification to the agencies, an EA will be prepared by, or on behalf of the applicant.

3.4.2 Wet Ash Ponds (Impoundments and Dams)

The storm water general construction permitting requirements for dry ash landfills would be applicable to wet ash impoundments as well. The rule is based on the one acre disturbance and requirement of a storm water construction permit. In addition, to the storm water construction permit, and subsequent NEPA process, impoundments are subject to Dam Safety Compliance. According to the TVA Dam Safety Program, state dam safety regulations do not apply. Rather, the TVA Dam Safety Program follows Federal programs in these regards. The following sections for each state are therefore provided for reference only.

Alabama**:** see Section 3.3.2

Kentucky: see Section 3.3.2

Tennessee: see Section 3.3.2

Federal: see Section 3.3.2

US EPA ARCHIVE DOCUMENT

This section addresses the various elements of managing construction projects within TVA. There are no references to specific regulations as these do not apply, but rather, the information presented herein is consistent with TVA's procedural and organizational approaches to management of construction project with either TVA or a Consultant in the roles defined.

4.1 OVERVIEW OF BID, EVALUATION, AND CONTRACT AWARD PROCESS

Bidding is the procurement of goods or services through a process of open and free competition. There are generally two types of bid processes, Request for Proposal and Competitive Bidding using Sealed Bids. Bids are evaluated and awarded based on a system of assessment criteria.

Prior to the bid process, the first step to successfully bidding a project for construction is to have a clearly defined project. An engineering firm or TVA depending on size/type of project should develop a complete set of design documents including drawings and specifications. This construction package should be reviewed and approved by the Owner prior to being issued for bid. It is from this construction package that a RFP will be developed.

The bid process begins through the development of the scope of work (SOW) statement (description of services) or specifications (description of goods), proposal evaluation criteria, and a recommended sources list. These are submitted to the Purchasing Department, which takes this SOW or specifications and develops a complete RFP including standard contract clauses, special clauses, instructions to prospective bidders, and any requisite technical exhibits or attachments. The RFP states a specific date and time deadline for proposal receipt and often has mandatory pre-bid meetings for bidders to attend. This meeting offers the opportunity to ask questions and gives the Owner a chance to determine whether any changes (addenda) need to be issued to the RFP. This is also an excellent time to conduct any requisite site visits to familiarize bidders with the project site(s).

After proposals are received, they are evaluated against the criteria stipulated in the RFP. After evaluating the proposals, the Owner chooses the awarded firm. Once approved, a purchase order and/or contract are processed. The following sections provide a general discussion of the bidding, evaluation, and contract award process.

4.1.1 Developing the Scope of Work

A well-written scope of work is critical to the success of a contract. A good scope of work is clear, complete, and logical enough to be understood by the bidder and Owner. Because it describes the details of performance, it is the yardstick against which the bidder's performance is measured. The structure of the document is meant to organize a bidder's response. It may be helpful to organize the RFP document in numbered sections, and require the bidders to use this same numbering/sectioning format in their responses. This ensures clarity and consistency in the RFP and in the bidders' responses, and will make the evaluation and selection process easier.

US EPA ARCHIVE DOCUMENT

- Suggested Content - Introduction and general information, task description, constraints on the contractor, contractor personnel requirements, Owner responsibilities, special conditions, and evaluation criteria.

4.1.2 Developing a List of Bidders

A list of bidders should be developed in order to solicit bids from qualified contractors. The number of bidders can vary depending upon the size of the scope, corporate requirements, and availability of qualified bidders. The bidders listed will receive a construction document package including the RFP, construction documents, and drawings. The list of bidders should be limited to contractors that meet a series of defined criteria. Contractors should be "pre-qualified" prior to being included on the bidders list. The development of bidder's qualifications will limit the list of bidders to contractors that possess the ability, manpower, equipment, performance reputation, and experience to perform the work defined in the RFP. A list of qualifications should be developed and demonstrated by potential bidders.

- Suggested list of bidder qualifications (experience, minority requirements, safety record (OSHA), and union requirements)

4.1.3 Bidding

A pre-bid meeting is typically held to issue the RFP and a set of Contract Documents to the qualified bidders. The pre-bid meeting provides the Owner an opportunity to describe the project scope, introduce the engineer and key project personnel, define roles and responsibilities, and answer questions. Bidders are typically given the opportunity to request additional information or clarifications in writing prior to the bid submittal date. In addition, the Owner will notify the bidders of the specific date and time for bids to be submitted as well as the expected award date. Bidders may also be given the opportunity to visit the project site(s).

4.1.4 Developing Proposal Evaluation Criteria

A prerequisite for award is that the bidder must be responsible and must submit a responsive offer. To be responsible means the bidder has the requisite business integrity, as well as financial and organizational capacities, to ensure good-faith performance. To be responsive, an offer must conform in all material respects to the RFP or at least offer valid alternatives such that all aspects of the RFP are addressed. Nonresponsive offers, those bids that do not address all aspects of the RFP nor detail valid alternatives, will not be considered. Beyond these two basic criteria, the proposal evaluation criteria, which are published in the RFP aid in the selection of the bidder.

A comparative analyses of different bidder's proposals must be defensible and objective. This makes the drafting of reasonable and definitive evaluation criteria very important to the RFP and source selection process.

US EPA ARCHIVE DOCUMENT

Some evaluation criteria to consider for inclusion in the RFP are as follows: (1) performance record of the contractor, (2) safety record, (3) relevant experience in providing comparable services on projects of similar size and scope, (4) overall quality of proposal, and (5) pricing.

The RFP should contain a cost proposal format that allows the bidders to explicitly identify their charges for the deliverables identified in the project. Deliverables must be well defined so that all bidders can respond to the same deliverables thus allowing the Owner to make comparative analyses of the bidder's costs.

4.1.5 Evaluating the Proposal

After the Purchasing Department has reviewed each bidder's proposal to determine that they are complete, the proposals are then forwarded to the department and/or committee members for evaluation. During the period of evaluation and prior to award, possession of proposals and accompanying information is limited to personnel responsible for participating in the evaluation. Any communications with bidders must be approved in advance through the purchasing agent handling the bid process.

Recommendation for award must be in written form and must address how each bidder has met or failed to meet the evaluation criteria stated in the RFP. All areas of non-conformity with any terms, conditions, or listed specifications shall be clearly stated in the evaluation.

To assist those individuals responsible for evaluating proposals, the Purchasing Department creates an evaluation matrix, based upon evaluation factors listed in the RFP. This matrix is a tool to assist those evaluating the proposals.

4.1.6 Awarding the Contract

Based on the evaluation process, a clear choice should present itself. The contract is awarded to the bidder that best meets the evaluation criteria. The Owner will issue a Letter of Intent to the apparent winning bidder and contract negotiations will follow. The Owner and winning bidder will enter into a contract and a Notice to Proceed will be issued.

4.2 CONSTRUCTION MANAGEMENT

Construction Managers are often contracted by the owner to oversee a project’s completion. The American Society of Civil Engineers (ASCE) Committee on Construction Management accepts the concept of construction management being performed by the firm responsible for the design. However, construction management is typically conducted by a third party firm not involved in the design or construction of the project.

4.2.1 Construction Manager – TVA Employee

The role of the construction manager is solely dependent on the needs and preferences of the owner. Comprehensive construction managers provide a wide range of services and can be

US EPA ARCHIVE DOCUMENT

involved in both the design and construction phases of a project. In general, a construction manager provides leadership to the construction team, and coordinates between the owner, engineer, and contractor to plan and oversee the completion of a project. Responsibilities of the construction manager may include:

- Performing constructability analysis and reviews of design submittals and contract documents during the design phase of the project.
- Bid opening and evaluation, and assistance in contractor selection.
- Managing the project budget for the owner.
- Monitoring construction progress and maintaining the project schedule.
- Advising on and coordinating the procurement of materials and equipment.
- Coordinating between the owner, engineer, and contractor to resolve design and construction issues.
- Providing general construction oversight to monitor conformity with construction requirements.
- Contract administration.
- Senior site representative leading progress meetings.
- Acting as a day to day liaison between the Owner, Construction Quality Assurance (CQA) Consultant, and the Contractor
- Interpreting construction plans, specifications, and contracts.
- Overseeing day to day activities and generating progress reports.
- Maintaining the logs, data, files, and correspondence for the field records.

4.2.2 Resident Engineer – TVA Employee / Third Party

The CQA Resident Engineer acts as the engineering site representative for the inspection of contractor construction activities. The CQA Resident Engineer is generally responsible for:

- Planning, organizing, and executing resident engineering activities.
- Review submittals for compliance with engineering design.
- Coordinating between the Owner, Engineer, and Contractor to resolve design issues.
- Coordinating responses to RFI's and other technical issues with design engineers.
- Providing engineering oversight and assuring compliance with the engineered design.

US EPA ARCHIVE DOCUMENT

4.3 CQA CONSULTANT – THIRD PARTY

The CQA Consultant is responsible for making observations and performing field tests to ensure that a facility is constructed in accordance with the applicable plans, and specifications and QA/QC Plan. The following section provides a description of the typical CQA Consultant team, including each member's roles and responsibilities.

4.3.1 Certifying Engineer

The CQA Certifying Engineer is responsible for certifying to the Owner and the permitting agency that the facility has been constructed in accordance with the plans, drawings, and the approved CQA Plan. The certifying engineer serves as the Professional Engineer for the project and properly certifies the as-built construction certification report.

4.3.2 Project Engineer

The CQA Project Engineer is responsible for providing engineering and technical support to the field CQA team throughout the construction process. The Project Engineer works closely with the Construction Manager to assist with calculations and complete take-offs in support of as-built quantities for payment. The Project Engineer also responds to contractor RFIs, reviews and maintains QA/QC data and coordinates all supplementary laboratory testing of geosynthetics and soils.

4.3.3 Field Technicians

Field Technicians provide general inspection of materials, equipment, and workmanship under the direction of the CQA Resident Engineer. Specific duties include conducting in-situ nuclear density testing and in-place geosynthetic testing, field testing and verifying compliance with standards. Field technicians are also responsible for maintaining testing data logs and daily reports.

4.3.4 Scheduler / Administrative Personnel

The scheduler / administrator is responsible for file management, invoicing and billing, and maintaining the schedule. The scheduler assembles documentation from the project team and maintains the files. The scheduler is responsible for reviewing and incorporating monthly invoices into tracking forms, and issuing payments for services and materials.

4.3.5 Soils / Geosynthetics Laboratory

Soils and Geosynthetic Laboratories are responsible for performing specific tests on materials submitted for testing. The laboratories are responsible for ensuring that tests are performed in accordance with applicable standards, following internal QC procedures, maintaining chain-of-custody records, and accurately reporting data. The testing lab should be accredited. For

US EPA ARCHIVE DOCUMENT

geosynthetic material testing, such an accreditation is available through the Geosynthetic Accreditation Institute Laboratory Accreditation Program (GAI-LAP).

4.4 CONSTRUCTION DOCUMENTATION PREPARATION

Definition

A Construction Certification Report (Certification Report) is a report that documents construction activities and provides record of a project's completion in accordance with applicable design drawings, specifications, and CQA Plan.

Minimum Standards

The minimum regulatory requirements for providing construction certification reports are outlined below:

Alabama: There are no specific requirements related to the preparation or submittal of construction certification documents in Alabama.

Kentucky: As stated in 401 KAR 45:030 Section 9(11), prior to receiving a construction/operation permit from the Kentucky DEP, a certification must be submitted by a registered engineer that the liner system, if required, and other features have been constructed in accordance with the approved plans a specifications.

Tennessee: Rule 1200-1-7-.04(1)(c) requires that a registered engineer must inspect the construction of any Class I, II, III, or IV disposal facility, but does not appear to require the submittal of construction inspection documentation. However, the requirement to submit construction QA/QC documentation may be a specific condition of the TDEC permit.

Depending on the nature and scope of the project, the Construction Certification Report may need to be submitted to the governing regulatory agency for review. The governing regulatory agency should be notified during the design phase of all construction projects to determine what, if any, construction documentation is required to be submitted for approval.

Selected Standard

It is recommended that a Construction Certification Report be prepared by the CQA Consultant for all major construction activities. At a minimum, the Certification Report shall include the following elements:

- Introduction: An introduction and description of the construction project, including tasks to be completed, a timeline of construction activities, and any applicable permits authorizing the construction activity.

- Personnel: A listing of all personnel associated with the construction, including TVA Construction Managers, Resident Engineers, CQA Consultant, and all Contractors associated with the construction.

US EPA ARCHIVE DOCUMENT

- Alterations to the Design: Should the design be modified during construction activities, a description of each specific design alteration shall be discussed.

- Impacts to Construction: A general discussion of impacts to the construction project, such as any weather delays or mechanical failure of construction equipment shall be included.

- Construction Narrative: The construction narrative provides the main body of the Certification Report. A narrative description of each construction activity shall be included to describe the construction activities performed such that an outside party could review and adequately understand the construction. The narrative shall include, at a minimum, the duration and dates of completion for each construction activity and should reference all CQA testing and documentation procedures performed as part of each construction activity.

- Daily Reports: The CQA Consultant shall complete daily reports of construction activities to document the construction and aid in the completion of the Certification Report.

- Photo Logs: In addition to daily reports, a log of photographs shall be maintained by the CQA Consultant and included in the Certification Report.

- CQA Test Results: The results of all CQA testing required in the CQA Plan shall be included in the Certification Report.

- Other CQA Documentation: Each construction activity may require the completion of specific CQA documentation. Examples of such documentation include subgrade acceptance forms and geosynthetic seam and repair logs. Required CQA documentation shall be described in the CQA Plan and completed documentation shall be included in the Certification Report to verify that the construction has been completed in accordance with the CQA Plan.

- Record (As-Built) Drawings: A package of Record Drawings shall be completed for each construction project. Typically, the record drawings include as-built surveys for each construction milestone. The CQA Plan shall provide specific instruction related to the record surveys required, and the specific elements to be included in each record drawing.

  Record surveys are to be completed by the Contractor and provided to TVA and the CQA Consultant to verify that the construction milestone has been completed in accordance with the applicable Design Drawings and Specifications. For example, in the construction of a new facility, a record survey of the excavation or subgrade may be required prior to the construction of the facility liner system. Once the record survey has been completed and submitted to the CQA Consultant, the Certifying Engineer will review the survey and either accept the record survey, or reject the record survey and provide guidance on the remedies or alterations required. The construction milestone

US EPA ARCHIVE DOCUMENT

should be considered complete only after the CQA Consultant provides approval of the record survey. Record Survey data will be submitted to TVA and documented in the TVA surveying system. TVA reserves the right to perform the Record Survey in lieu of the contractor.

In addition to record surveys, the Resident Engineer shall maintain a set of the Design Drawings and make note of any changes to the design. These as-built notations shall be combined with the record surveys by the CQA Consultant into a final package of Record Drawings.

- Notarized Statement of Accuracy: The Certification Report shall include a statement that the documentation provided in the Certification Report is true and accurate. The statement shall be notarized and signed and by the Certifying Engineer and TVA management personnel.

US EPA ARCHIVE DOCUMENT

5.1 CLOSURE REQUIREMENTS

This section identifies regulatory requirements for closure of dry ash landfills and wet ash impoundments in each of the three states in which TVA has facilities.

Definitions

Alabama: In Alabama, closure is the process by which a landfill unit permanently ceases to accept waste, to include those actions taken by the permittee or owner of the facility to prepare the site for post-closure monitoring and maintenance or to make it suitable for other uses.

Kentucky: In Kentucky, closure means the time at which a waste treatment, storage, or disposal facility permanently ceases to accept wastes, and includes those actions taken by the owner or operator of the facility to prepare the site for post-closure monitoring and maintenance or to make it suitable for other uses (KRS 224.01-010(4).

Tennessee: In Tennessee, closure means taking the actions at the termination of a disposal operation that are necessary to finally close the disposal facility or disposal facility parcel.

5.1.1 Dry Ash Landfills

Alabama: The closure requirements for a permitted landfill facility are contained in the ADEM administrative code §335-13-4-.20. The closure plan must be submitted as part of the permit application to the Department. This plan must address specific elements of the cover system, which for municipal and industrial waste landfills includes an infiltration layer equal to the permeability of the bottom liner system or 1 X 10^{-5} cm/sec, whichever is less, and an erosion layer on minimum of 6 inches of earthen material capable of sustaining native plant growth. For permitted special waste landfills, the cover system must be constructed in a manner deemed necessary by the Department on a case by case basis. The length of time necessary for post-closure care is established by the Department.

Kentucky: The closure requirements for a special waste landfill are contained in 401 KAR 45:110 Section 5. As part of the initial permit application, the applicant must submit a closure plan for approval that specifies the design of the final cover including functionality, erodibility, stability, etc. The closure plan design shall comply with 401 KAR 30:031 performance standards including, but not limited to, no discharge of pollutants into the waters of the state no release to underground drinking water source beyond the point of compliance in excess of maximum contaminant levels (MCL's).

Tennessee: The closure requirements for a permitted landfill facility are contained in Tennessee Division of Solid Waste Management Rule 1200-1-7-.04. The closure plan must be submitted as part of the permit application to the Department. This plan must address specific elements of the cover system, which for Class II landfills includes a 24 inch compacted soil layer with a permeability equal to that of the bottom liner system or 1 X 10^{-7} cm/sec, whichever is less, and a vegetative layer of a minimum thickness of 12 inches. By regulation the agency can approve an alternate cover if it can be demonstrated to provide equivalent or superior performance. The

US EPA ARCHIVE DOCUMENT

length of time necessary for post-closure care is 30 years unless alternate period is established by the Department.

Federal: Tennessee, Alabama and Kentucky have developed and implemented a Subtitle D permit program and therefore, provided USEPA does not determine in the future that their programs are inadequate; there will be no additional federal requirements.

5.1.2 Wet Ash Ponds (Impoundments)

According to the TVA Dam Safety Program, state dam safety regulations do not apply. Rather, the TVA Dam Safety Program follows Federal programs in these regards. The following sections for each state are therefore provided for reference only.

Alabama: Although there are no regulations that specifically address the closure of wastewater impoundments, ADEM has promulgated guidance. This document, titled The ADEM Water Division – Industrial Section provides Closure Guidelines for Industrial Wastewater Impoundments (Rev. 03/00), requires the submittal of a site specific closure plan. This plan must be certified by a professional engineer if it involves the practice of engineering and certified by a professional geologist if it involves the public practice of geology. The only regulatory citation in the guidelines has to do with the cap design meeting the requirements of ADEM Solid Waste Rule 335-13-4-.20(2). Alabama currently has no dam safety legislation or formal dam safety program. Currently the Alabama Dam Security and Safety Initiative has been created to establish dam safety laws and regulations in Alabama.

Kentucky: Kentucky does not have any specific regulations relative to closure of nonhazardous wastewater impoundments nor does it provide formal closure and post-closure guidelines for impoundments. The Kentucky Water Quality Control Act prohibits the discharge of any substance into the waters of the state that could cause damages or pollution to such waters. Therefore, each impoundment must be properly closed to eliminate discharge of pollutant to both the surface water and ground water. Although not specifically addressed in the Groundwater Protection Plan required by 401 KAR 5:037, it is recommended that this plan address proper closure of all wastewater impoundments to ensure protection of all current and future uses of groundwater and to prevent groundwater pollution. Both the Kentucky Division of Waste Management and Division of Water should be notified of intent to close an impoundment in order to get approval of proposed procedure.

Tennessee: Tennessee does not have any specific regulations relative to closure of nonhazardous wastewater impoundments nor does it provide formal closure and post-closure guidelines for impoundments. The Tennessee Water Quality Control Act prohibits the discharge of any substance into the waters of the state that could cause damages or pollution to such waters. Therefore, each impoundment must be properly closed to eliminate discharge of pollutant to both the surface water and ground water. Both the Tennessee Division of Solid Waste and Division of Water Pollution Control should be notified of intent to close an impoundment to obtain approval of proposed procedure.

Federal**:** There are no specific federal requirements for wastewater impoundment enclosures.

US EPA ARCHIVE DOCUMENT

6.1 POST-CLOSURE REQUIREMENTS

This section identifies state and federal regulatory requirements for post-closure for each type of facility.

Definition

Post-closure requirements are requirements that must be met following the closure of a landfill or an impoundment.

6.1.1 Dry Ash Landfills

Minimum Standards

Alabama: Code Section 335-13-4-.20(3) – For municipal and industrial landfills, the operator must maintain post-closure care for a minimum 30 years; however, length may be decreased by the Department if owner/operator can demonstrate that a shorter post-closure period can be justified [335-13-4-.20(3) (b)]. For CCP, which is a nonregulated waste, this period, would be set by the department at the time of a permit application, depending on the type of permit applied for.

Kentucky: Per 401 KAR 45:110 Section 5(3), the landfill operator must maintain post-closure for a minimum 5 years.

Tennessee**:** Rule 1200-1-7-.04(8) post-closure care requirements 8(d) & (e) is for 30 years for Class I and II landfills, 2 years for Class III and IV landfills. CCP landfills are Class II landfills and therefore require 30 years of post-closure care.

Federal**:** Tennessee, Alabama and Kentucky have developed and implemented a Subtitle D permit program and therefore, provided EPA does not determine in the future that their programs are inadequate; there will be no additional federal requirements.

6.1.2 Wet Ash Impoundments

According to the TVA Dam Safety Program, state dam safety regulations do not apply. Rather, the TVA Dam Safety Program follows Federal programs in these regards. The following sections for each state are therefore provided for reference only.

Minimum Standards

Alabama: The ADEM Water Division – Industrial Section provides Closure Guidelines for Industrial Wastewater Impoundments (Rev. 03/00). There are no post-closure guidelines.

Kentucky: No Post Closure Care rules are provided in Kentucky. Closure and Post Closure Care Rules may be adapted from 401 KAR 45:110 Section 5(3) – See 2.4.1.2.

US EPA ARCHIVE DOCUMENT

Tennessee: No Post Closure Care rules are provided in Tennessee. Closure and Post Closure Care Rules may be adapted from Division of Solid Waste Rule for Landfills or NRCS Rules for waste impoundments, See 2.4.1.3

Federal**:** Tennessee, Alabama and Kentucky have developed and implemented a Subtitle D permit program and therefore, there will be no additional federal requirements.

US EPA ARCHIVE DOCUMENT

Industrial and commercial facilities that store waste materials should prepare a response or contingency plan to respond to emergencies involving the accidental release of these substances into the environment. Response plans should identify potential hazards, develop systems for preventing accidents, provide appropriate mechanisms for minimizing risk, loss and damage resulting from such incidents and provide an incident management structure to guide response activities.

In this section, the Document details the steps necessary to identify potential emergency situations and verify that a system of emergency procedures and directions, and general standard responses for possible emergency situations, are in place prior to facility operation, and that those procedures are sufficient to meet applicable regulatory standards.

7.1 EMERGENCY OPERATIONS AND CONTIGENCIES

Definitions

In the context of this industry involving both CCP dry disposal facilities (landfills) and wet impoundments (ponds, wet stacks), an emergency is any unexpected or accidental, serious occurrence or situation involving the release or imminent release of CCPs that could result in adverse effects on human health or the environment that requires immediate action. Items to be considered will include natural disasters (hurricanes, tornadoes, floods, earthquakes, etc.); operational failures (power outages, equipment breakdown, etc.); and, accidents (fire, vehicular or machinery accidents, etc.). It may also be necessary to consider terrorist attacks.

For the purpose of this Document, risk assessment means identifying those components of CCP landfills and impoundments that could lead to operational conditions or emergencies that may need immediate attention. Operational risk can be managed by development of administrative and operational programs designed to reduce the risk of these emergencies. Such programs include design standards discussed previously in this document. However, in this section, the applicable information is intended for the operational life of the facilities and can include standard operating procedures, equipment and facility preventive maintenance, and employee training, all in the context of emergency planning.

A contingency is an uncertainty, such as when things are left to chance, which can lead to accidents and otherwise preventable emergency situations. In order to reduce the risks associated with CCP landfills and impoundments and to avoid emergencies to the extent possible, facilities are designed, constructed and operated in accordance with strict guidelines that are imposed as a result of regulation, an engineer's recommendations, or the owner's preferences, typically whichever is most protective. Because emergencies do occur and require immediate attention when they happen, an Emergency Operations and Contingency Planning document, also know as an Emergency Action Plan (EAP) is recommended to adequately plan for emergency situations and to train employees to address them before they occur.

Minimum Standards

Alabama: In Alabama, CCPs are specifically exempted from the definition of "solid waste" and "industrial waste." Therefore, no specific requirements for CCPs are presented in this section.

US EPA ARCHIVE DOCUMENT

Alabama currently does not have any dam safety regulations. However, regulations are in the process of being developed.

Kentucky: In Kentucky, CCPs are managed as special wastes with limited requirements with respect to specific design, construction and/or operational requirements. Minimum standards for solid waste facilities are given in 401 KAR Chapter 48, which defines contingency plan. "Contingency plan means a document setting out an organized, planned and coordinated course of action to be followed in the event of a fire, explosion, or release of waste or waste constituents into the environment which has the potential for endangering human health and the environment. Financial planning to identify resources for initiation of such action is a part of contingency plan development. Kentucky 401 KAR 48.070, design requirements for contained landfills includes a Safety and Communication Plan – The landfill safety and communication plan shall contain:

- Safe operating and maintenance procedures for heavy equipment;
- Procedures to protect employees in a manner complying with the Kentucky Labor Cabinet OSHA requirements;
- A description of equipment to achieve emergency communication. At a minimum, the applicant shall specify an on-site telephone or a two-way radio connection to an off-site telephone. The radio base station shall be monitored during landfill operations.
- A fire fighting contingency plan containing a topographic map denoting the location of the landfill, a site map and an emergency contact. The operator shall mail a copy of the safety and communications plan to the local fire chief. The plan shall include the location of fire fighting water sources, roads, and major site features. Note: the potential for fire at a contained landfill, which can include municipal solid waste (with large percentages of flammable wastes such as paper and plastic) or flammable industrial wastes is greater than for a special waste CCP facility and, as such, the requirement for a fire fighting contingency plan is made.

Kentucky regulates dams and dam safety in the state via 401 KAR 4:030, and provides design requirements in Division of Water Engineering Memorandum No. 5 (2-1-75). There are no specifics provided regarding contingency planning.

Tennessee: In Tennessee, management of CCPs is specifically regulated as presented in TDEC 1200-1-7-.04. Section two refers to General Facility Standards, which include:

- Overall Performance Standard - The facility must be located, designed, constructed, operated, maintained, closed, and cared for after closure in such a manner as to minimize to the extent practicable:
  - The potential for explosions or uncontrolled fires.
  - The potential for releases of solid wastes, solid waste constituents, or other potentially harmful materials to the environment except in a manner authorized by state and local

US EPA ARCHIVE DOCUMENT

air pollution control, water pollution control, and/or waste management control agencies.

- The exposure of the public to potential health and safety hazards through uncontrolled or unauthorized public access.

- Fire Safety:
  - The facility must have, on-site and continuously available, properly maintained fire suppression equipment in sufficient quantities to control accidental surface fires that may occur, or arrangements must be made with the local fire protection agency to provide immediate fire fighting services when needed.
- Communications - The facility must have operating and effective communications devices (e.g., telephone, 2-way radio) capable of summoning emergency assistance on-site and available to facility personnel at all times the facility is in operation.

Tennessee provides its dam safety regulations in Chapter 1200-5-7, "Rules and Regulations Applied to the Safe Dams Act of 1973". They include:

- Inspection Process
  - State - The public safety and welfare requiring it, the commissioner shall conduct a program of regular inspections of dams, reservoirs, and downstream floodplains within the state. The frequency of such inspections shall be as determined by the commissioner, who may establish different inspection intervals for dams based on their hazard categories. An inspection frequency table is not in the laws or regulations, but a permit cannot be issued for more than five years.
  - Owner - Owner inspections are not specified in the rules and laws. Also, it is not mentioned if owner inspections are required to be conducted by an engineer.

- Emergencies

  The owner is responsible for taking emergency action when necessary but, when the owner fails to take satisfactory action where, in the judgment of the commissioner, the danger to life or property will not permit delay, the commissioner shall request that a state of emergency be declared by the governor, and upon such declaration, shall take such action as he deems necessary to render the dam or reservoir safe.

  The regulations require that new dams with a high-hazard potential rating submit EAPs to the commissioner. The regulations list what should be included in the EAP (1200-5-7-.10).

Selected Design Standard

Because all CCP identified dams in the TVA operations can be addressed by the comprehensive Federal Guidelines for Dam Safety: Emergency Action Planning for Dam Owners (FEMA 64) guidelines, which encourages strict safety standards in the practices and procedures employed by federal agencies or required of dam owners regulated by the federal agencies, this guide shall be used for EAP planning purposes for both types of TVA CCP disposal and impoundment

US EPA ARCHIVE DOCUMENT

facilities. The Guidelines address management practices and procedures, but do not attempt to establish technical standards. They provide the most complete and authoritative statement available of the desired management practices for promoting dam safety and the welfare of the public. The individual facility EAPs must also address any specific requirements of state guidance, such as those set forth in Tennessee as discussed above.

TVA facilities will also prepare and have readily available contingency plans for operational issues. Contingency plans should include protocol to temporarily replace necessary equipment, such as pumps, generators, loaders, etc. to ensure operations can continue without disruption in the event of unexpected events that could affect plant operations. Alternate disposal management planning shall also be included to effectively redirect the CCP materials to an alternate site/location in the event the current disposal plan can no longer operate (e.g., alternate landfill locations, etc.). Planning should include a written plan summarizing possible events that may warrant a shift to a contingency plan, a list and description of alternate disposal options, potential costs and driving factors in the decision process, and any general contact information and/or contract agreements.

FEMA 64 contains guidelines for preparing or revising EAPs for all high and significant hazard potential dams. These guidelines shall also be applied to landfills. Ownership and development of the floodplain downstream from dams and downslope of landfills varies; therefore, the potential for damage to the environment or loss of life as a result of failure or operation of a dam or landfill will also vary. Every EAP must be tailored to site-specific conditions**.**

EAPs generally contain six basic elements described in the next section, including:

- Notification Flowchart (Information provided by TVA appears to list many of the emergency contacts for each facility.)
- Emergency Detection, Evaluation, and Classification
- Responsibilities
- Preparedness
- Inundation Maps
- Appendices

All of these elements should be included in a complete EAP. TVA shall be responsible for the development of the EAP, specific to each facility. However, the development or revision of an EAP shall be done in coordination with those having emergency management responsibilities at the state and local levels as well as familiarity with the facility. Therefore, for new facilities, comprehensive EAPs shall be prepared as part of the permitting process so that they are complete and available at the time of operation. Emergency management agencies will use the information in a dam owner's EAP to facilitate the implementation of their responsibilities. State and local emergency management authorities will generally have some type of plan in place, either a Local Emergency Operations Plan or a Warning and Evacuation Plan.

US EPA ARCHIVE DOCUMENT

The Six Basic Elements of an EAP

The requirements of these elements are discussed in detail in Chapter II of FEMA 64, which presents a recommended format for uniformity among EAPs. Although intended for use with dams, this document adapts the FEMA EAP format for use at landfills.

1. Notification Flowchart. A notification flowchart shows who is to be notified, by whom, and in what priority. The information on the notification flowchart is necessary for the timely notification of persons responsible for taking emergency actions.

2. Emergency Detection, Evaluation, and Classification. Early detection and evaluation of the situation(s) or triggering event(s) that initiate or require an emergency action are crucial. The establishment of procedures for reliable and **timely** classification of an emergency situation is imperative to ensure that the appropriate course of action is taken based on the urgency of the situation. It is better to activate the EAP while confirming the extent of the emergency than to wait for the emergency to occur.

3. Responsibilities. A determination of responsibility for EAP-related tasks must be made during the development of the plan. Owners (such as TVA) are responsible for developing, maintaining, and implementing the EAP. State and local emergency management officials having statutory obligation are responsible for warning and evacuation within affected areas of dams. The EAP must clearly specify the Owner's responsibilities to ensure effective, timely action is taken should an emergency occur at the dam. The EAP must be site-specific because conditions at the dam and downstream of all dams are different.

> Landfills: With landfills, state and local emergency management officials have little or no obligation to address most occurrences associated with landfills. The most prevalent emergency is fire; however, landfill fires are most commonly associated with municipal solid wastes or industrial waste facilities and the need for involvement by local fire officials at a TVA facility is unlikely.

4. Preparedness. Preparedness actions are taken to moderate or alleviate the effects of a failure or operational release and to facilitate response to emergencies. This section identifies actions to be taken before any emergency.

5. Inundation Maps. An inundation map should delineate the areas that would be flooded as a result of a dam failure. Inundation maps are used both by the dam owner and emergency management officials to facilitate timely notification and evacuation of areas affected by a dam failure or flood condition. These maps greatly facilitate notification by graphically displaying flooded areas and showing travel times for wave front and flood peaks at critical locations. Mapping of adjacent properties and knowledge of neighbors may be appropriate for some of the TVA dry disposal facilities as well. Similarly, the possible impact to adjacent areas from a landfill emergency shall be defined in the EAP.

> Landfills: Mapping of adjacent properties and knowledge of neighbors may be appropriate for some of the TVA dry disposal facilities as well. Similarly, the

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possible impact to adjacent areas from a landfill emergency shall be defined in the EAP.

6. Appendices. The contain information that supports and supplements the material used in the development and maintenance of the EAP.

General Procedures to Address Potential Emergencies

In the event a potential emergency is identified (boil, slough, seep, fire, slope failure, etc.), the following general procedures shall be followed:

- Notify the designated shift/ unit supervisor.
- Make an initial assessment as to the severity of the potential emergency and address immediately if possible (i.e. use fire extinguisher to extinguish small, contained fire). For all other situations that cannot be addressed immediately by plant personnel, make appropriate contacts to those on the EAP contact list
    - 911/ Police/ Fire for immediate danger situations;
    - Qualified geotechnical engineer for identified earthen or otherwise man-made impoundments and fills (slope failure, seeps, etc.); and,
    - Appropriate plant personnel for plant and personnel related conditions (e.g. a broken weld on a catwalk should be cordoned off until fixed and approved by plant maintenance).
- Those specialists contacted in certain situations, such as the geotechnical engineer brought in to evaluate a slope failure, may also raise the level of awareness/ severity as necessary, and thereby contact local officials to evacuate in extreme cases, or to otherwise address situations as may be necessary.
- In the vast majority of these cases, the more likely scenario after visit by the engineer will include
    - Fully describe and document the conditions observed (size, shape, severity, volume, color, date, time, time elapsed since first noticed, condition stable/ worsening, etc.)
    - Interview appropriate plant personnel
    - Obtain relevant data (water levels in ponds, adjacent wells/ piezometers, survey data)
    - Make recommendations and implement additional data gathering needs (borings, wells, document review, calculations, drawings, engineering analyses, etc.)
    - Develop a remedial work plan to address the situation
    - Implement the approved remedial plan

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The following sections address specific triggering events or emergencies that should be addressed in Section 2 of the EAP: Emergency Detection, Evaluation, and Classification. If local conditions warrant, other potential triggering events should be identified.

7.1.1 Fire

Fires are common at some landfill sites from a variety of sources. Although not anticipated to be common at TVA facilities due to the type of waste products being disposed, fires are still a possibility, especially when considering heavy equipment use and probable fuel storage. Whatever the cause, it is important that those involved stay calm and have an accessible plan for managing the fire, personnel and neighbors.

Identifying an appropriate fire suppression source (i.e. water, fire extinguishers, etc.) is one of the most critical components of an EAP to address fires. TVA shall identify and coordinate with the respective local fire departments to develop a plan for managing fires, establish evacuation protocols, and to train all pertinent personnel.

7.1.2 Power Outage

A power outage at a facility may result from inclement weather, lightning strikes, capacity or grid problems, or other reasons. Without power, any facility that uses conveyance systems to transport waste may be required to employ alternative power sources or transport wastes for a specified period of time by truck.

At all facilities with non-gravity drained leachate or stormwater systems, the transfer of leachate and stormwater to the appropriate storage or treatment and disposal facilities cannot occur in the event of a power outage unless alternative power sources are available. Depending on the period of power outage, alternative means of power or of liquid transfer may be required. Such contingency plans shall be specified in the EAP.

7.1.3 Leachate Outbreaks

Leachate outbreaks may occur from exterior landfill slopes during operations and even after closure. When an outbreak occurs, the proper steps should be taken to mitigate the situation. This includes the following steps:

- Contain and properly mange the leachate outbreak
- Minimize, control, or eliminate the conditions that are contributing to additional leachate production
- Collect and transport leachate to the appropriate containment or disposal facility

In some instances, the state or local regulatory official will also need to be notified. The appropriate steps for managing a leachate outbreak shall be outlined in the EAP.

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7.1.4 Inclement Weather

Inclement weather can occur throughout the calendar year. It may impede operations at TVA facilities and create unsafe conditions. CCP landfills and disposal impoundments are designed to be in continuous operation. Provisions should be in place to address in the EAP for inclement weather conditions including methods to allow for the receipt and compaction of incoming CCPs and/or temporary storage, as appropriate. Access roads and facilities that are part of the operations should be maintained during inclement weather. If weather creates a situation in which compliance is compromised, the operator may need to contact regulatory officials.

7.1.5 Spills and Releases

This section of the EAP should outline operational methods to manage an accidental spill or release, and the location, capability, and limitations of equipment to be used to contain or remediate the spill. The Response Plan should not provide detailed descriptions, but refer to separate standard operating procedures or detailed technical documents need to specify which plans the reader should be accessing that apply to spill response operations.

The plan should list available on-site and off-site equipment, how it is to be accessed and who has the responsibility for it. The plan should also describe how people and equipment will get to the site, how they will be supported during the crisis, how security and safety will be maintained and how crews will be supplied for the duration of the incident.

7.1.6 Local Transportation Issues

For any site that will require over-the-road transport of CCP materials or leachate for disposal, a plan shall be developed for secondary carriers in the case of equipment breakdown, contractor failure to perform, work stoppages, etc. Also, the plan shall address alternate routes for transport in case of route closures due to reduced load limits on roadways, storm/ washouts of roadways, regular highway maintenance, etc.

On-site transportation issues shall be addressed by the Operator and will include provisions to rent more equipment from outside vendors as the need may arise. All onsite access roads and issues associated with them are the responsibility of TVA or their designated operator, so route issues will be addressed on an as-needed basis.

7.1.7 Other Emergencies

Whether classified as other emergencies or rare operational issues that need to be addressed quickly, there are several other potential problems that may need to be addressed in the individual plant EAPs.

In landfills, waste settlement and/or slope failure of the waste and/ or interior sideslopes, or final existing cover soils, can occur during construction, during operation, or after closure. Waste settlement or slope failure may expose waste and require timely, if not emergency, repairs. In

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rare instances, large displacement failures may also jeopardize operations or human health and the environment. Settlement in a CCP landfill should be minimal if waste is properly placed and compacted due to the uniformity of CCPs, but may be caused by consolidation of waste or foundation materials under the weight of waste material and cap. Possible consequences of settlement include instability in the waste or cover soil, which can damage the cap, and disruption of storm water drainage.

The weight of the new cap can be significant enough to cause additional waste settlement and compaction. The effect of this additional weight may initiate differential settlement across the cap, thus compromising the integrity of the cap, or create stability problems such as slippage failures in the waste and/or existing cover system. Differential settlement occurs when one area of waste settles more readily than another because of differences in moisture content, waste compaction, or waste composition. Settlement, and especially differential settlement, may create cracks in the cap and allow rainwater to reach the waste. Changes in the topography of the landfill because of settlement may also create areas on the cap surface where rainwater can pond. These conditions, if left unchecked, can lead to bigger issues of slope instability and failure, and leachate generation.

The following list provides other typical types of landfill facility emergencies requiring emergency actions be taken:

- Slope Failures and Settlement

  Slope failures, cracks or sloughing in constructed earthen slopes should be fixed immediately as part of construction. The facility design, construction specifications, construction procedures, construction oversight, or all of these possible reasons should be investigated to determine the cause of the failure and to prevent re-occurrence of slope failures during construction. The observance and notation of these types of failures resides with the construction contractor and CQA oversight firm.

  During operation, slope failures or ponding water should be addressed immediately as part of operations by regrading waste to eliminate ponding and to repair any failed waste slopes. Operational procedures, grade control, compaction control, moisture control, and record-keeping should all be investigated to determine the cause of internal waste slope failures or settlement and to prevent any recurrence.

  Although settlement and moreover differential settlement should be minimal in these types of residual waste landfills, any obvious settlement of the CCPs should be addressed as part of operation the landfill to maintain positive surface water drainage. Differential settlement is typically addressed by the addition of more waste (if feasible in an active waste placement area of the landfill) or the by the addition of intermediate or final cover soil, depending on where the settlement occurs. Additional fill should be added to return the grades to the necessary design grades to promote proper drainage.

In above-grade ponds, impoundments or wet stacks, emergency issues may involve the movement of water through the dike (seeps), beneath the dike (boils) or over the dike (overflows) which may lead to significant erosion leading to dike failure. The movement of

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water, when left unchecked, leads to erosion of the earthen structure until failure occurs. TVA dam safety will be reviewing all the ash disposal impoundments and will decide which will be considered to be dams. Those that are will be brought under the TVA Reservoir Operations, Dam Safety Inspection and Maintenance program. Emergency responses for dams are covered by EAPs administered by that program. For all of these earthen structures, regular inspections will be made to look for specific indications of pending problems. If encountered, the EAPs will identify the contacts to be made and the procedures to be enacted, such as follow.

- Once noted on an inspection form, notation of seeps and boils should require the inspection by a qualified engineer. Depending on the advice of the engineer, measures may be taken to stop or slow the water flow by placement of sandbags or additional earthen material, followed by a period of planned investigation into the cause of the outbreak. Once the cause is determined, an engineered fix can be designed an implemented to stop the flow and reduce the hydrostatic head pressure.

- Overtopping of any impoundment should be prevented through regular inspection procedures however can occur for several reasons (significant storm events, plugged pipes or drains, etc.) The EAP for all impoundments shall provided measures for quickly lowering water levels or redirecting overflows should they occur.

7.2 COMMUNICATION PROCEDURES

During an emergency, effective and reliable electronic communications equipment and procedures are vital. This section of the plan should detail the types of communication equipment to be used by personnel during an emergency response. Since normal means of communication can break down in an emergency, alternative means must be considered. Cellular telephones, public address systems, two-way radios and messengers should be used.

Training and arrangements may be necessary to ensure that telephone services are available for official calls during an emergency and that unauthorized calls will not be placed. Within an Incident Command Post, telephone circuits may quickly become jammed with calls. Direct hot lines that are not available to outside lines may be considered for critical communications. Use of 1-800 numbers for public inquires is another option to manage external calls.

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APPENDIX A

Document 17

Seepage Action Plan (SAP), Kingston Fossil Plant, Harriman, Roans County, Tennessee, Stantec Consulting Services, June 10, 2010

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Seepage Action Plan (SAP)

Kingston Fossil Plant
Harriman, Roane County,
Tennessee

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1409 North Forbes Road
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Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

June 25, 2010

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Seepage Action Plan (SAP)

Kingston Fossil Plant
Harriman, Roane County,
Tennessee

Prepared for:
Tennessee Valley Authority
Chattanooga, Tennessee

June 25, 2010

Seepage Action Plan (SAP)

Kingston Fossil Plant
Harriman, Roane County, Tennessee

Table of Contents

Section **Page No.**

1. Potential Seepage Areas 1

2. Basic SAP Data 1
2.1. Purpose 1
2.2. Potential Impacted Area 2
2.3. Primary Responsibility and Frequency of Dike Safety Inspections 2

3. Seepage Action Level Determination 2
3.1. Action Level 1 – Non Flowing 3
3.2. Action Level 2 – Flowing Seepage – No Erosion 3
3.3. Action Level 3 – Flowing Seepage – Active Erosion 4

4. Intermediate Corrective Measures 6
4.1. Action Level 1 – Non Flowing 6
4.2. Action Level 2 – Flowing Seepage – No Erosion 6
4.3. Action Level 3 – Flowing Seepage – Active Erosion 7

5. Materials On-Site 8

6. The SAP Process 9
6.1. Step 1 – Dike Observation or Event Detection 9
6.2. Step 2 – Emergency Level Determination 9
6.2.1. Action Level 1 – Non Flowing 9
6.2.2. Action Level 2 – Flowing – No Erosion 9
6.2.3. Action Level 3 – Flowing – Active Erosion 10
6.3. Step 3 – Notification and Communication 10
6.3.1. Notification 10
6.3.2. Communication 10

List of Tables

Table **Page No.**

Table 1. Stockpile Material Quantities 8

US EPA ARCHIVE DOCUMENT

Table of Contents
(Continued)

List of Figures

Figure		Page No.
Figure 1.	Seepage Inspection Location	1
Figure 2.	Example of Action Level 1 – Non-Flowing – Wet Area	3
Figure 3.	Example of Action Level 2 – Clear Flowing – Seepage Boil	4
Figure 4.	Example of Action Level 3 – Turbid Flowing – Seepage Boil	5
Figure 5.	Example of Action Level 3 – Deposition of Sediment from Dike	5
Figure 6.	Example of Action Level 3 – Underwater Turbid Flowing – Seepage Boil	6
Figure 7.	Sand Bag Treatment (Temporary)	8
Figure 8.	Level 2 Emergency Contact Flowchart	11
Figure 9.	Level 3 Emergency Contact Flowchart	12

List of Appendixes

Appendix A	Peninsula Gypsum Pond, Dredge Cell, Main Ash Pond, and Stilling Pond Site Plans
Appendix B	Possible Seepage Problems and Recommendations
Appendix C	Seepage Log
Appendix D	KIF CCP Emergency Action Plan

US EPA ARCHIVE DOCUMENT

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Seepage Action Plan (SAP)

Kingston Fossil Plant
Harriman, Roane County, Tennessee

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 Peninsula Gypsum Pond, Dredge Cell, Main Ash Pond and Stilling Pond. 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.

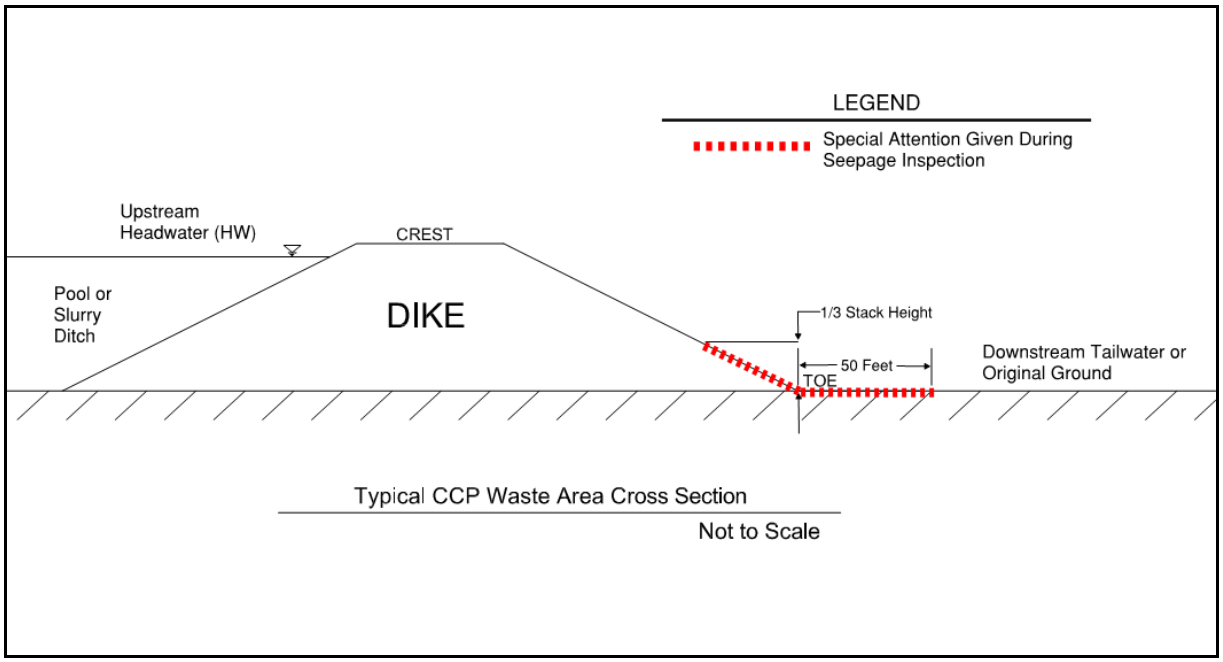


Figure 1. Seepage Inspection Location

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.

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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 Kingston 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 Kingston Fossil Plant and the Tennessee River.

2.3. Primary Responsibility and Frequency of Dike Safety Inspections

1. TVA RHO&M Field Supervisor for Kingston Fossil Plant (**Field Supervisor**)
2. TVA RHO&M East Region Construction Manager
3. TVA RHO&M Program Manager for Kingston 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

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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.

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Figure 4. Example of Action Level 3 – Turbid Flowing – Seepage Boil

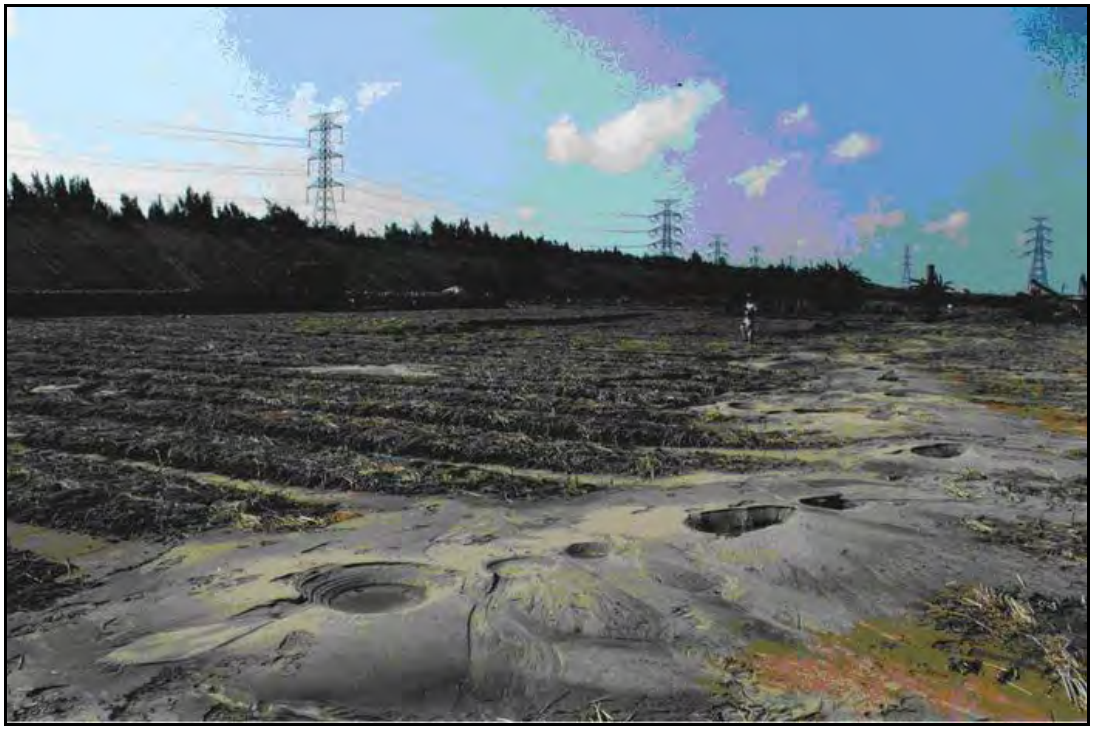


Figure 5. Example of Action Level 3 – Deposition of Sediment from Dike

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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 (TDOT Concrete Sand)
  - One foot of TDOT No. 89 Stone
  - Twp feet of TDOT Machine Rip Rap Class A-3
  - 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**.

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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
TDOT Machine Rip Rap Class A-3	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 location determined by the **Field Supervisor.** 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 Kingston 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 KIF 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 KIF personnel if repairs are needed
- Determine if other work activities need to be made aware of new conditions.

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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.

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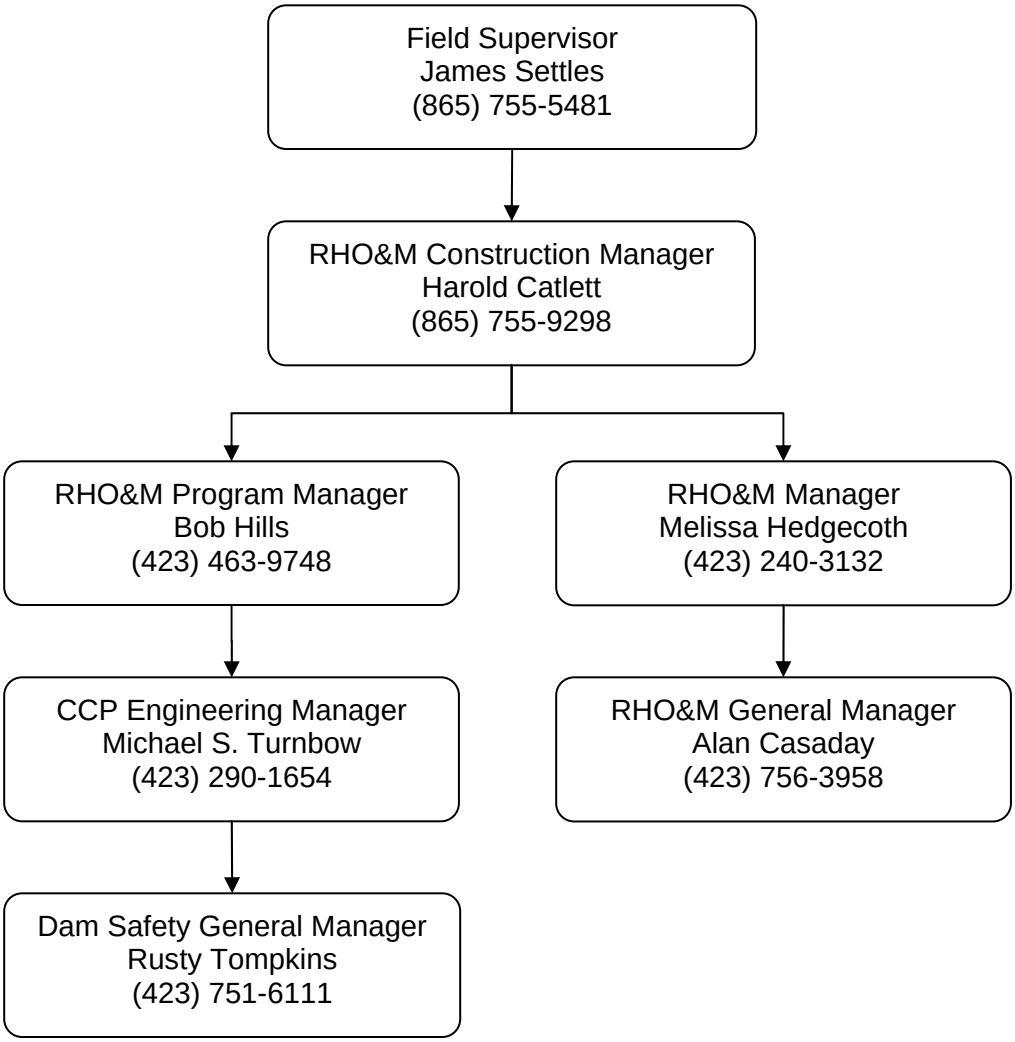


Figure 8. Level 2 Emergency Contact Flowchart

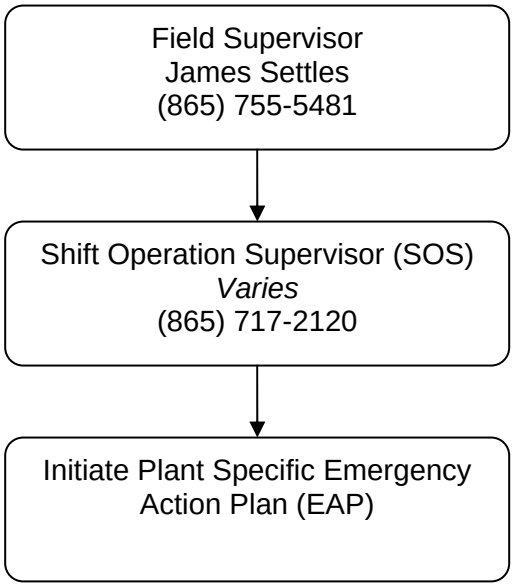


Figure 9. Level 3 Emergency Contact Flowchart

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Appendix A

Peninsula Gypsum Pond, Dredge Cell, Main Ash Pond, and Stilling Pond Site Plans

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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

Potential Seepage Zone

0 250 500 Feet

STANTEC CONSULTING SERVICES INC.
1409 N. Forbes Rd.
Lexington, Kentucky
40511-2050
859-422-3000

Stantec

DRAFT

SEEPAGE ACTION PLAN

Tennessee Valley Authority
Kingston Fossil Plant
Kingston, Roane County, Tennessee

PROJECT NO.	175560021
DATE	MARCH 2010
DRAWN BY	AMG
CHECKED BY	
CHECKED BY	
SCALE	1" = 500 '
REVISED	
1.	
2.	
3.	
4.	
5.	
6.	
7.	
8.	

SHEET

1 OF 2

V:\1.7560\active\175560021\gis\mxd\kingston.mxd

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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
0 250 500

STANTEC CONSULTING SERVICES INC.
1409 N. Forbes Rd.
Lexington, Kentucky
40511-2050
859-422-3000

Stantec

DRAFT

SEEPAGE ACTION PLAN

Tennessee Valley Authority
Kingston Fossil Plant
Kingston, Roane County, Tennessee

PROJECT NO.	175560021
DATE	MAY 2010
DRAWN BY	AMG
CHECKED BY	
CHECKED BY	
SCALE	1" = 500 '
REVISED	
1.	
2.	
3.	
4.	
5.	
6.	
7.	
8.	

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SHEET
2 OF 2

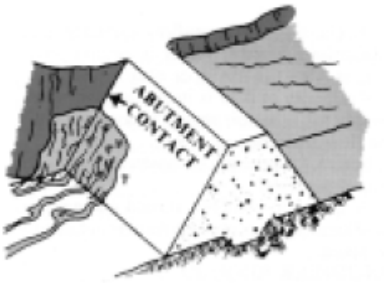
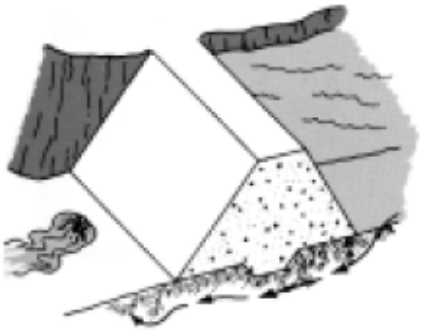
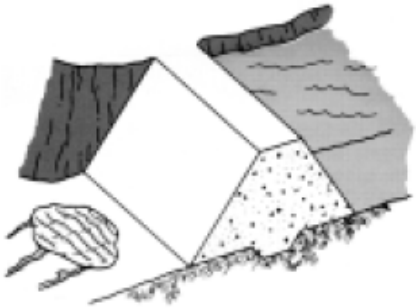
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Appendix B

Possible Seepage Problems and Recommendations

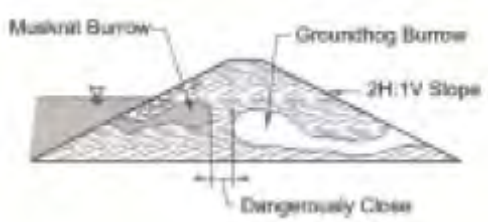
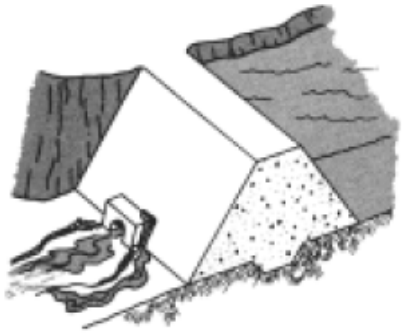
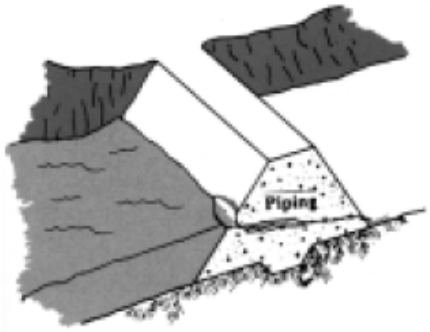
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Appendix B – Possible Problems and Recommendations

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.

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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.

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Appendix C

Seepage Log

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KIF Seepage Log
Kingston Fossil Plant
Kingston, Tennessee
Updated 6/22/2010 Rev. 1

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		3/20/2010	N/A	-	1	Wet weather rutting on the access road-initially discovered by Stantec on 3/20/2010. SAP Level 1.	This area should be monitored for change. Once the area is dry the slopes should be graded to allow for proper drainage.
2	Survey Requested		5/4/2010	N/A	-	1	Seepages observed by Stantec on 5/4/2010 at multiple locations near the waterline at the southern portions of the wetland. SAP Level 1	Monitoring of the seeps and ongoing analysis being conducted by Stantec.
3	Survey Requested		3/20/2010	N/A	-	1	Multiple seeps near the toe of the exterior northeast slope of the stilling pond. Initially observed by Stantec on 3/20/2010. SAP Level 1	Monitoring of the seeps and ongoing analysis being conducted by Stantec.
4	554532.07	2442015.24	5/4/2010	N/A	-	1	Possible red water seepage into Stilling Pond through Divider Dike-Initially Observed on 5/4/2010 by Stantec during the 2010 Annual Inspection. SAP Level 1	This area should be monitored for change.

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

3/20/2010
Wet weather rutting on the access road:
Initially discovered by Stantec on 3/20/2010.
SAP Level 1.



Area of Concern 1

3/20/2010
Wet weather rutting on the access road:
Initially discovered by Stantec on 3/20/2010.
SAP Level 1.



Area of Concern 2

5/4/2010
Wet weather rutting on the access road:
Initially discovered by Stantec on 5/4/2010.
SAP Level 1.

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Area of Concern 3

3/20/2010
Multiple seeps near the toe of the exterior northeast slope of the stilling pond. Initially observed by Stantec on 3/20/2010. SAP Level 1



Area of Concern 3

3/20/2010
Multiple seeps near the toe of the exterior northeast slope of the stilling pond. Initially observed by Stantec on 3/20/2010. SAP Level 1



Area of Concern 4

5/4/2010
Possible seepage into Stilling Pond through Divider Dike. Initially Observed on 5/4/2010 by Stantec. SAP Level 1

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Appendix D

KIF CCP Emergency Action Plan

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APPENDIX A

Document 18

Stantec Consulting Services Memo Dated August 8, 2011 "Reference: Dike C Slope Inclinometer Data Weekly Review"

Memo

Stantec Consulting Ltd.
4370 Dominion Street, 5th Floor
Burnaby, BC V5G 4L7
Tel: (604) 436-3014
Fax: (604) 436-3752

To: Tod Woodson **From:** Tom Crilly

Project: Kingston Fossil Plant Date: August 8, 2011

Reference: **Dike C Slope Inclinometer Data Weekly Review**

Because of annual calibration of the SI probe used on site, all slope inclinometers have been re-baselined on July 29, 2011 using a new (rental) SI probe. The limited monitoring of the SI's using the new probe are attached on Figures 1 – 7.

The limited data using the new probe is generally consistent with the movements monitored using the original probe.

Attachment: SI Data

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APPENDIX A

Document 19

Stantec Consulting Services Memo Dated August 8, 2011, "Reference" Dike C Weekly Piezometer Data Review (Data from July 22, 2011, to August 5, 2011)"

Memo

Stantec Consulting Ltd.
4370 Dominion Street, 5th Floor
Burnaby, BC V5G 4L7
Tel: (604) 436-3014
Fax: (604) 436-3752

Stantec

To: Tod Woodson **From:** Tom Crilly

Project: Kingston Fossil Plant Date: August 8, 2011

Reference: **Dike C Weekly Piezometer Data Review (Data from July 22, 2011 to August 5, 2011)**

The weekly monitoring results for the piezometers at Dike C are included on the attached Figures B-22 to B-29. The attached piezometer results include monitoring up to August 5, 2011. Outlined below is a summary of the monitoring results from July 22, 2011 to August 5, 2011. The water level of Watts Bar Lake has remained approximately the same during the period at 740.3 ft (August 6). Trigger levels will be referenced to summer lake levels.

Sta. 108+93 (PZ-17 to PZ-20) – Figure B-22

- All PZ's decreased by 0 to 0.6 ft.
- The identified critical piezometer for this group (PZ-19(L)) decreased by 0.6 ft to Elev. 749.9 ft. This remains below the established trigger level of 760.0 ft.

Sta. 119+69 (PZ-13 to PZ-16) – Figure B-23

- All piezometers decreased by 0 to 1.1 ft.
- The identified critical piezometer for this group (PZ-13) decreased by 1.1 ft to Elev. 740.0 ft, below the established trigger level of 742.5 ft.
- PZ-129 located within the starter dike at Sta. 122+00 **(Figure B-27)** has decreased from 741.3 ft on July 19 to 740.4 ft on August 4. The piezometers at Sta. 122+00 have been monitored on a daily basis since June 22. All piezometers within this group have generally decreased during the monitoring period July 19 to August 4.

Sta. 132+37 (PZ-9 to PZ-12) – Figure B-24

- All PZ's decreased by 0 to 0.5 ft.
- The identified critical piezometer for this group (PZ-9) decreased by 0.5 ft to Elev. 741.2 ft. This remains below the established trigger level of 754.0 ft.

Reference: Dike C Weekly Piezometer Data Review (Data from July 22, 2011 to August 5, 2011)

Sta. 143+05 (PZ-5 to PZ-8) – Figure B-25

- All PZ's decreased by 0 to 0.8 ft.
- The identified critical piezometer for this group (PZ-5) was unchanged at Elev. 742.9 ft. This remains below the established trigger level of 754 ft.
- The piezometers installed within the starter dike at Sta. 138+27 have shown a similar trend of following the level of the lake, and are 1.6 ft lower than PZ-5. For the piezometers installed within the raised dike; PZ-124A (shallow screen tip) is reading 4.2 ft lower than PZ-6(U); PZ-124B (intermediate screen tip) is approximately 3.9 ft lower than PZ-7(L); and PZ-125 (deep screen tip) is reading approximately 2 ft higher than PZ-8. All piezometers within this group have generally decreased by 0.3 to 1.4 ft over the monitoring interval. The piezometer readings for Sta. 138+27 are shown on **Figure B-28.** From the monitoring to date, there is no indication that the stability analysis at Sta. 138+27 using piezometers at Sta. 143+05 (to be representative of the dike from Sta. 138+00 to 146+50) should be revised to include the piezometer results at Sta. 138+27.

Sta. 154+82 (PZ-1 to PZ-4) - Figure B-26

- All PZ's decreased by 0.1 to 2.4 ft.
- The identified critical piezometer for this group (PZ-3(L)) decreased by 2.4 ft to Elev. 750.6 ft. This is below the established trigger level of 762 ft.
- The piezometers installed within the starter dike at Sta. 149+14 have shown a similar trend of following the level of the lake, and are being monitored 1.3 to 3.9 ft higher than the starter dike piezometer at Sta. 154+82. For the piezometers installed within the raised dike; PZ-121A (shallow screen tip) is reading 0.5 ft higher than PZ-2(U); PZ-121B (intermediate screen tip) is approximately 3.4 ft lower than PZ-3(L); and PZ-122 (deep screen tip) is reading approximately 1.4 ft lower than PZ-4. The piezometer readings for Sta. 149+14 are shown on **Figure B-29.** From the monitoring to date, there is no indication that the stability analysis at Sta. 149+14 using piezometers at Sta. 154+82 (to be representative of the dike from Sta. 146+50 to 160+00) should be revised to include the piezometer results at Sta. 149+14.

Summary

None of the established critical piezometers reached their established trigger levels for the weekly piezometer monitoring to August 5, 2011. The PZ's monitored daily at Sta. 122+00 adjacent to the buttress construction decreased slightly during the monitoring period.

Attachment: PZ Data

C:\Users\tcrilly\Desktop\TVA Invoicing and files working in wpg\monitoring to August 5 2011\August_5_2011_PZ_review.docx

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APPENDIX A

Document 20

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-

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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.

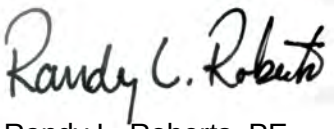
Results

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

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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

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Kingston Fossil Plant
(KIF)

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Pseudostatic Slope Stability Analysis
CCP Storage Facilities - Existing Conditions
Tennessee Valley Authority Fossil Plants

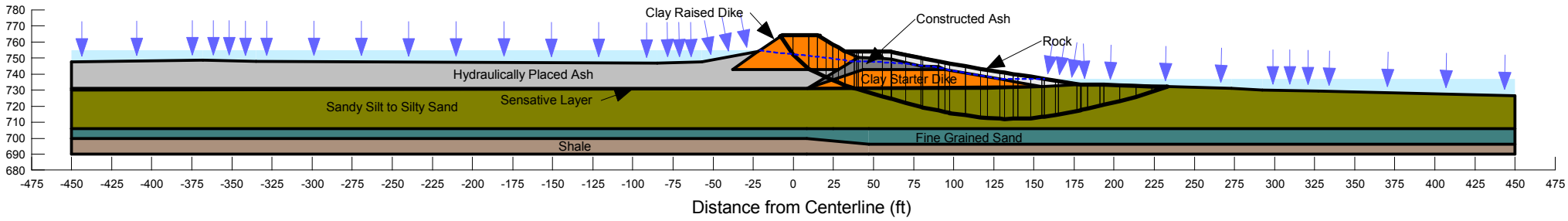
Section STA. 132+37
Kingston Fossil Plant
Harriman, Tennessee

Note:
The results of the 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
Clay Raised Dike	125 pcf	65 psf	23 °
Constructed Ash	93 pcf	0 psf	30 °
Hydraulically Placed Ash	96 pcf	0 psf	10 °
Sandy Silt to Silty Sand	105 pcf	0 psf	27 °
Fine Grained Sand	122 pcf	0 psf	31 °
Clay Starter Dike	129 pcf	300 psf	26 °
Sensative Layer	127 pcf	c/p = 0.32	N/A
Rock	110 pcf	0 psf	38 °
Shale	N/A	N/A	N/A

Factor of Safety: 1.0

Horizontal Seismic Coefficient Kh = 0.115 g
2500-year Return Period Event



Date of Assessment - 11/4/2011

Project No 175551015

BORING LOCATION TABLE			
Boring No.	Northing	Easting	Elevation (Feet)
STN-2	556804.57	2442329.32	751.20
STN-2B	556801.29	2442323.04	751.00
STN-3	556756.78	2442263.59	763.70
STN-3C	556763.50	2442257.42	763.80
STN-3D	556762.50	2442258.42	763.80
STN-3E	556764.10	2442559.45	763.80
STN-4	556625.56	2442047.01	763.30
STN-5	556611.15	2442365.31	764.90
STN-6	556416.17	2442407.10	763.40
STN-8	556248.52	2442540.30	752.20
STN-8A	556254.96	2442535.17	751.70
STN-9	556233.82	2442499.72	764.80
STN-9A	556234.82	2442499.81	764.80
STN-10	556162.76	2442251.63	765.00
STN-11	556034.63	2442535.05	763.20
STN-12	555873.09	2442622.48	765.10
STN-14	555685.58	2442733.21	753.10
STN-14B	555690.29	2442730.41	753.00
STN-15	555662.69	2442667.52	763.70
STN-15C	555671.50	2442690.88	765.30
STN-15D	555665.72	2442690.78	765.30
STN-15E	555694.24	2442681.34	765.00
STN-16	555501.40	2442725.94	764.50
STN-18	555204.87	2442894.11	751.00
STN-18A	555204.57	2442886.00	751.80
STN-19	555204.68	2442842.94	765.60
STN-19A	555204.68	2442843.94	765.60
STN-20	555168.84	2442666.59	762.90
STN-21	555076.61	2442149.85	765.00
STN-22	554990.27	2441723.40	765.00
STN-23	555020.22	2442857.46	764.70
STN-24	554803.45	2442843.16	765.10
STN-26	554624.86	2442889.00	750.00
STN-26B	554604.57	2442896.12	751.00
STN-27	554601.77	2442850.67	765.10
STN-27A	554600.64	2442840.21	765.00
STN-27B	554606.18	2442840.52	765.00
STN-27C	554607.18	2442840.92	765.00
STN-28	554406.25	2442841.10	764.80
STN-29	554155.15	2442854.72	764.70
STN-31	553954.94	2442758.22	749.50
STN-31A	553980.30	2442764.66	749.70
STN-32	553994.90	2442746.44	764.80
STN-32A	553996.72	2442757.13	764.80
STN-34	553882.51	2442381.24	764.70

BORING LOCATION TABLE			
Boring No.	Northing	Easting	Elevation (Feet)
STN-36	553776.74	2442198.78	751.90
STN-36B	553753.47	2442155.66	751.50
STN-37	553799.90	2442184.40	763.80
STN-37C	553798.81	2442171.01	763.60
STN-37D	553798.26	2442168.95	763.60
STN-37E	553799.26	2442183.40	763.80
STN-38	553730.83	2441988.70	764.10
STN-41	553583.10	2441510.71	752.70
STN-41A	553584.59	2441512.86	751.80
STN-42	553623.48	2441513.69	764.70
STN-42A	553624.48	2441512.69	764.70
STN-43	554004.68	2441548.50	765.90
STN-43A	554005.71	2441549.61	765.90
STN-45	553740.50	2441308.40	763.90
STN-47	553747.39	2441146.83	753.40
STN-47B	553742.43	2441143.28	753.80
STN-48	553773.29	2441154.53	765.30
STN-48C	553770.51	2441164.28	765.30
STN-49	553921.85	2441024.39	763.10
STN-50	553631.96	2440496.85	741.60
STN-50A	553632.98	2440497.85	741.60
STN-51	553696.02	2440548.48	750.40
STN-52	553992.98	2440817.40	753.20
STN-53	554011.12	2440902.46	763.90
STN-53A	554012.14	2440904.05	763.90
STN-54	555263.94	2441476.12	765.00
STN-55	554943.73	2442287.49	764.10
STN-56	554555.61	2441998.50	765.80
STN-59	556075.53	2442603.06	752.20
STN-60	555886.92	2442663.31	752.50
STN-61	555513.59	2442792.80	752.50
STN-62	555020.69	2442907.23	749.80
STN-63	554822.75	2442910.57	750.00
STN-64	554411.29	2442911.08	749.40
STN-65	554147.51	2442915.09	748.60
STN-66	553888.83	2442584.24	750.90
STN-69	553607.58	2441718.01	752.30
STN-71	553840.20	2440981.04	752.00
STN-72	556243.56	2442498.40	765.70
STN-73	556264.59	2442544.34	763.90
STN-74	555189.21	2442848.63	766.20
STN-74	555184.59	2442849.41	766.20
STN-75	555194.95	2442895.83	753.40
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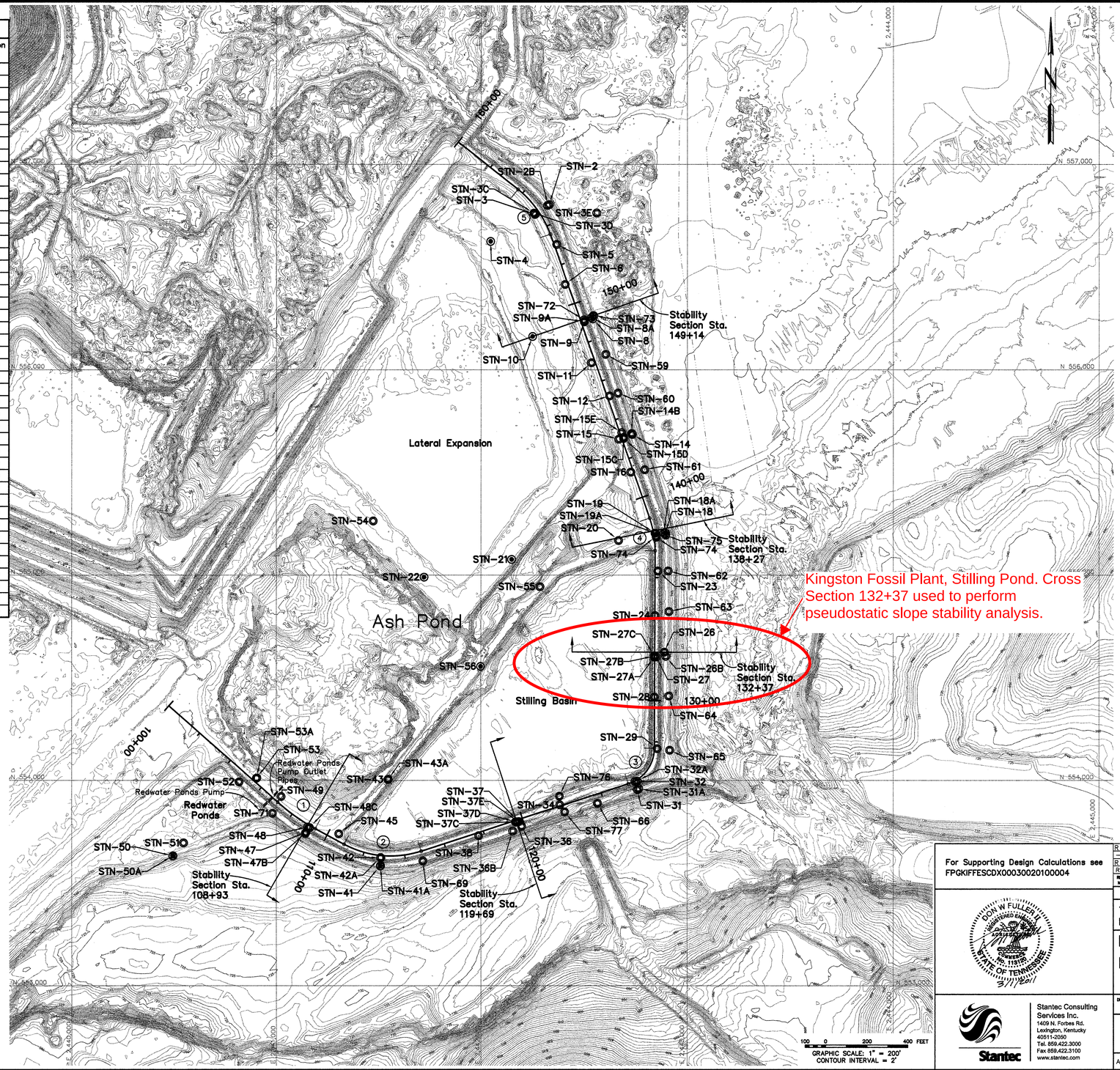


TABLE OF BASELINE COORDINATES			
Station	Baseline	Northing	Easting
100+00.00	Begin Project Baseline	554,363.86	2,440,477.77
160+00.00	End Project Baseline	557,109.43	2,441,888.38

BASELINE CURVE DATA

① P.I. Sta. = 108+20.74; Northing = 553,819.18; Easting = 2,441,091.71; Δ = 13°20'06"; R = 974.68'; T = 113.94'; L = 226.85'; E = 6.64'; P.C. Sta. = 107+06.80; P.T. Sta. = 109+33.65

② P.I. Sta. = 112+99.93; Northing = 553,591.92; Easting = 2,441,514.77; Δ = 46°19'10"; R = 479.84'; T = 205.26'; L = 387.92'; E = 42.06'; P.C. Sta. = 110+94.67; P.T. Sta. = 114+82.59

③ P.I. Sta. = 126+71.85; Northing = 554,024.61; Easting = 2,442,840.47; Δ = 71°45'50"; R = 182.94'; T = 132.33'; L = 229.13'; E = 42.85'; P.C. Sta. = 125+39.52; P.T. Sta. = 127+68.65

④ P.I. Sta. = 138+12.12; Northing = 555,200.41; Easting = 2,442,843.77; Δ = 19°21'32"; R = 479.84'; T = 81.84'; L = 162.13'; E = 6.93'; P.C. Sta. = 137+30.28; P.T. Sta. = 138+92.41

⑤ P.I. Sta. = 154+96.30; Northing = 556,792.40; Easting = 2,442,289.43; Δ = 32°28'34"; R = 479.84'; T = 139.75'; L = 271.98'; E = 19.94'; P.C. Sta. = 153+56.55; P.T. Sta. = 156+28.53

LEGEND

- Soil Boring with Undisturbed (Shelby) Tube Samples and/or Standard Penetration Tests
- Soil Boring with Undisturbed (Shelby) Tube Samples and/or Standard Penetration Tests and Rock Core
- Previously Drilled by AECOM
- Cone Penetration Test

NOTES

1. Topographic mapping was generated from a LIDAR survey performed on January 8, 2009 by Tuck Mapping Solutions, Inc. This plan was developed for discussion purposes only and is not to be used for construction.
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.

FOR INFORMATION ONLY
This Record Drawing which has been previously submitted to TVA is provided for Information Only.

RECORD DRAWING

For Supporting Design Calculations see FPGKIFFESCDX00030020100004

DON W. FULLER II REGISTERED ENGINEER 119130 STATE OF TENNESSEE 5/17/2011

Stantec Consulting Services Inc.
1409 N. Forbes Rd.
Lexington, Kentucky
40511-2050
Tel. 859.422.3000
Fax 859.422.3100
www.stantec.com

REV. NO.	DATE	DSGN	DRWN	CHKD	SUPV	RVWD	APPD	ISSD	PROJECT MGR	AS CONST
R 0	03/01/11	JDP	TJ	TC	DWF	–	MST	JCK	–	–
RECORD DRAWING										

SCALE: 1"=200' EXCEPT AS NOTED

YARD
DIKE C

BUTTRESS DESIGN
BORING PLAN AND BASELINE LAYOUT

DESIGNED BY: J.D. PINEAU	DRAWN BY: T. JOHNSON	CHECKED BY: T. CRILLY	SUPERVISED BY: D.W. FULLER	REVIEWED BY: –	APPROVED BY: M.S. TURNBOW	ISSUED BY: J.C. KAMMEYER

KINGSTON FOSSIL PLANT
TENNESSEE VALLEY AUTHORITY
FOSSIL AND HYDRO ENGINEERING

AUTOCAD R 2000	DATE 03/01/11	36	C	10W287-01	R 0

PLOT FACTOR: 200
W_TVA

C.A.D. DRAWING
DO NOT ALTER MANUALLY

STANTEC	0
TASK COMPLETED BY:	REV NO.

PLOT DATE: 05/24/2011 USER: HOWL GORDON
S:\KINGSTON\SITES\1726660\3_ENGINEERING\KF_DWGS\CIVIL\BUTTRESS\BORINGPLAN\REV_1\KINGTONKF-C-05-256-2011\KINGKF-C-05-256-2011.DWG

US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 21

Geosyntec Consultants "Foundation Stability Analyses," Kingston Peninsula Permit Modification, October 29, 2010

GEOSYNTEC CONSULTANTS

COMPUTATION COVER SHEET

Client: TVA **Project:** Kingston Peninsula Permit Modification **Project/Proposal #:** GR4694 **Task #:** 05

TITLE OF COMPUTATIONS **FOUNDATION STABILITY ANALYSES**

COMPUTATIONS BY:
Signature Kushwant Chohan/Fan Zhu DATE 10/29/2010
Printed Name and Title Kush Chohan/Fan Zhu, Senior Staff Engineer/ Senior Staff Engineer

ASSUMPTIONS AND PROCEDURES CHECKED BY: (Peer Reviewer)
Signature R. Kulasingam DATE 11/01/2010
Printed Name and Title R. Kulasingam, Senior Engineer

COMPUTATIONS CHECKED BY:
Signature Fan Zhu DATE 10/29/2010
Printed Name and Title Fan Zhu, Senior Staff Engineer

COMPUTATIONS BACKCHECKED BY: (Originator)
Signature Kushwant Chohan/Fan Zhu DATE 10/29/2010
Printed Name and Title Kush Chohan/Fan Zhu, Senior Staff Engineer/ Senior Staff Engineer

APPROVED BY: (PM or Designate)
Signature Majdi Othman DATE 11/2/10
Printed Name and Title Majdi Othman, Principal

APPROVAL NOTES: _____

REVISIONS (Number and initial all revisions)

NO.	SHEET	DATE	BY	CHECKED BY	APPROVAL

FOUNDATION STABILITY ANALYSES

PURPOSE

The purpose of this calculation package is to evaluate the static and seismic slope stability of the proposed disposal facility at the Kingston Fossil Plant (KIF). For these analyses, potential slip surfaces passing through the waste material and underlying native foundation soils are considered. It is noted that the analysis presented herein is based on the dry disposal of the waste materials with the exception of the wet disposal of the gypsum in the Phase I area for the first year of operation.

METHOD OF ANALYSIS

Static Stability Analysis:

Slope stability analyses were performed using the Spencer method [Spencer, 1967] with the circular search method for potential slip surfaces as implemented in the computer program SLIDE [2009]. The computer program was used to generate potential slip surfaces and calculate the factor of safety for each of these surfaces. SLIDE identifies the slip surface with the lowest factor of safety. Information required for the analyses include:

- the geometry of the gypsum (Phase I) and fly ash (Phase II) disposal facility at the cross section location;
- the subsurface soil stratigraphy at the cross section location;
- the material properties for gypsum, fly ash, subgrade fill, and subsurface materials;
- the water level within the gypsum and fly ash stacks; and
- the groundwater table elevation along the cross section location.

Analyses were performed for the final build out of Phase I and II. Both drained and undrained analyses were performed.

Seismic Stability Analysis:

Seismic slope stability analyses were performed using a procedure consistent with a guidance document prepared by the U.S. Environmental Protection Agency [USEPA, 1995]. The procedure is as follows:

1. Estimate the peak ground acceleration (PGA) at the site. Estimate the peak horizontal acceleration (a_{max}) of the potential critical slip surface based on PGA.

2. Perform pseudo-static slope stability analyses for the potential critical section to evaluate the yield acceleration. The yield acceleration is the horizontal acceleration at which a marginally stable condition is produced (i.e., factor of safety of 1.0) for the potential slip surface. A trial-and-error process was applied to evaluate the yield acceleration.

3. The yield acceleration (k_y) was compared to the peak horizontal acceleration (a_{max}) of the slide mass due to the design earthquake. If k_y is greater than a_{max}, the analysis is concluded, as the landfill will not likely undergo permanent displacement. If k_y is less than a_{max}, then the landfill will likely undergo permanent displacement and a displacement analysis is performed to evaluate the magnitude of the permanent displacement.

4. The seismic displacement, corresponding to the computed k_y/a_{max} ratio, is estimated using the results presented by Hynes and Franklin [1984] and the "modified mean + one standard deviation curve" developed by Geosyntec, as presented in Figure 1. The "modified mean + one standard deviation curve" considers data associated with only large earthquakes, and therefore, is more conservative to use. This procedure is consistent with those given in the USEPA guidance document [USEPA, 1995].

According to United States Geological Survey (USGS) seismic hazard map [2008], the PGA with a 2 percent probability of exceedence in 50 years (or 10 percent probability of exceedence in 250 years) is 0.20 g for the KIF site as presented in Figure 2. The peak horizontal acceleration (a_{max}) was assumed the same as the PGA because subsurface soil conditions were considered not to have the potential for amplification of the ground motion.

Target Factors of Safety:

The target calculated factor of safety for static stability analyses is 1.5.

The criterion for seismic stability is based on calculated permanent deformation. Based on the limiting seismic slope stability design criteria of the Tennessee Division of Solid Waste Management, a division of the Tennessee Department of Environment and Conservation (TDEC) [TDEC, 1993], "*No landfill shall be acceptable if the predicted seismic induced deformations within the waste fill exceed one-half the thickness of the clay liner component of the liner system.*" Also, based on the recommendations of Seed and Bonaparte [1992] and Anderson and Kavazanjian [1995] for seismic

analysis, maximum permanent seismic deformations of 6 to 12 inches are typically acceptable for the design of geosynthetic liner systems.

CROSS SECTIONS ANALYZED

Five cross sections were analyzed. The locations of the cross sections with respect to the final cover system of the KIF Phase I and Phase II facility features are shown in Figure 3. The cross section geometries at each location (including gypsum/fly ash, soil stratigraphy, water table, and piezometric surface within the material) are shown in Figures 4 through 8. Each cross section is considered potentially critical since the maximum waste height and slope are located at these sections.

SITE STRATIGRAPHY

Information on the site stratigraphy used in these analyses is summarized in MACTEC [2005], MACTEC [2006], and TVA [2005]. The top of bedrock elevations were obtained from a contour map developed from a series of site investigations that included soil borings, CPT soundings, and GeoProbe soundings performed at the site as presented in TVA [2005]. Current ground elevations were obtained from the KIF topographic map provided by TVA. Nearby borings were projected to the cross section to develop the thicknesses of the compressible native material along the cross section. This native material was subdivided into two groups based on the Standard Penetration Test (SPT) blow count and water content of the material. A description of the subsurface stratigraphy and the corresponding material properties are presented in the following section.

MATERIAL PARAMETERS

Information on the material parameters used in these analyses is obtained from MACTEC [2004], MACTEC [2005], and MACTEC [1995]. Material parameters used for the stability analyses are summarized in Table 1.

Gypsum (Phase I)

Samples of gypsum are not yet available from the KIF. Material from the Cumberland Fossil Plant is considered representative of the material that will be produced at the KIF once the scrubber is brought online.

Two types of gypsum (i.e., coarse and fine grained) were reported from Cumberland Fossil Plant by MACTEC [2004]. For design purposes, strength properties of the fine gypsum are used herein as they were lower and, thus, more conservative. Shear strength parameters were estimated based on a three-point CU triaxial test assuming failure occurs where the shear induced excess pore pressures are

zero. Based on these results, the effective stress shear strength parameters used in the analyses presented herein are an effective stress friction angle of 30 degrees and a zero cohesion intercept. An undrained shear strength ratio (S_u/σ_{vo}') of 1.5 was also selected. The unit weight of the gypsum was selected as 100 pcf.

Dry Fly Ash (Phase II)

The effective stress shear strength parameters used in the analyses presented herein are an effective stress friction angle of 30 degrees and a zero cohesion intercept, based on information presented in MACTEC [1995] and Geosyntec [2010]. An undrained shear strength ratio (S_u/σ_{vo}') of 0.8 was selected. A unit weight of 100 pcf was selected. It is noted that Phase II area will be receiving other coal combustion by-products including bottom ash and boiler slag; however, their generation quantities will be relatively low compared to the quantities of fly ash.

Soil Fill/Subgrade

On site material will be used to construct the initial soil berm around the gypsum pond and the subgrade fill. Standard Proctor tests were run on 17 samples of native material from depths ranging from 6 to 12.5 ft. The unit weight of the soil fill material was selected as 117 pcf, which is 95 percent of the average of the maximum dry unit weights resulting from the Standard Proctor tests. Effective stress properties for the soil berm and subgrade material are average values from three, three-point CU triaxial tests performed on remolded samples taken from depths ranging from 6 to 10 ft. Based on these results, the effective stress shear strength parameters used in the analyses presented herein are an effective stress friction angle of 30 degrees and a zero cohesion intercept.

Native Soil

The onsite native material is primarily classified as a medium stiff to stiff silty clay. The average blow count of the material onsite ranges from 6 to 20 blows per foot (bpf). Approximately one-half of the borings encountered a "soft" material, classified by Standard Penetration Test (SPT) N values less than or equal to 4 bpf. This soft material ranged in thickness from 0 to 20 ft along the cross sections selected for the stability analyses and occurred just above the bedrock material. For the analyses performed herein, drained and undrained shear strength parameters were selected for two layers of foundation material (i.e., N>4 and N≤4). Triaxial tests summarized in MACTEC [2006] and CPT soundings summarized in TVA [2005] were used to develop the short and long term shear strength of the native material.

- N>4: CU and unconsolidated undrained (UU) triaxial tests were performed on eight samples obtained from depths ranging from 13 to 41 ft below ground surface. This triaxial data in combination with data from ten Cone Penetration Test (CPT) soundings performed across the site to depths of 42 ft were used to estimate native soil undrained shear strength. The undrained shear strength can be estimated from the measured tip resistance according to the following equation developed by Schmertmann [1978]:

$$S_u = \frac{q_c - \sigma_{vo}}{N_k}$$

where N_k is a normalizing factor that ranges from 12 to 19 and is related to the plasticity index of the in situ material. For the native soil at the KIF gypsum disposal facility, an N_k factor of 19 was chosen to calibrate the calculated CPT undrained shear strength data to the undrained shear strength developed from the triaxial test data.

The undrained shear strength data were plotted versus effective confining pressure to develop an undrained shear strength profile for the native material (i.e., a best fit linear trend line through the data as shown in Figure 9) resulting in the following equation:

S_u = 1,792 psf + 0.27*Confining Pressure (psf)

Conservatively assuming that the native soil is saturated, and a unit weight of 120 pcf, an undrained strength profile with depth can be estimated using the following equation:

S_u = 1,792 psf + 15.6*depth

where depth is measured in feet below the pre-construction ground surface (i.e. at the elevation of the top of the native material).

For the drained analyses, an average effective stress friction angle of 34 degrees was used based on triaxial testing results.

- N≤4: No triaxial tests were performed on native material with SPT blow counts of less than or equal to 4 bpf. Four of the ten CPT soundings performed at the site encountered the soft native material. The average undrained shear strength of this material was developed from these CPT soundings and is 800 psf for the analyses performed herein. The improved undrained shear strength of the "soft" native material was not developed.

  For the drained analyses, an effective stress friction angle for the soft native material was estimated based on the plasticity index (PI) using the following relationship [Mitchell, 1976]:

$$\sin\phi'_{cv} \approx 0.8 - 0.094\ln(PI)$$

  Considering an average PI of the soft native material to be 43, an effective stress friction angle of 26.5 degrees was calculated. A friction angle of 25 degrees was used for the analyses described herein.

Bedrock

Due to the anticipated high shear strength of the bedrock, the top of bedrock elevation is considered the lower limit for the potential critical slip surface; reasonable cohesion, friction angle, and unit weight values were selected as required by the computer simulation.

RESULTS

Table 2 summarizes the results of the static slope stability analyses for Cross Sections 1-5 under both drained and undrained conditions. Table 3 summarizes the results of the seismic slope stability analyses performed for the KIF gypsum disposal facility. Seismic slope stability analyses were performed for the final closure condition for Phase I and Phase II. Strength gain of the foundation soils due to waste loading was conservatively not considered. The calculated yield accelerations were larger than the peak ground acceleration at the site, thus, permanent displacements are not anticipated. The associated output files and figures from SLIDE are presented in Attachments A and B, for static and seismic analyses, respectively.

SUMMARY AND CONCLUSIONS

The stability of the KIF disposal facility (Phase I and Phase II) was evaluated with respect to static and seismic foundation stability. The most critical cross sections with respect to foundation stability were analyzed. Results indicate that the minimum static stability factor of safety for a potential slip surface through the gypsum/fly ash and foundation soils is 1.73, which is greater than the target factor of safety of 1.5.

Results indicate that the minimum yield acceleration for slip surfaces through the waste and the foundation soils is 0.30 g, which is greater than the site peak ground horizontal acceleration. Based on this analysis, no permanent displacements are anticipated during the design earthquake event.

REFERENCES

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MACTEC, "*Report of Additional Geotechnical Exploration: Proposed Gypsum Disposal Area at Kingston Fossil Plant, Kingston, Tennessee,*" MACTEC Project 3043051064.01, prepared for TVA, February 2006.

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Spencer, E., *"A Method of Analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces"*, Geotechnique, Vol. 17, No. 1, 1967, pp. 11-26.

Tennessee Division of Solid Waste Management, a Division of the Tennessee Department of Environment and Conservation (TDEC), "*Technical Guidance Document: Earthquake Evaluation Guidance Policy*," 1993, 41 p.

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USEPA. RCRA Subtitle D (258) *Seismic Design Guidance for Municipal Solid Waste Landfill Facilities*. Document Number EPA600-R-95-051, 1995.

TABLES

Table 1. Summary of Material Properties.

Material	Unit Weight (pcf)	Effective Stress		Undrained Strength, S_u (psf)
		Cohesion (psf)	Friction Angle (degrees)	
Gypsum (Phase I)	100	0	30	S_u/σ_{vo}'=1.5
Dry Fly Ash (Phase II)	100	0	30	S_u/σ_{vo}'=0.8
Soil Fill	117	0	30	-
Phase I GCL	1	φ_{peak} = 10°, φ_{res} = 6°		
Phase II Geosynthetic	1	back calculated		
Native Soil (N>4)	120	0	34	1792+15.6 * depth
Native Soil (N<4)	105	0	25	800
Bedrock	155	10,000	30	0

Note:

The influence of the liner shear strengths on the calculated FSs for the foundation stability analyses is insignificant. Failure through the liner system is evaluated in the calculation package titled "*Liner Waste Block Slope Stability Analyses*".

Table 2. Summary of Static Foundation Stability Analyses.

Cross Section	Drained/ Undrained	Factor of Safety	Target Factor of Safety	Is FS OK?
1	Undrained	3.39	1.5	Yes
1	Drained	1.75	1.5	Yes
2	Undrained	4.08	1.5	Yes
2	Drained	1.73	1.5	Yes
3	Undrained	3.01	1.5	Yes
3	Drained	1.78	1.5	Yes
4	Undrained	3.03	1.5	Yes
4	Drained	1.74	1.5	Yes
5	Undrained	3.19	1.5	Yes
5	Drained	1.73	1.5	Yes

Table 3. Summary of Seismic Foundation Stability Analyses.

Cross Section	Drained/ Undrained	Yield Acceleration (g)	Seismic Deformation Analysis Needed?
1	Undrained	0.34	No
2	Undrained	0.43	No
3	Undrained	0.41	No
4	Undrained	0.28	No
5	Undrained	0.21	No

FIGURES

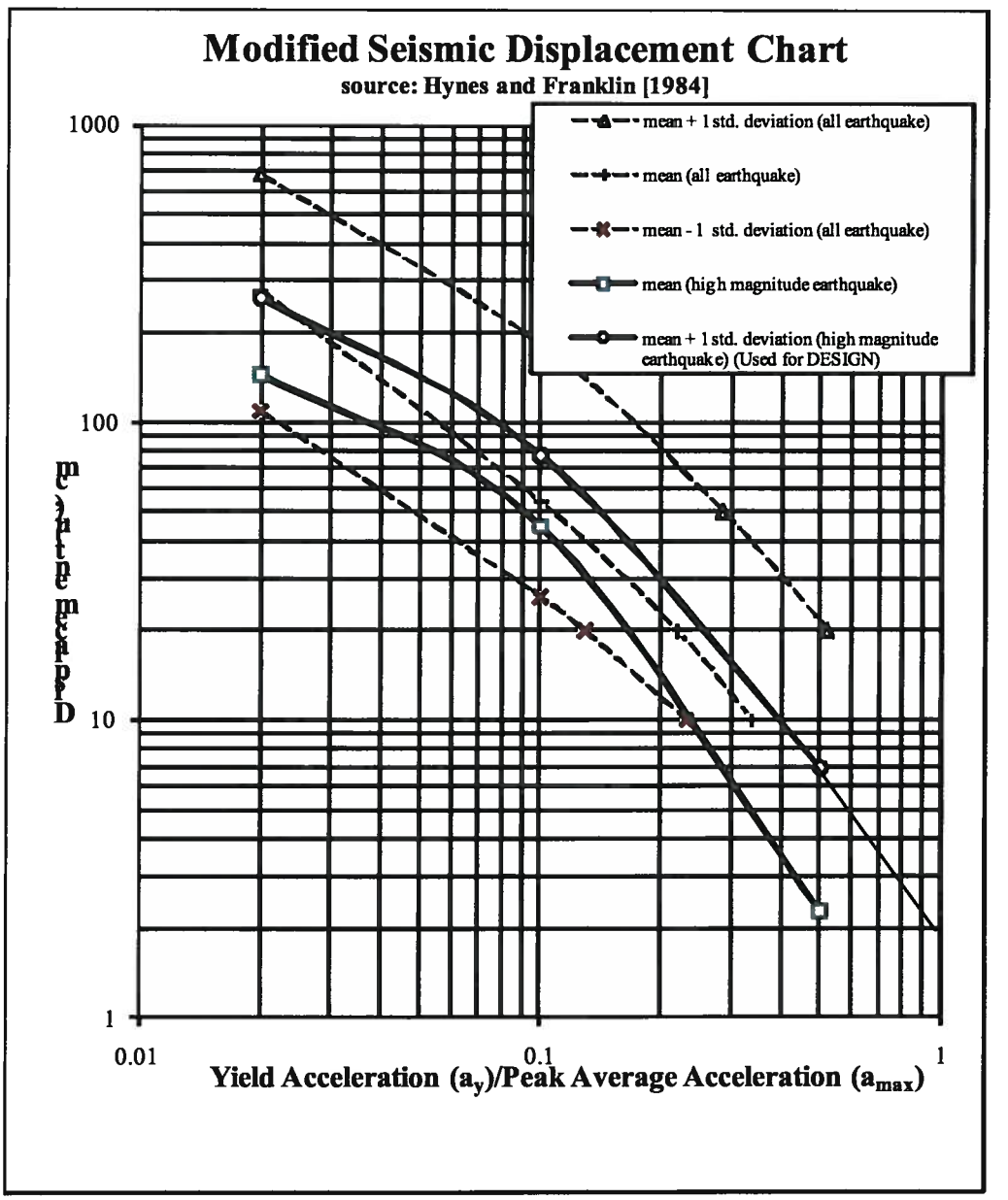
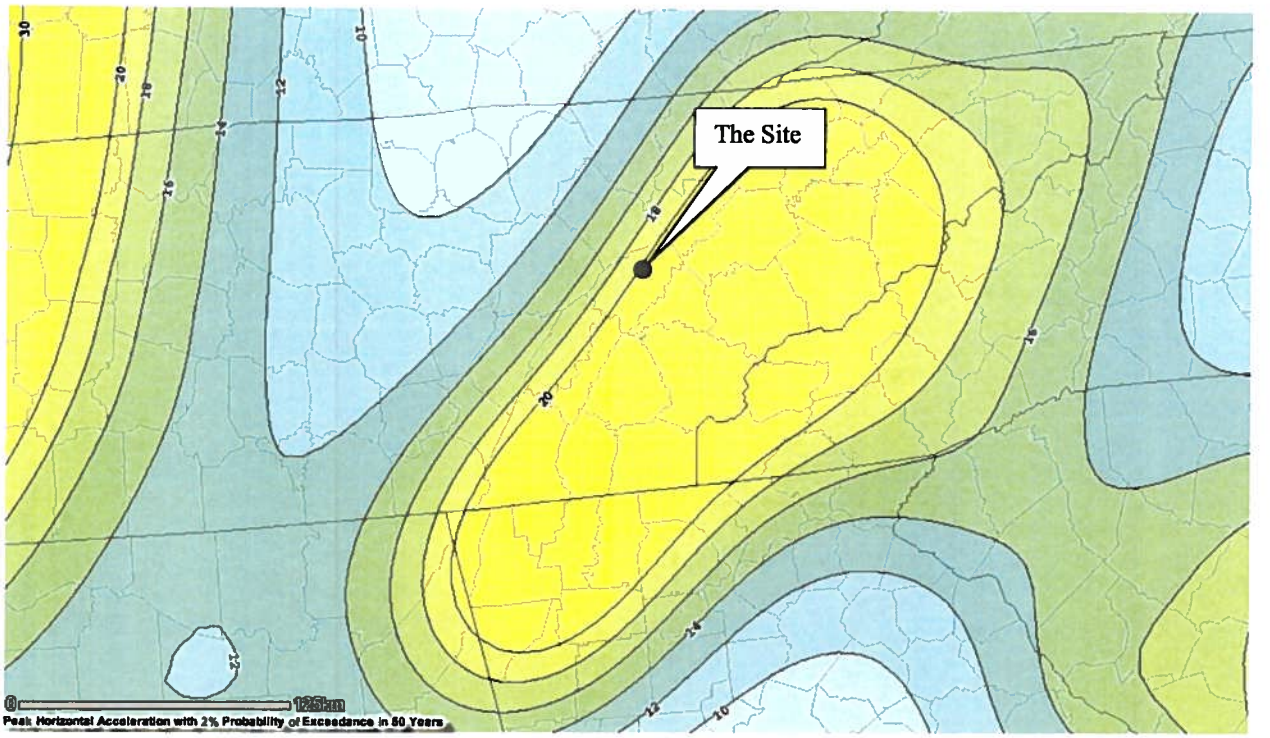


Figure 1. Selection of Calculated Displacement



N 35.89472°
W 84.50351°

Figure 2. The Peak Ground Acceleration (PGA) with a 2 Percent Probability of Exceedence in 50 Years (or 10 Percent Probability of Exceedence in 250 Years) [USGS, 2008]

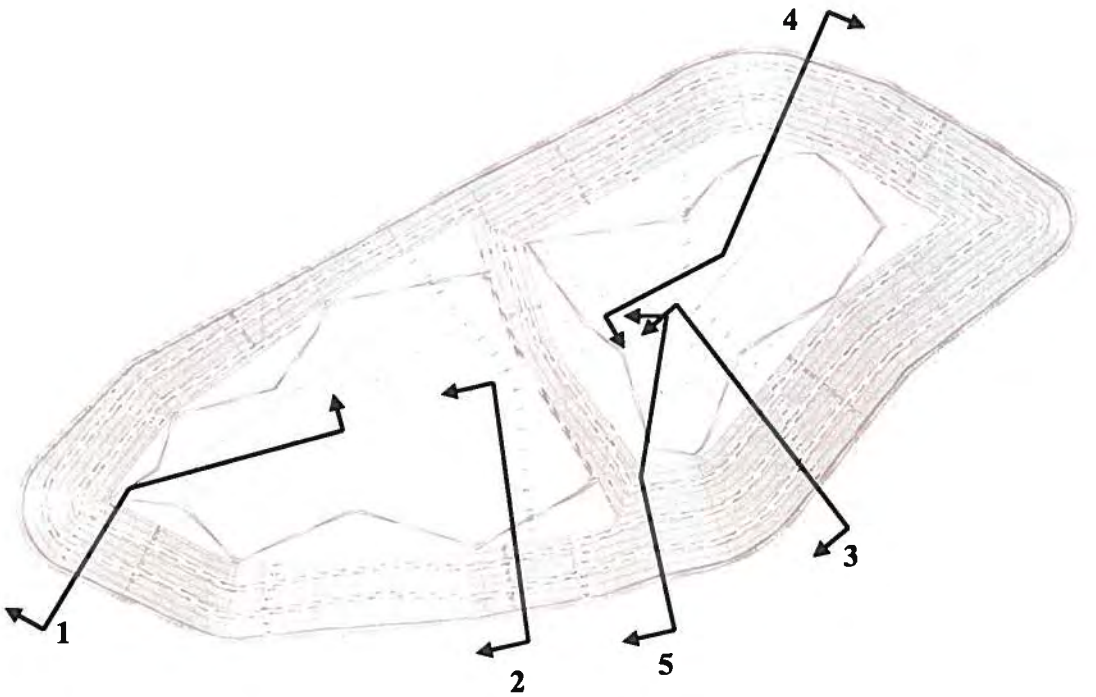


Figure 3. Location of the Analyzed Cross Sections on Final Cover Grading Plan

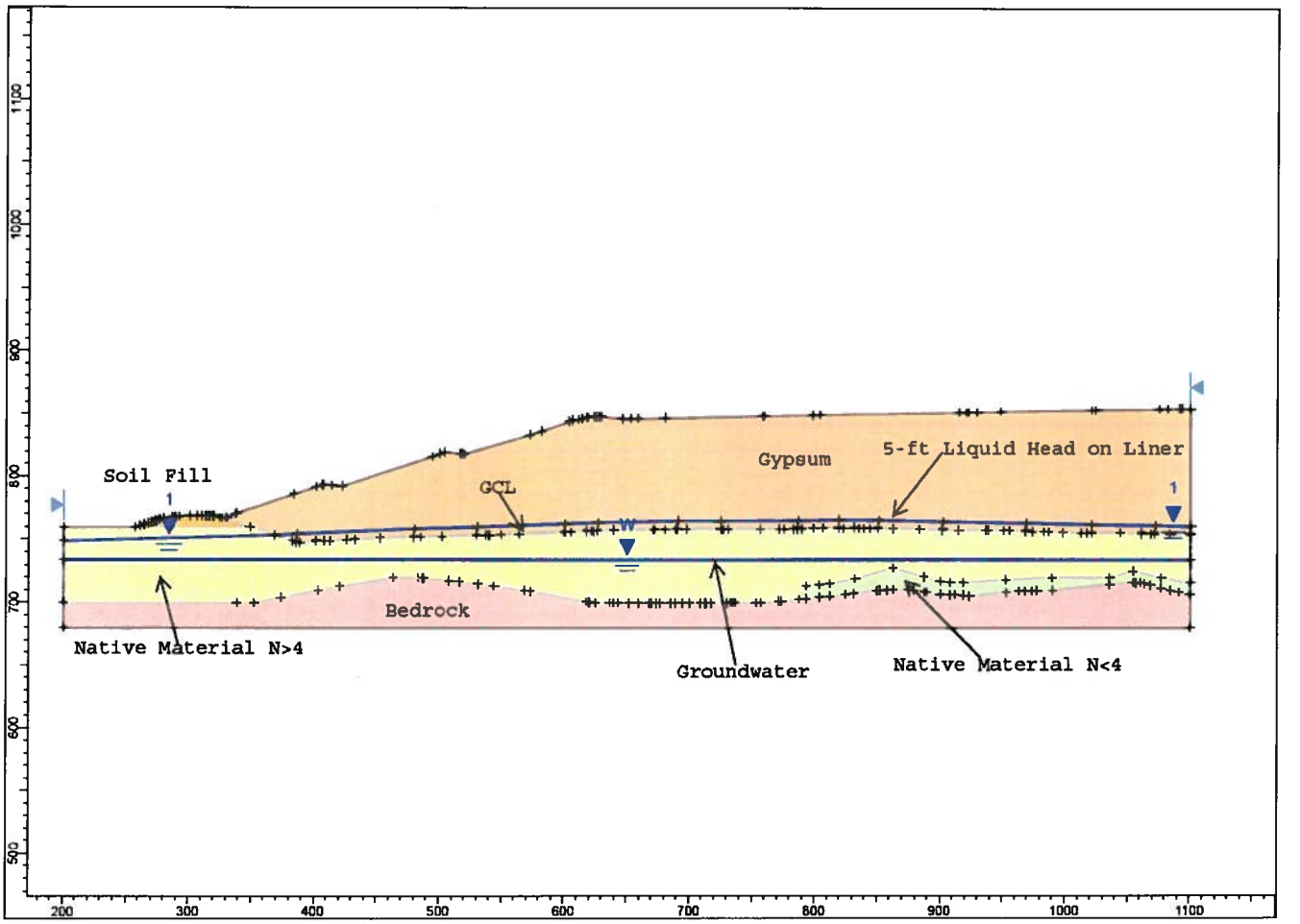


Figure 4. Geometry of Cross Section 1

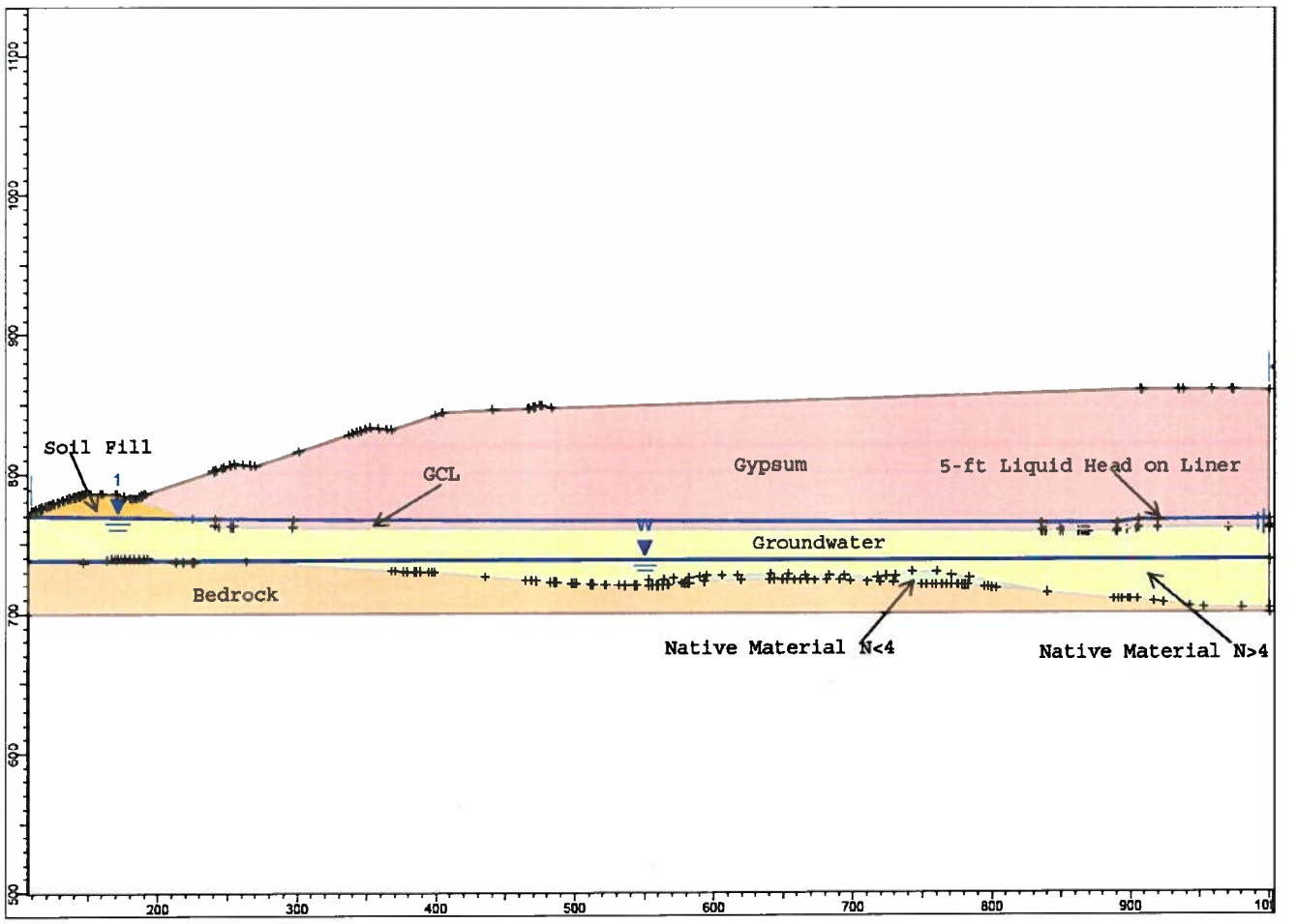


Figure 5. Geometry of Cross Section 2

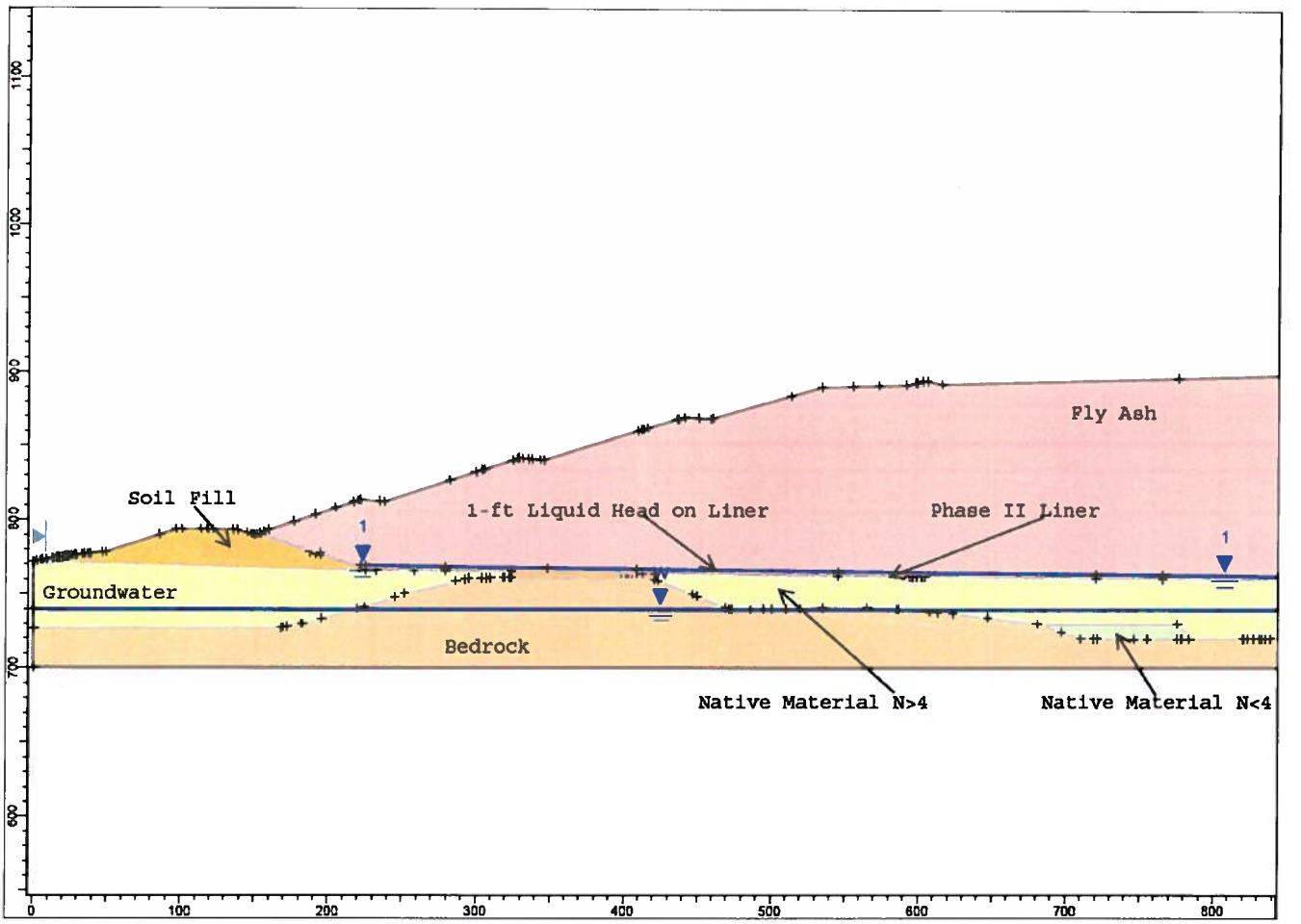


Figure 6. Geometry of Cross Section 3

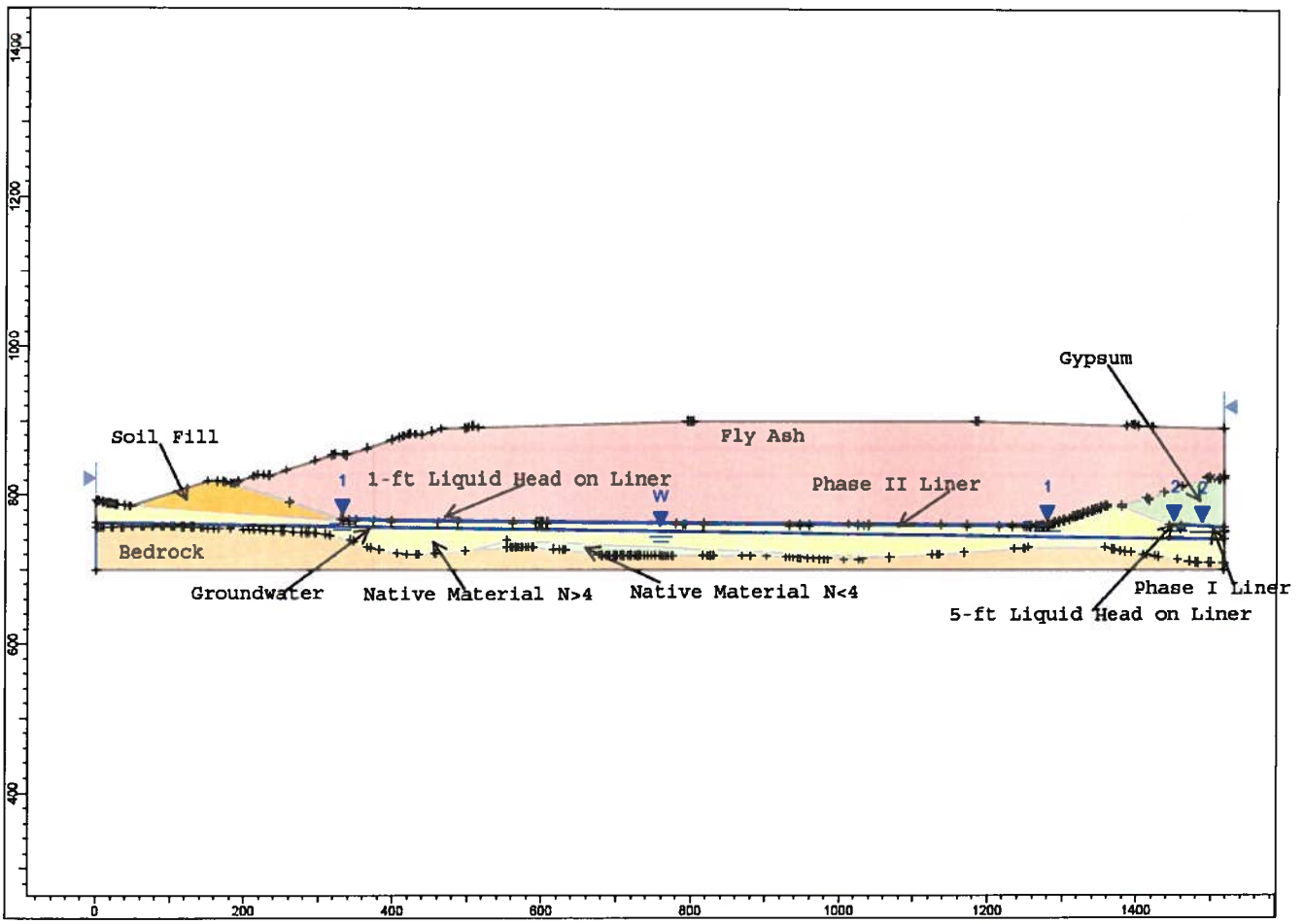


Figure 7. Geometry of Cross Section 4

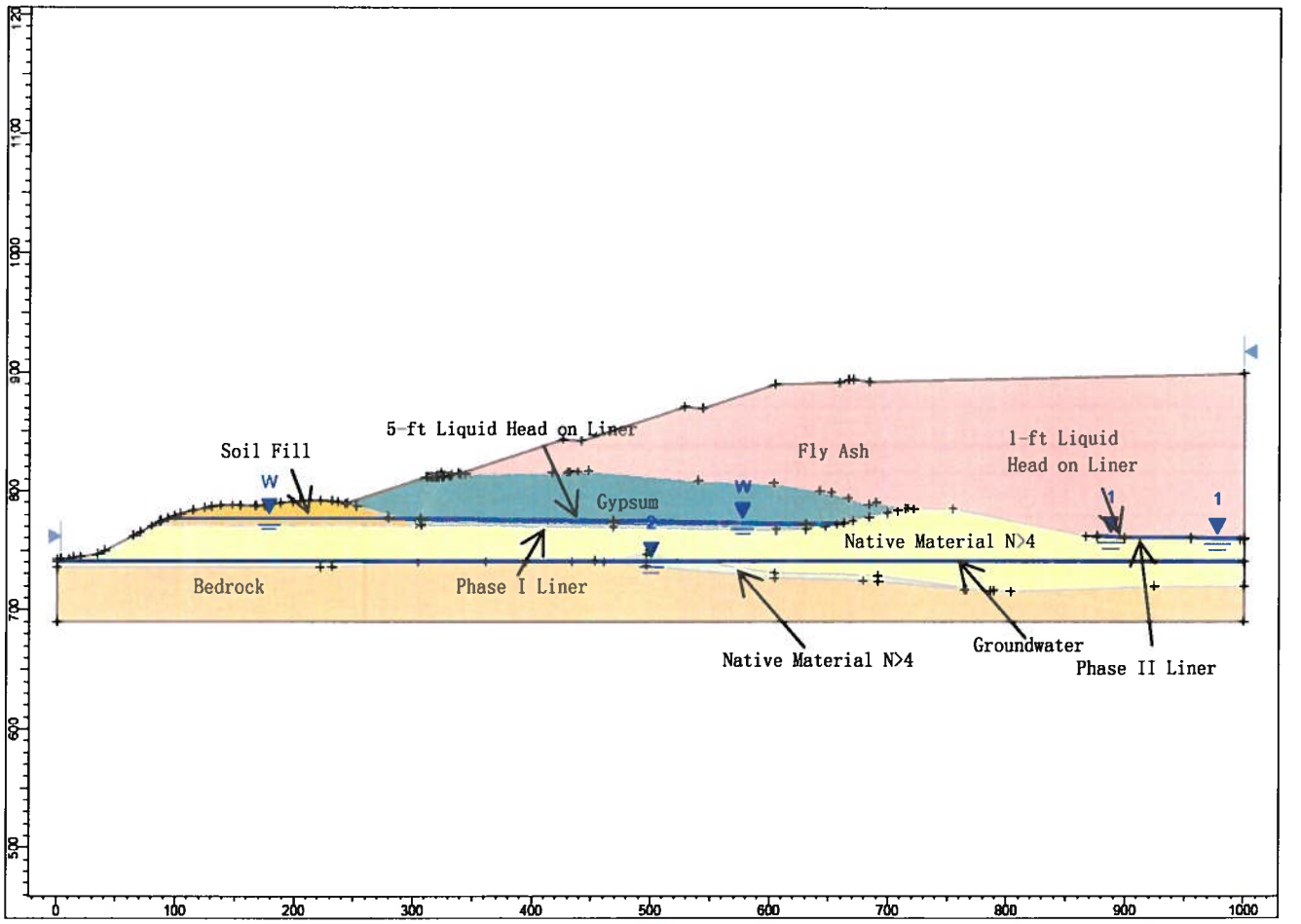
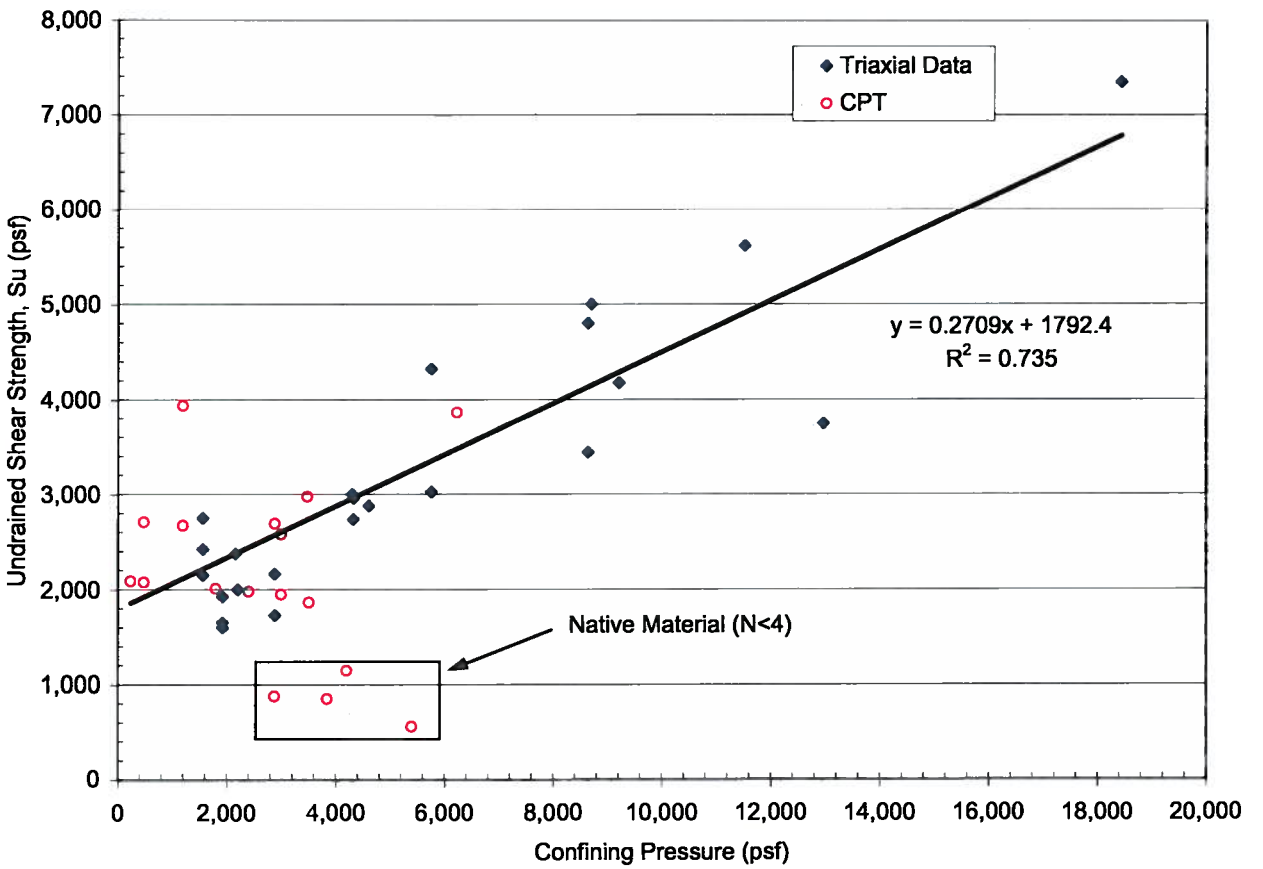


Figure 8. Geometry of Cross Section 5



Notes:

1-Undrained shear strengths were derived from the CPT soundings based on the method developed by Schmertmann [1978]. An N_k factor of 19 was chosen to calibrate the CPT data to the triaxial data.

2-Undrained shear strengths based on CPT soundings in the soft native material (N<4) are indicated. An average undrained shear strength of 800 psf was chosen based on these data.

Figure 9. Determination of Undrained Shear Strength Profile for Foundation Material.

ATTACHMENT A

SLIDE OUTPUT
CROSS SECTIONS 1-5 – STATIC

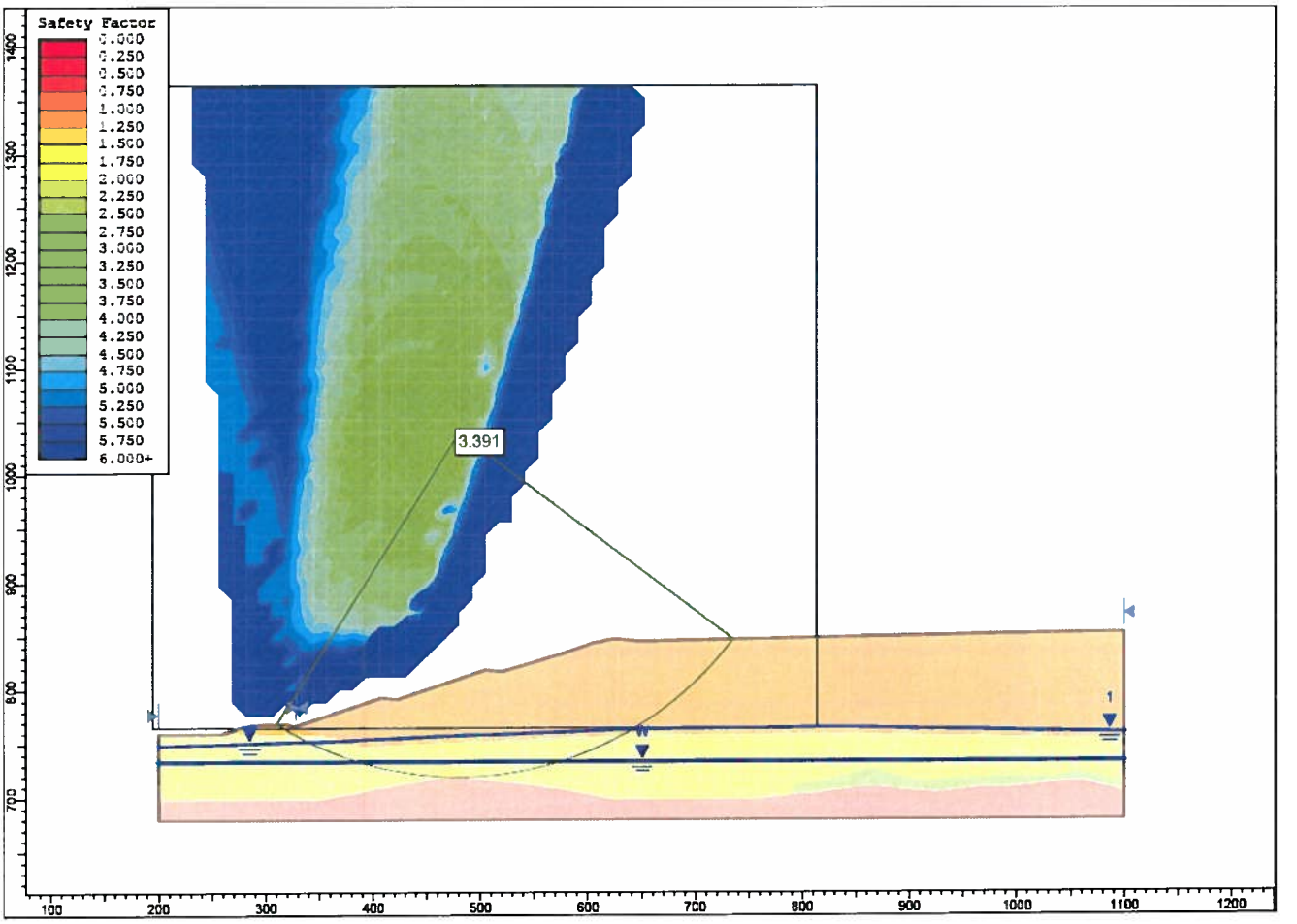


Figure A-1. Static Slope Stability Analysis Result for Section 1 (undrained condition)

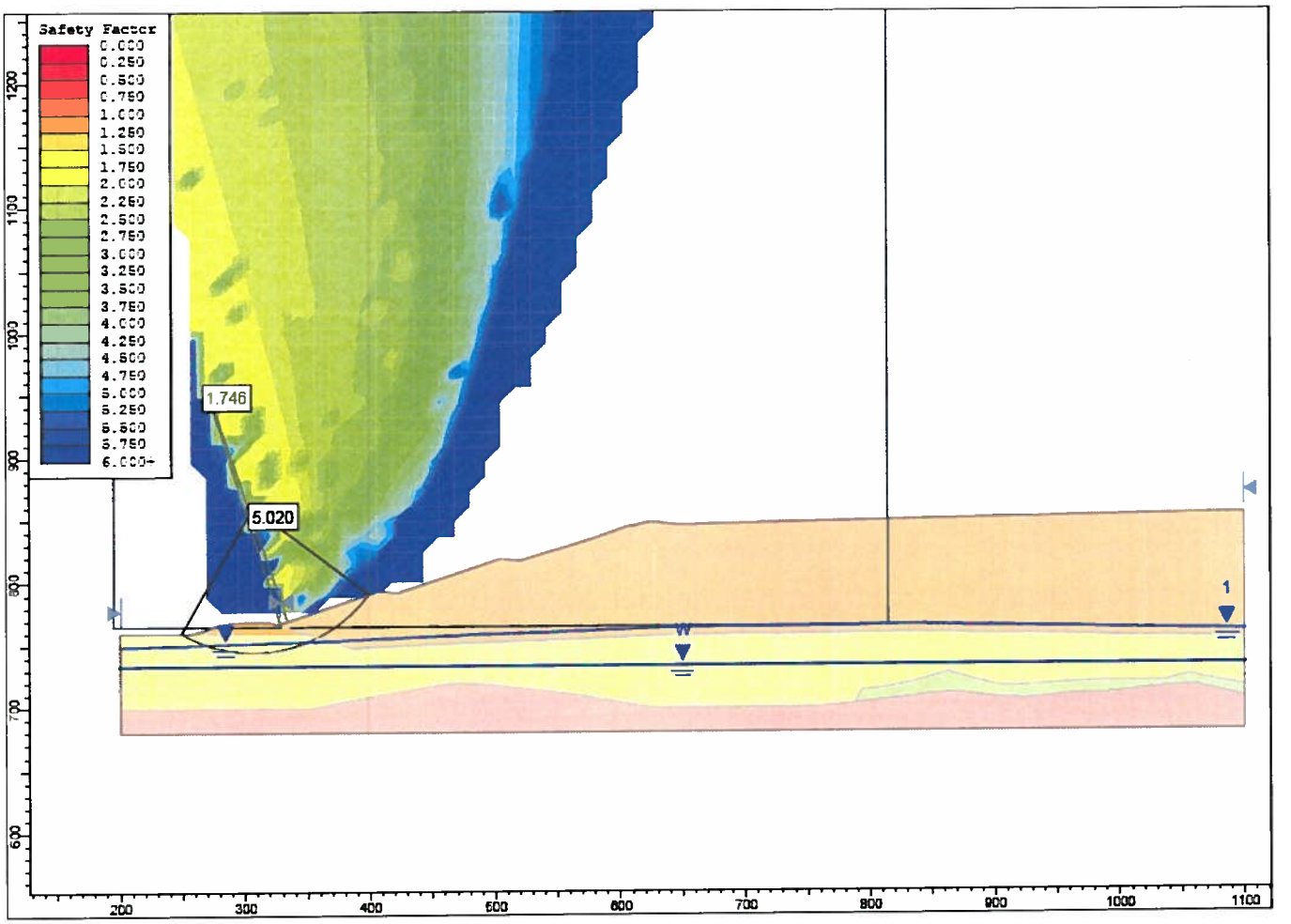


Figure A-2. Static Slope Stability Analysis Result for Section 1 (drained condition)

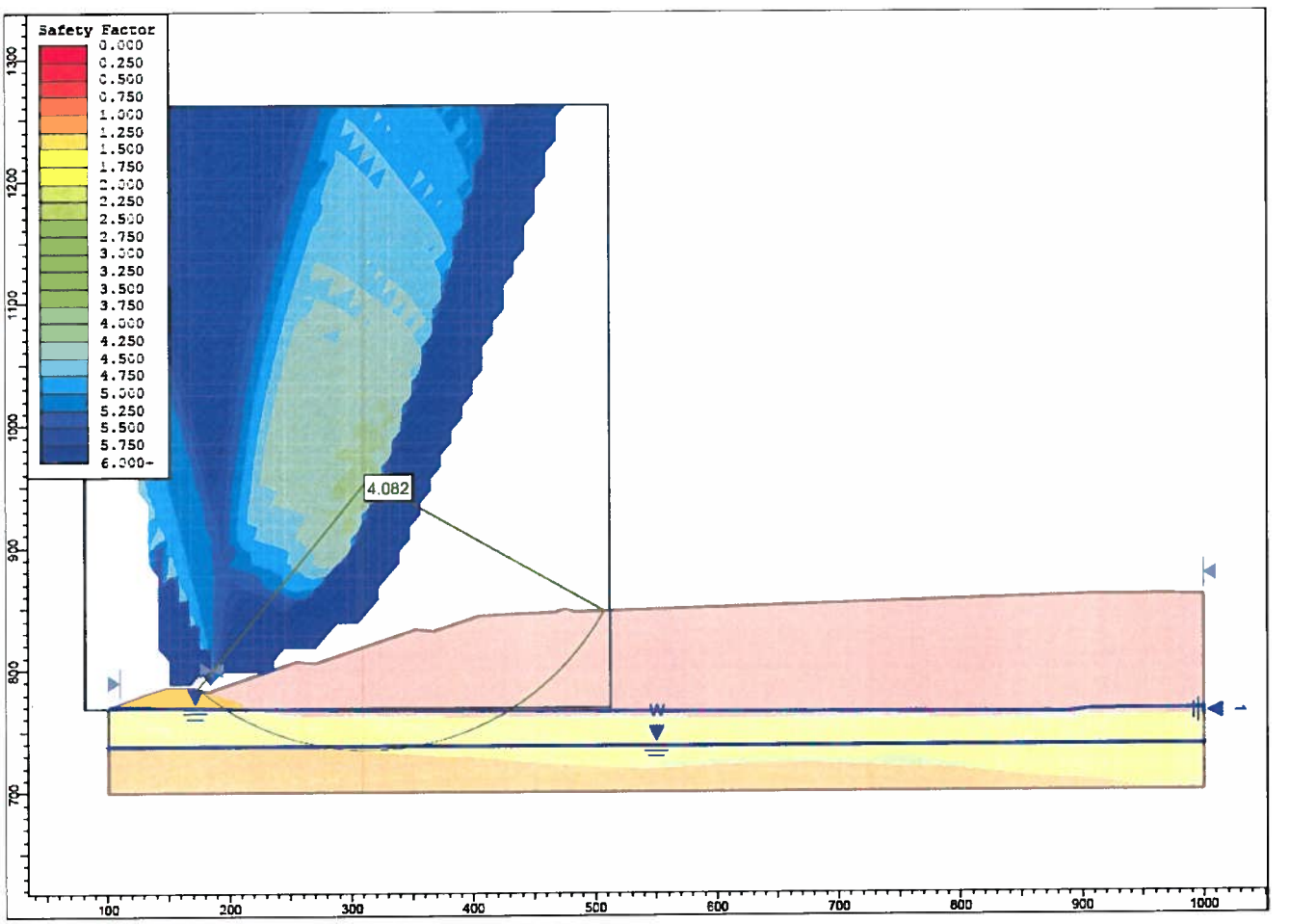


Figure A-3. Static Slope Stability Analysis Result for Section 2 (undrained condition)

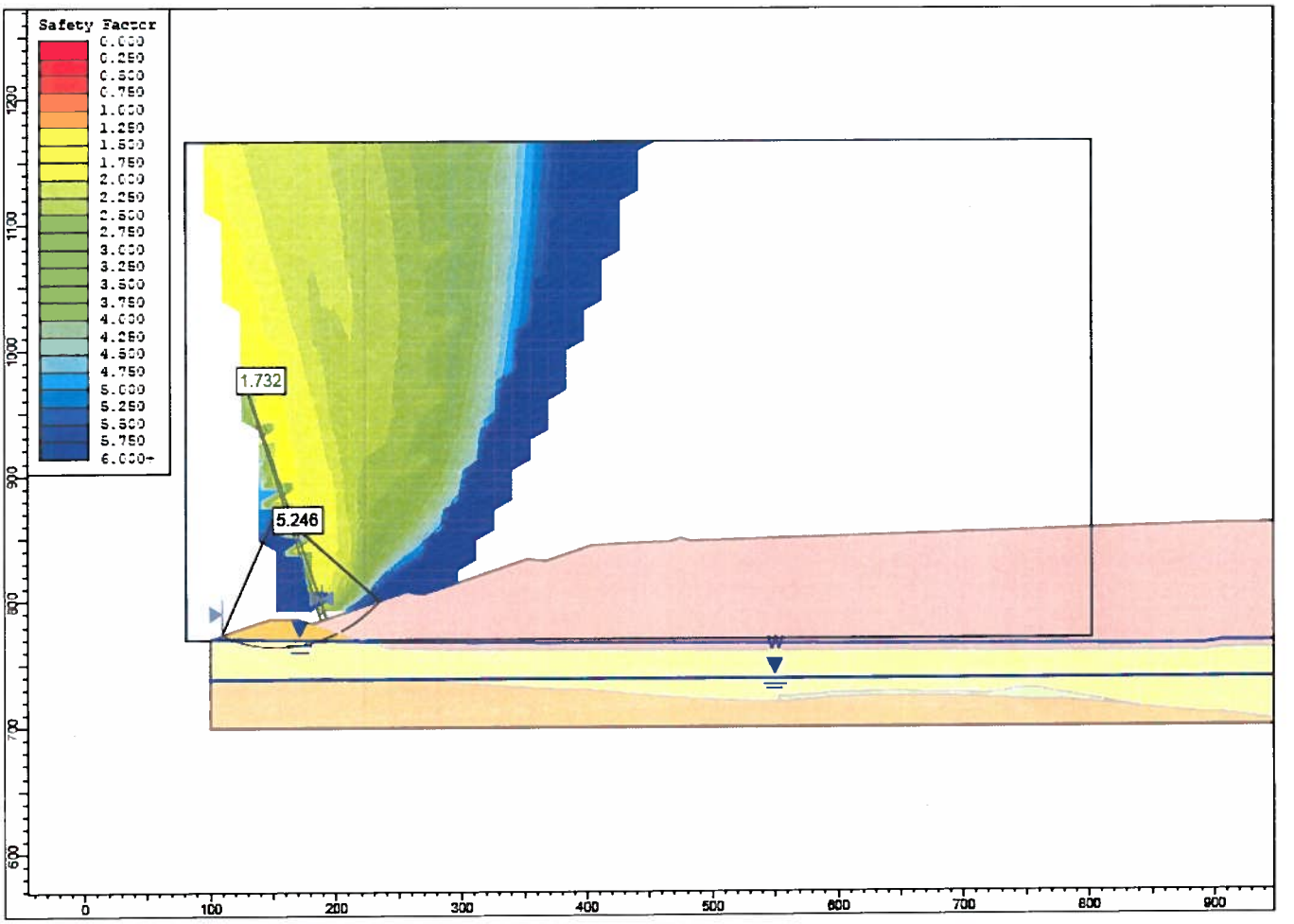


Figure A-4. Static Slope Stability Analysis Result for Section 2 (drained condition)

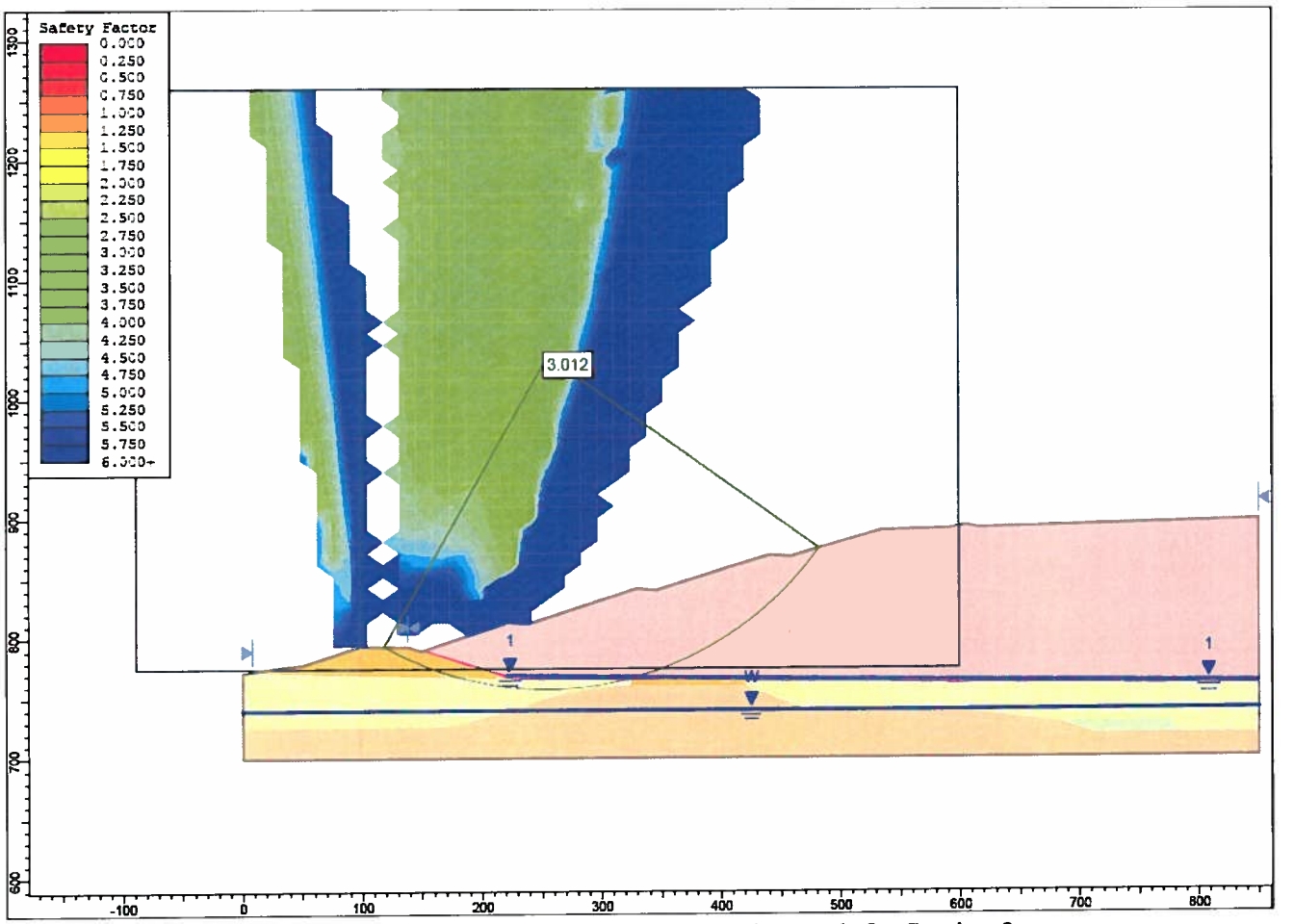


Figure A-5. Static Slope Stability Analysis Result for Section 3 (undrained condition)

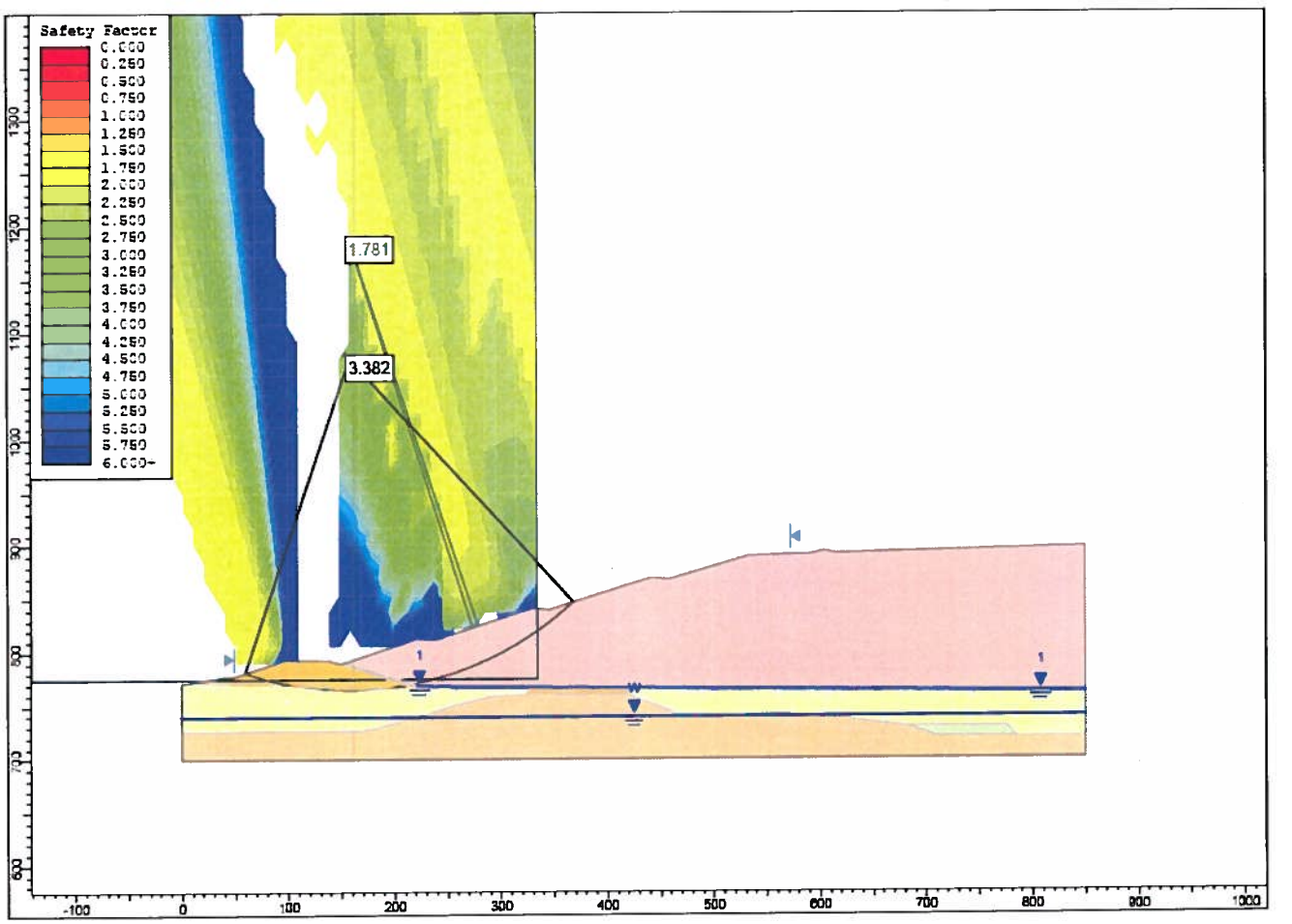


Figure A-6. Static Slope Stability Analysis Result for Section 3
(drained condition)

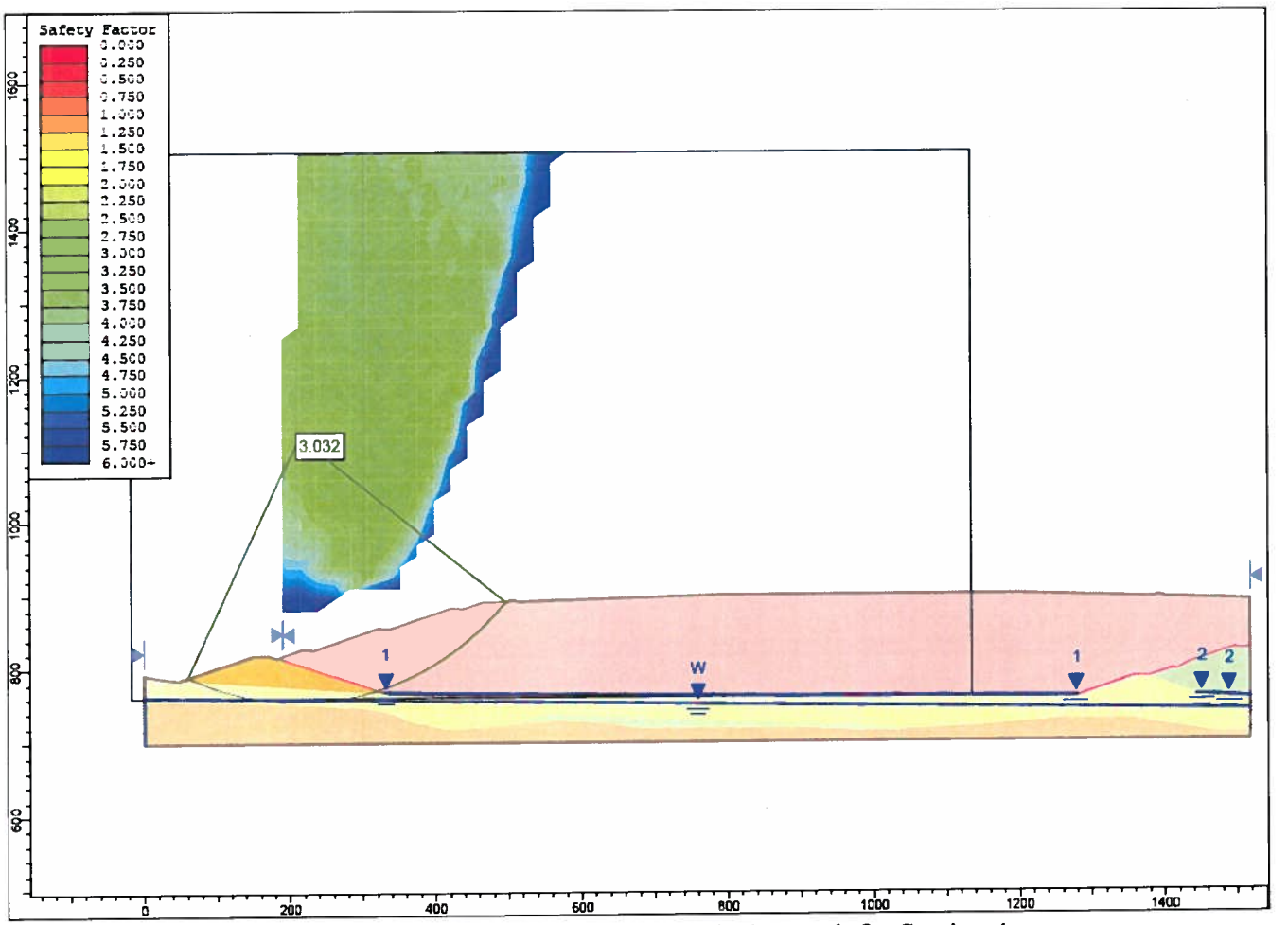


Figure A-7. Static Slope Stability Analysis Result for Section 4 (undrained condition)

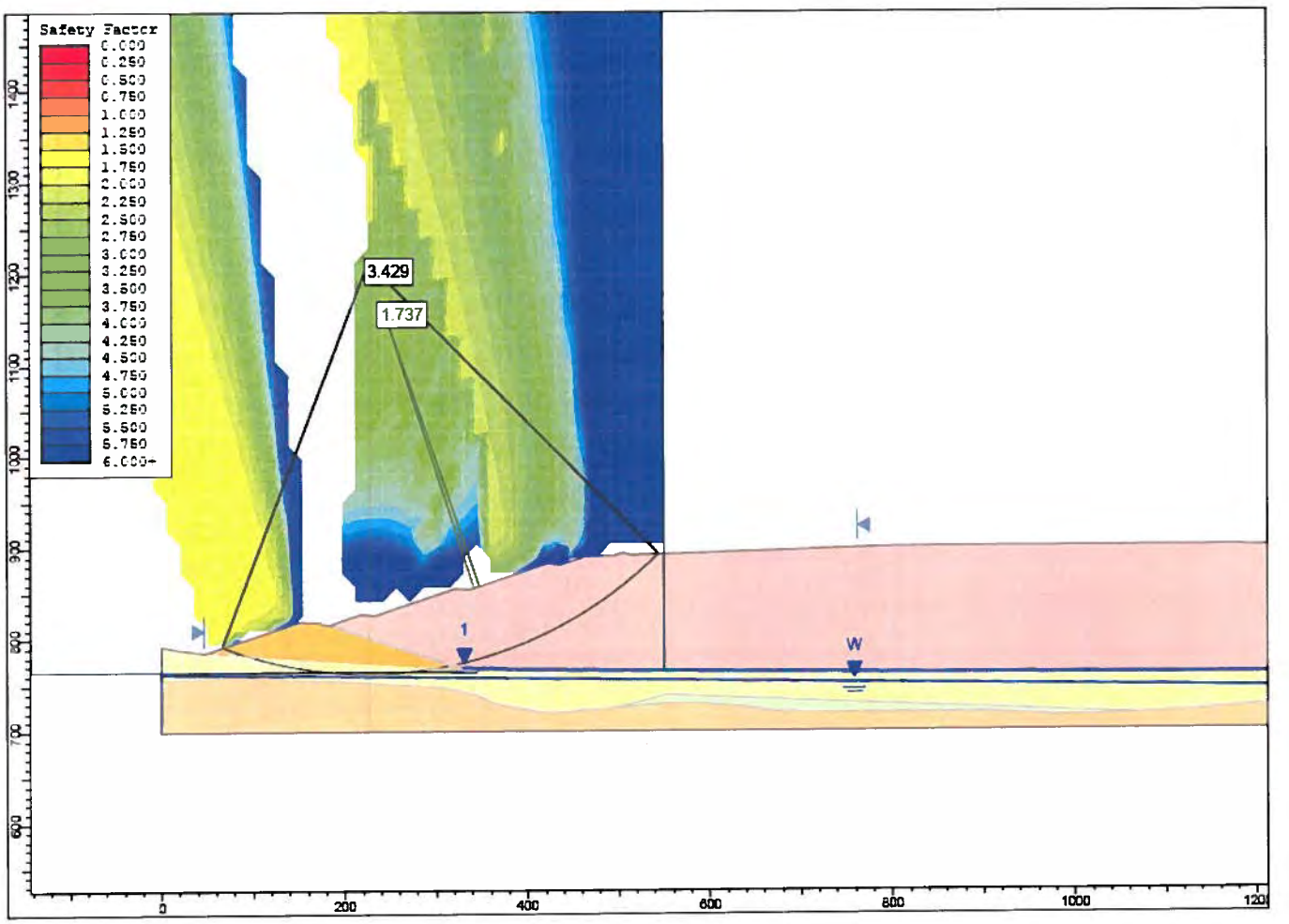


Figure A-8. Static Slope Stability Analysis Result for Section 4
(drained condition)

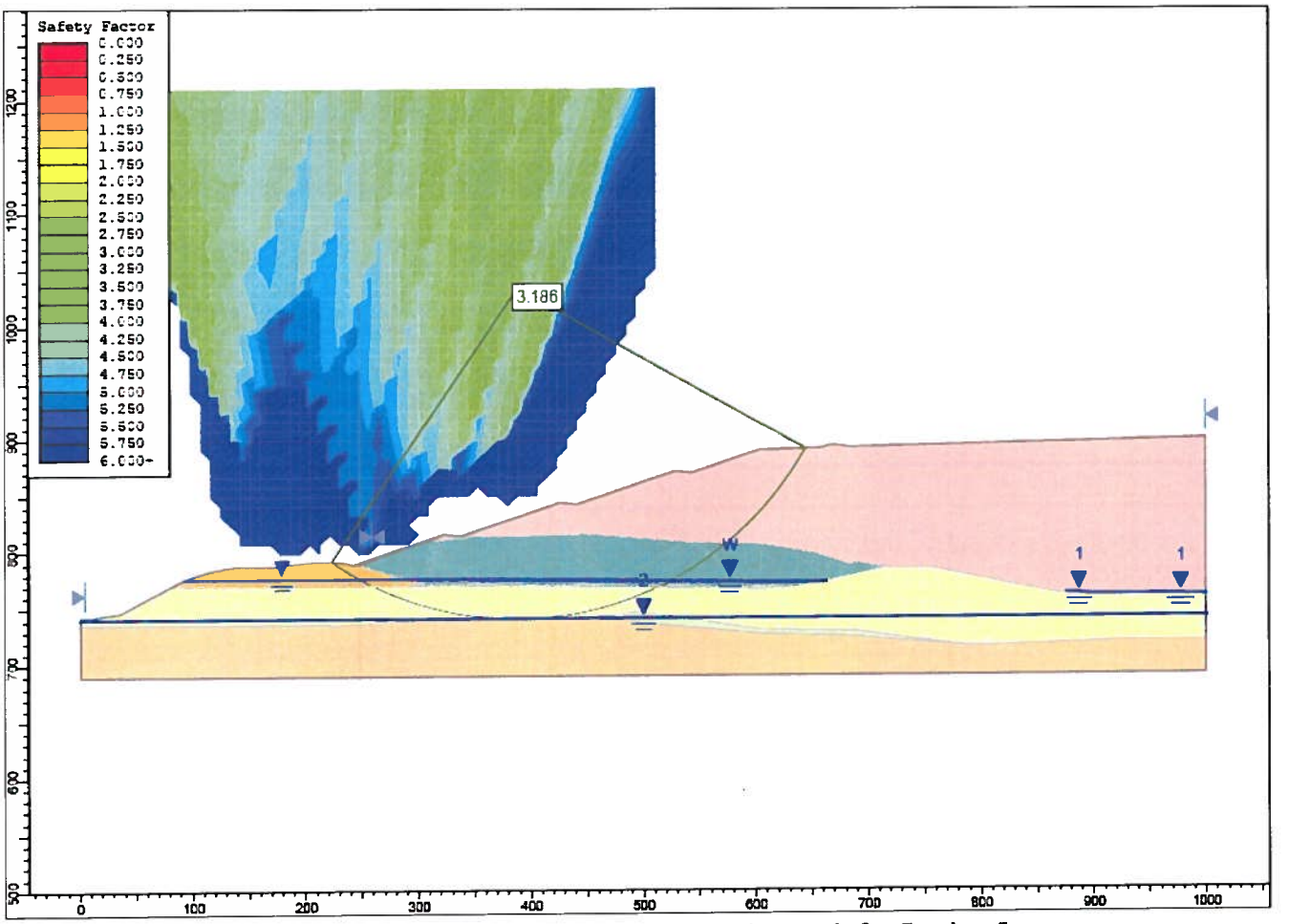


Figure A-9. Static Slope Stability Analysis Result for Section 5
(undrained condition)

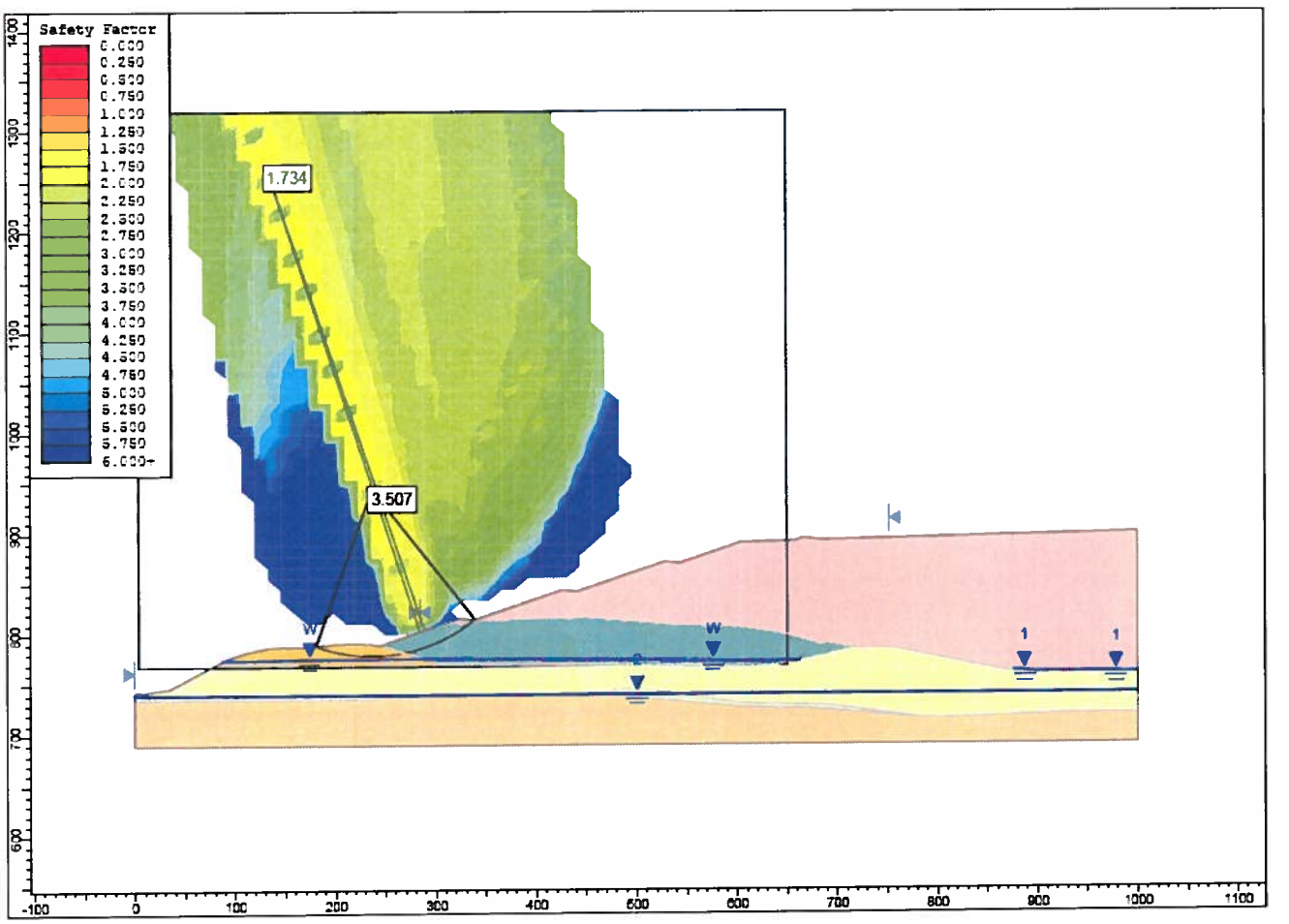


Figure A-10. Static Slope Stability Analysis Result for Section 5
(drained condition)

ATTACHMENT B

SLIDE OUTPUT
CROSS SECTIONS 1-5 – SEISMIC

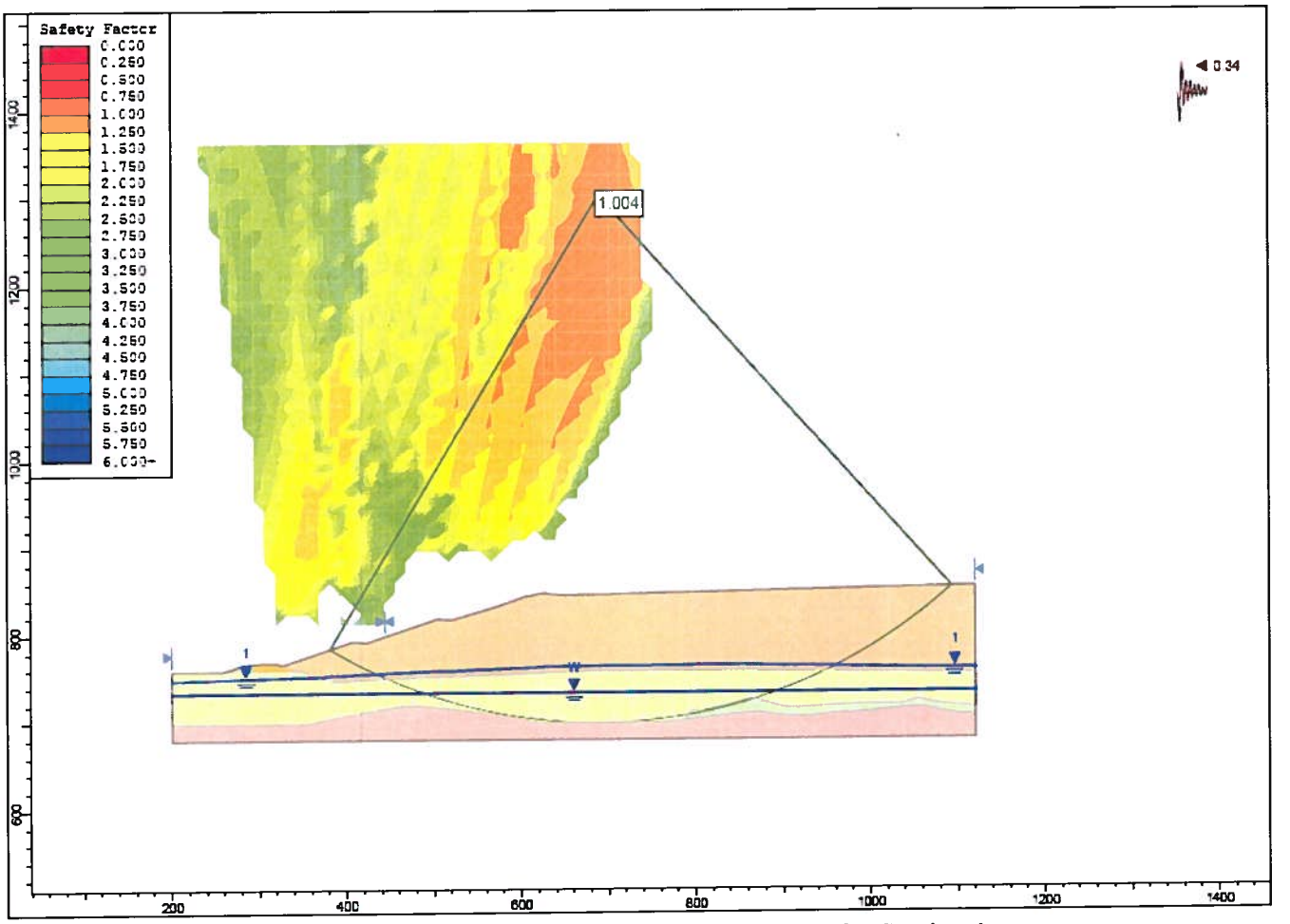


Figure B-1. Calculation of Yield Acceleration for Section 1

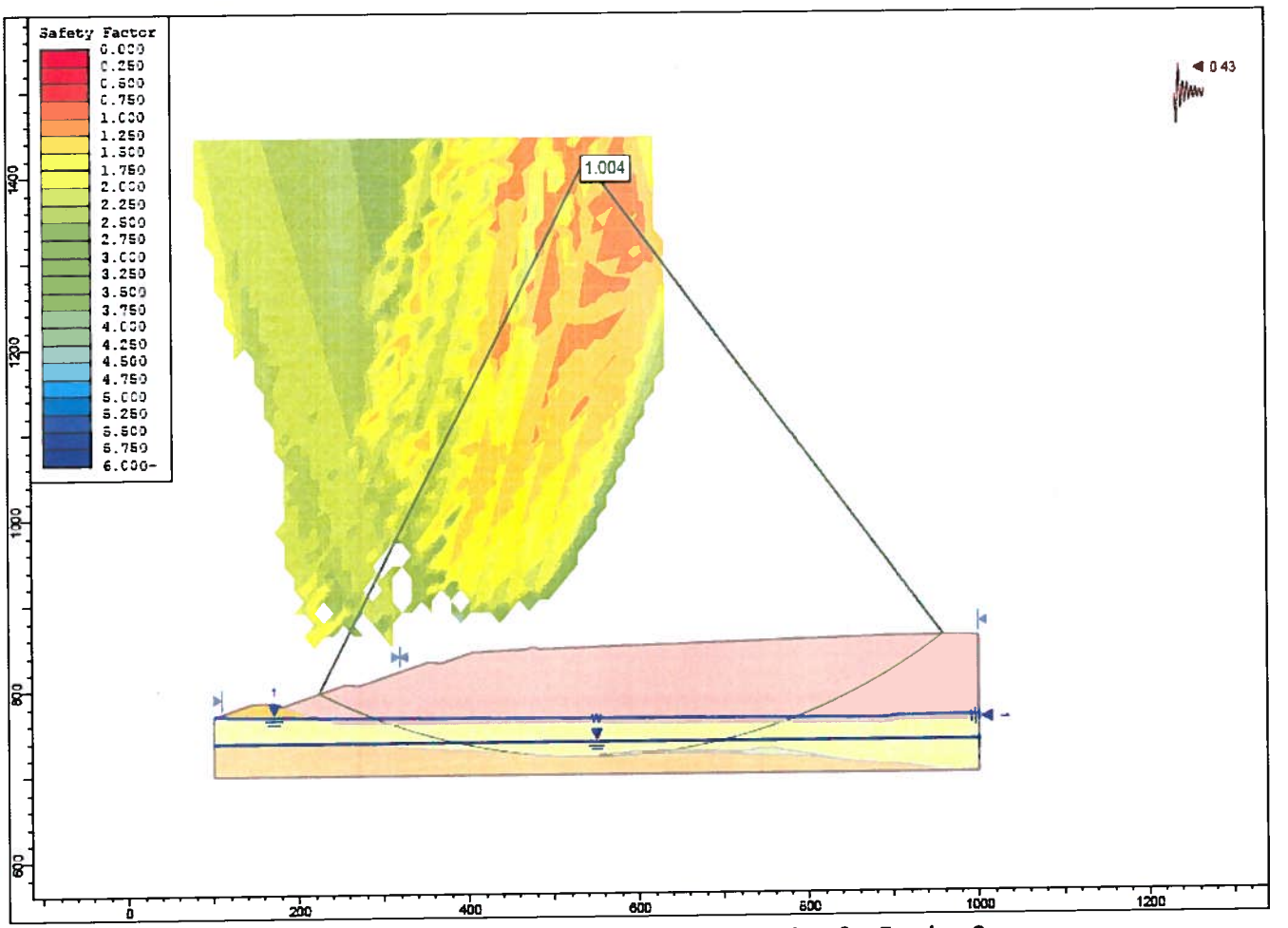


Figure B-2. Calculation of Yield Acceleration for Section 2

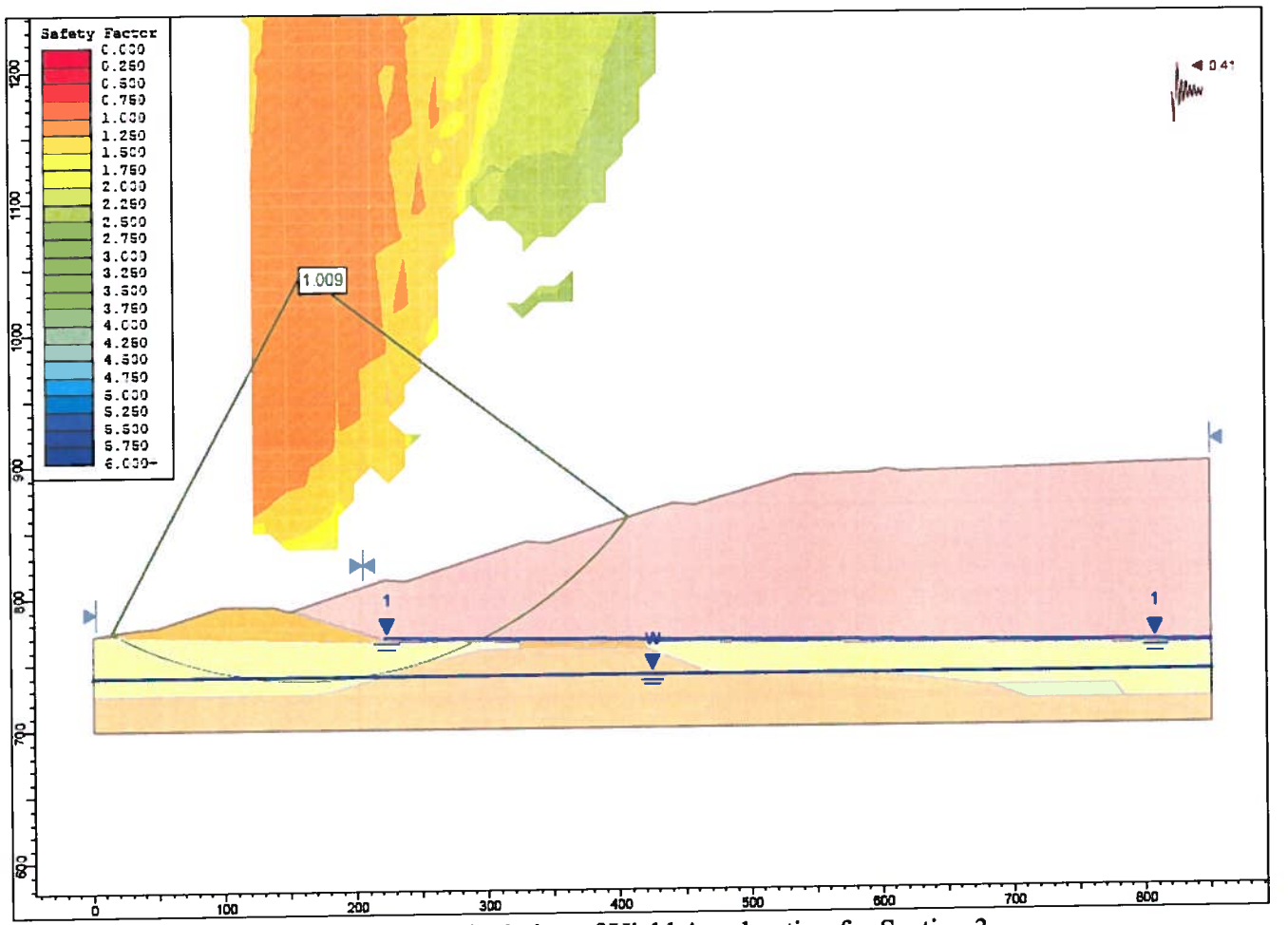


Figure B-3. Calculation of Yield Acceleration for Section 3

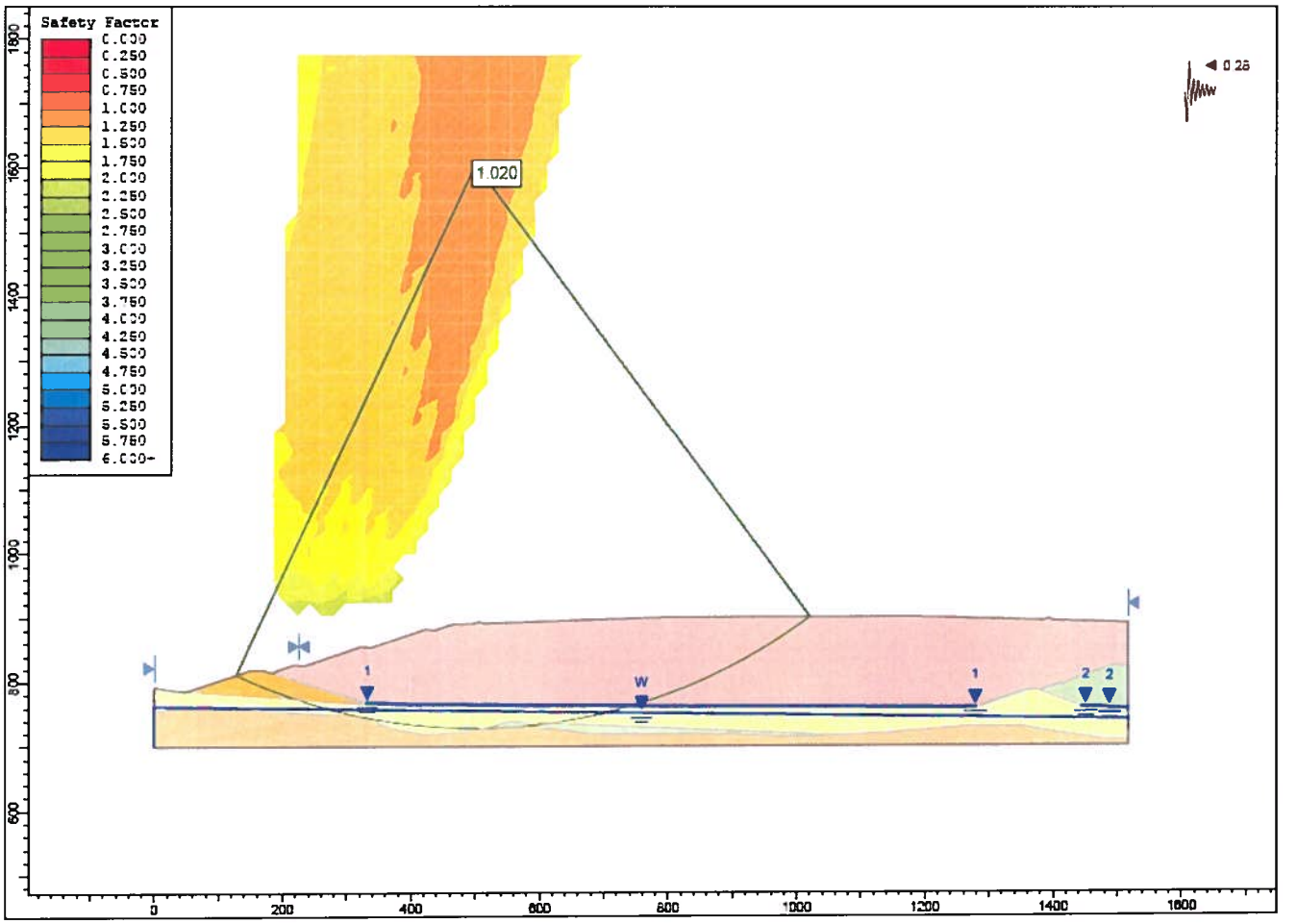


Figure B-4. Calculation of Yield Acceleration for Section 4

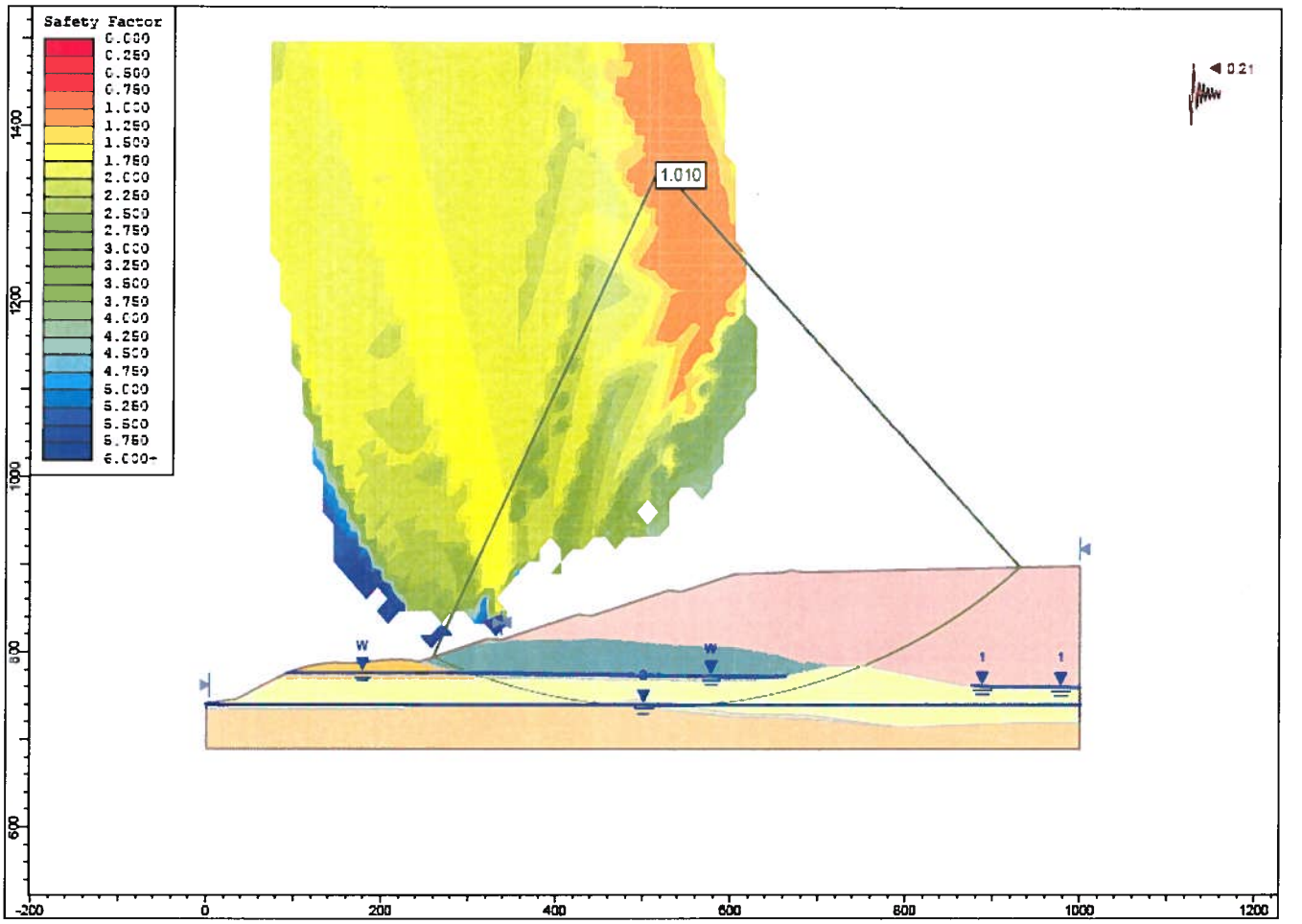


Figure B-5. Calculation of Yield Acceleration for Section 5

US EPA ARCHIVE DOCUMENT

APPENDIX A

Document 22

Dewberry Memorandum, "Qualitative Assessment. Liquefaction Potential, TVA Fossil Plant CCR Impoundments, Dewberry Project No. 50047151," May 25, 2012

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.

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APPENDIX B

Document 23

Dam Inspection Check List Form: Ash Pond C

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Site Name:	**Kingston Steam Plant**	**Date:**	**21 September 2011**
Unit Name:	**Ash Pond and Stilling Pond**	**Operator's Name:**	**Tennessee Valley Authority (TVA)**
Unit I.D.:	**Ash Pond and Stilling Pond**	**Hazard Potential Classification:**	**High** ☐ **Significant** ☒ **Low** ☐
	Inspector's Name:	Stanley W., Notestine, P.E. and Joseph P. Klein, III, 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)?	754.7		19. Major erosion or slope deterioration?		X
3. Decant inlet elevation (operator records)?	750		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)?	765		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)?	N/A		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?	N/A	
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
2.	Pool elevation based on "Diffuser Replacement Project Details – Siphon Spillways Work Plan 6" dated May 3, 2010
3.	Decant elevation is siphon inlet design elevation shown on Drawing referenced in Issue #2. Siphon system in use at time of site visit. Original Stilling Pond spillway system of six 48-inch diameter reinforced concrete pipes remains in place but is currently out of service.
7.	The Ash Pond is adjacent to the Dredge Cell that experienced a significant embankment failure is December 2008. Reconstruction of the Dredge Cell designed for use as a dry ash storage facility includes construction along Dike D between the Dredge Cell and the Ash Pond and interior grading of the Ash Pond.

8.	Documentation of original dike foundation preparation not available. Borings conducted as part of 2009 Geotechnical Exploration and Slope Stability for Dike C do not indicate the presence of vegetation or other organic matter in the uppermost foundation materials.
19.	Minor slope toe damage near outlet structure. Expected to be result of mowing tractors or other equipment traffic along toe.

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Coal Combustion Waste (CCW)

Impoundment Inspection

Impoundment NPDES Permit	**TN0005452**	**INSPECTOR**	Stanley W., Notestine, P.E. and Joseph P. Klein, III, P.E.
Date	Permit Effective Date October 1, 2003		
Impoundment Name	TVA Fossil Plant Ash Pond Outfall 001		
Impoundment Company	Tennessee Valley Authority		
EPA Region	4		
State Agency (Field Office) Address	Tennessee Department of Environment and Conservation 401 Church Street L&C Annex 6th Floor Nashville, TN 37243		
Name of Impoundment	Ash Pond Outfall 001		

(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 and settling of sluiced ash and other plant process waters.

Nearest Downstream Town Name: Kingston, TN

Distance from the impoundment: Approx. 6 miles

Location:

Latitude	35	Degrees	54	Minutes	16.59	Seconds	**N**
Longitude	84	Degrees	30	Minutes	20.84	Seconds	**W**

State Tennessee **County** Roane

	Yes	No
Does a state agency regulate this impoundment?	☒	☐

If So Which State Agency? Tennessee Department of Environment and Conservation

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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.

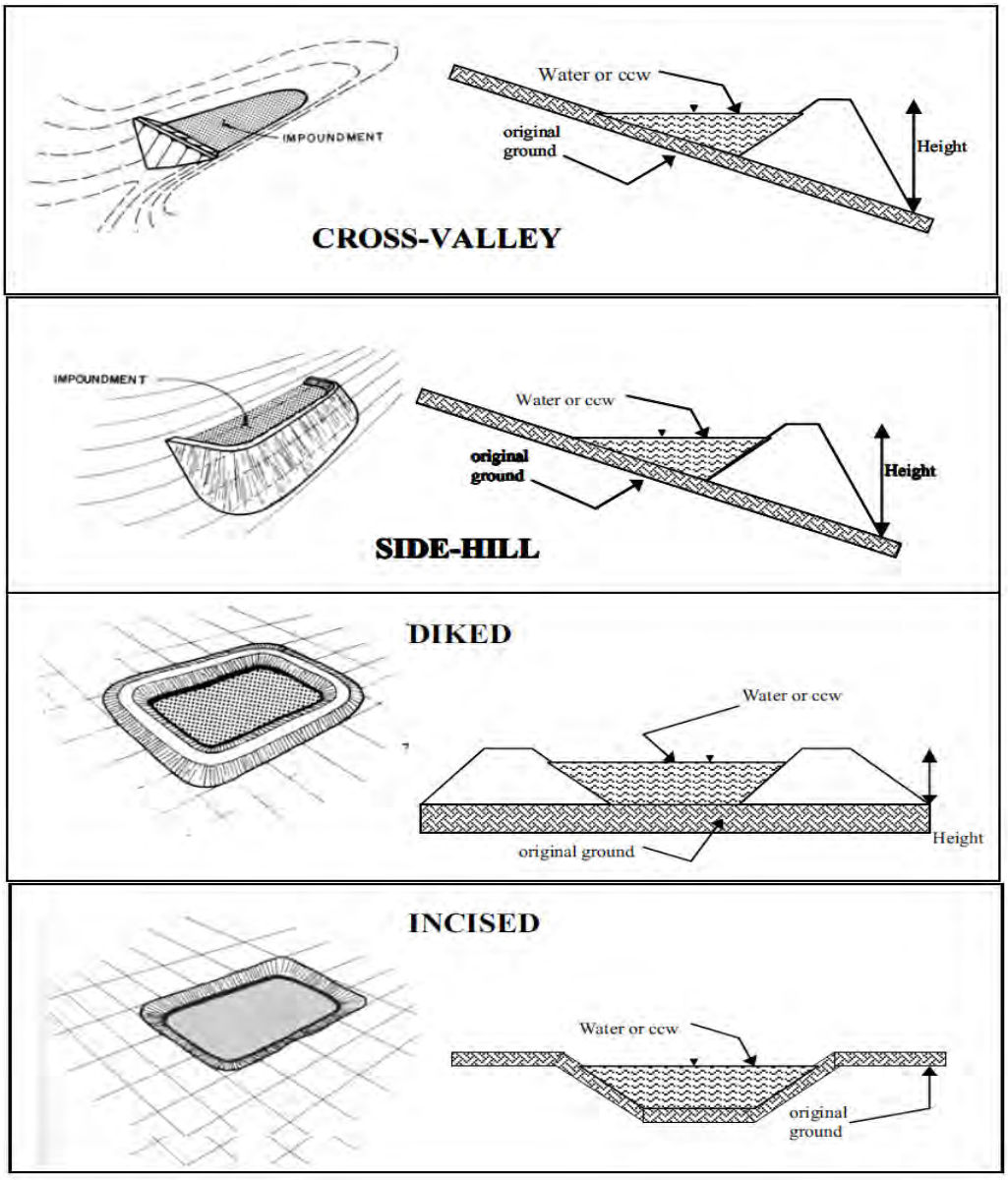
☐ **HIGH HAZARD POTENTIAL:** Dams assigned the high hazard potential classification are those where failure or misoperation will probably cause loss of human life.

DESCRIBE REASONING FOR HAZARD RATING CHOSEN:

Based on the size and location of the impoundment, a release due to embankment failure or misoperation there is not a probable loss of human life. As the impoundment is adjacent to the Emory River such a release is expected to cause economic loss and environmental damage.

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CONFIGURATION:



☐ Cross-Valley ☐ Side-Hill ☒ Diked

☐ Incised (form completion optional) ☐ Combination Incised/Diked

Embankment Height (ft)	25	**Embankment Material**	Starter Dike: Clay Starter Dike Cap: Constructed ash Raised Dike: Clay
Pool Area (ac)	120	**Liner**	No
Current Freeboard (ft)	10.3	**Liner Permeability**	N/A

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TYPE OF OUTLET (Mark all that apply)

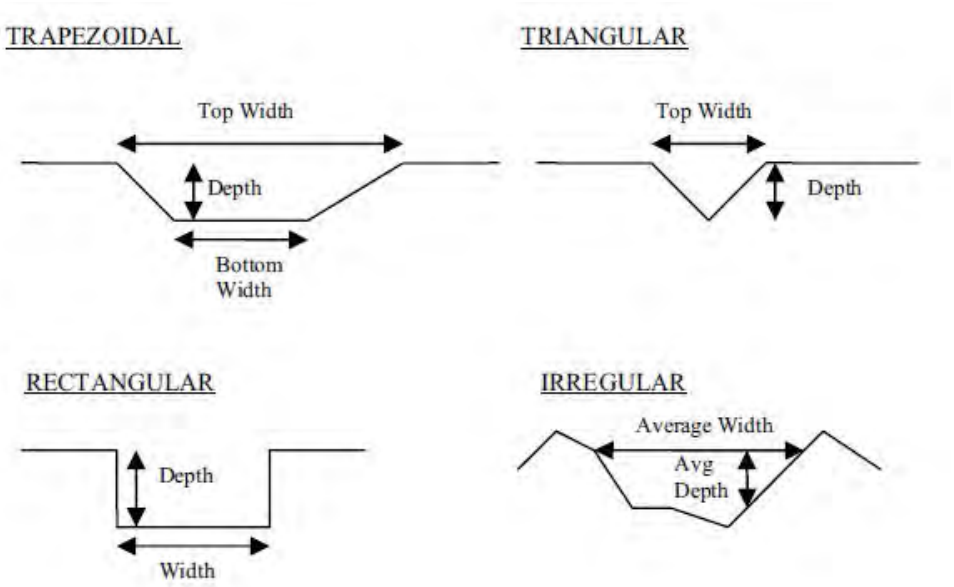
☐ **Open Channel Spillway**

- ☐ Trapezoidal
- ☐ Triangular
- ☐ Rectangular
- ☐ Irregular

depth (ft)

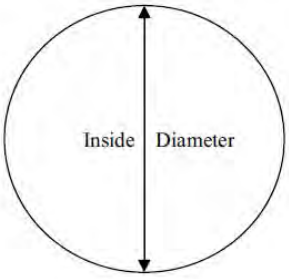
average bottom width (ft)

top width (ft)



☒ **Outlet -** Four pipe siphon system

24 " inside diameter



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):

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The Impoundment was Designed By Data not available at the time of the site visit.

	Yes	No
Has there ever been a failure at this site?	☒	☐

If So When? Dec. 2008

If So Please Describe: A dike failure in the adjacent Dredge Cell is reported to have damaged an area at the north end of Dike D, the western boundary of the Ash Pond. The damage was repaired and no release from the Ash Pond was reported.

The Stantec geotechnical report also indicates prior shallow sliding of the downstream face of Dike C in an area along the perimeter of the dredge call has been reported. The slides were reported to have been repaired as part of the TVA routine maintenance program.

	Yes	No	
Has there ever been significant seepages at this site?	☐	☐	See Below

If So When?

If So Please Describe: The August 2009 Geotechnical Exploration report prepared by Stantec Consulting Services did not cite past seepages. However a seepage analysis was conducted and concluded that conditions favorable to seepage occurrence existed within the embankment.

Rain shortly before and during the site visit limited observation of potential seepage areas.

	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,...)? Network of piezometers

If So Please Describe: Stantec installed a total of 20 piezometers in the starter dike and raised dike as part of the 2009 geotechnical exploration.

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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.

The Stantec geotechnical report does not indicate the presence of wet ash, slag, or other unsuitable materials beneath the starter dike.

The report indicates that the dike was raised using inward construction such that the raised clay dike is founded on previously deposited sluiced ash.

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.

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APPENDIX B

Document 24

Dam Inspection Check List Form Primary Bottom Ash Pond

Site Name:	**Kingston Steam Plant**	**Date:**	**21 September 2011**
Unit Name:	**Gypsum Impoundment**	**Operator's Name:**	**Tennessee Valley Authority**
Unit I.D.:	**Gypsum Impoundment**	**Hazard Potential Classification:**	**High ☐ Significant ☒ Low ☐**
	Inspector's Name:	Stanley W. Notestine, P.E. and Joseph P. Klein, III, 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)?	759		19. Major erosion or slope deterioration?		X
3. Decant inlet elevation (operator records)?			20. Decant Pipes:		
4. Open channel spillway elevation (operator records)?	X		Is water entering inlet, but not exiting outlet?		
5. Lowest dam crest elevation (operator records)?	767.9		Is water exiting outlet, but not entering inlet?		
6. If instrumentation is present, are readings recorded (operator records)?	X		Is water exiting outlet flowing clear?		
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)?			From underdrain?		
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?			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?			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
7.	The embankment impounding the stormwater pond and cell 1 of the gypsum impoundment is essentially completed and the stormwater pond is in service. Gypsum cell 1A lining is currently being installed as is the subgrade drainage system. A temporary interior dike that will separate cells 1A and 1B has been started, but no activity is underway in cell 1B. It is being used as a lay-down area for materials and equipment working in cell 1A. No CCR is currently present in the impoundment.
13.	Small karst features were encountered during excavation and grading for the gypsum impoundment. Design changes were made to stabilize the karst features and reduce the risk of adverse impacts of potential future karst development.
23.	The Clinch River is adjacent to the toe of the gypsum impoundment south dike embankment.

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Coal Combustion Waste (CCW)

Impoundment Inspection

Impoundment NPDES Permit	TN 0080870	**INSPECTOR**	Stanley W. Notestine, P.E. and Joseph P. Klein, III, P.E.

Date Permit Effective Date: October 1, 2009

Impoundment Name Kingston Plant Flue Gas Desulfurization Detention Pond

Impoundment Company Tennessee Valley Authority

EPA Region 4

State Agency (Field Office) Address Tennessee Department of Environment and Conservation
401 Church Street
L&C Annex 6th Floor
Nashville, TN 37243

Name of Impoundment Kingston Plant Flue Gas Desulfurization Detention Pond

(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:

Nearest Downstream Town Name:

Distance from the impoundment:

Location:

Latitude	35	Degrees	53	Minutes	37.91	Seconds	**N**
Longitude	84	Degrees	30	Minutes	49.70	Seconds	**W**

State Tennessee **County** Roane

	Yes	No
Does a state agency regulate this impoundment?	☒	☐

If So Which State Agency? Tennessee Department of Environment and Conservation

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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.

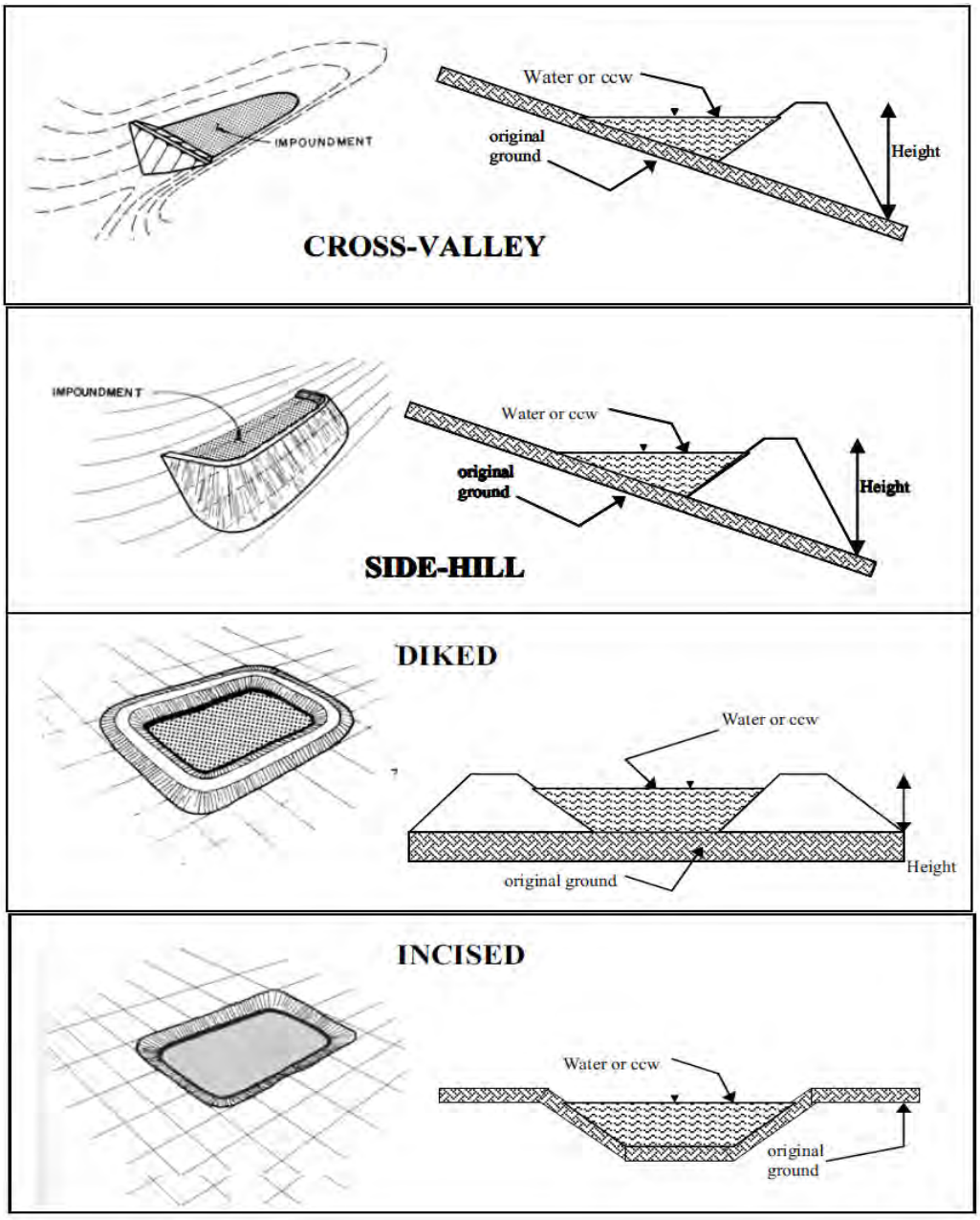
☐ **HIGH HAZARD POTENTIAL:** Dams assigned the high hazard potential classification are those where failure or misoperation will probably cause loss of human life.

DESCRIBE REASONING FOR HAZARD RATING CHOSEN:

Based on the size and location of the impoundment, a release due to embankment failure or misoperation there is not a probable loss of human life. As the impoundment is adjacent to the Clinch River such a release is expected to cause economic loss and environmental damage.

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CONFIGURATION:



☐ Cross-Valley ☒ Side-Hill ☐ Diked

☐ Incised (form completion optional) ☐ Combination Incised/Diked

Embankment Height (ft)

Pool Area (ac)

Current Freeboard (ft)

Embankment Material

Liner HDPE

Liner Permeability

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TYPE OF OUTLET (Mark all that apply)

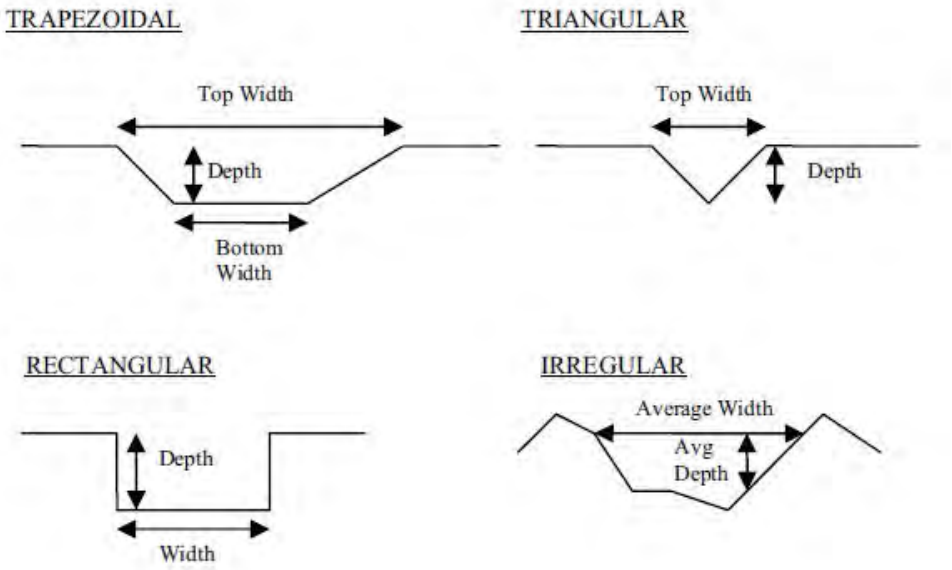
☐ **Open Channel Spillway**

- ☐ Trapezoidal
- ☐ Triangular
- ☐ Rectangular
- ☐ Irregular

depth (ft)

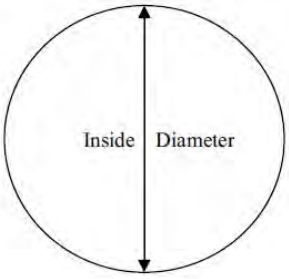
average bottom width (ft)

top width (ft)



☐ **Outlet**

91" 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

	Yes	No
Has there ever been a failure at this site?	☐	☒
If So When?		

If So Please Describe :

	Yes	No
Has there ever been significant seepages at this site?	☒	☐
If So When?	2010	

If So Please Describe : Sinkholes in Gypsum Stack foundation. Facility redesigned and reconstructed with liner.

	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,...)?		

If So Please Describe :

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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.

No. Observations during the site visit the embankment was substantially complete and the stormwater pond was in service. However, the gypsum cells were uncompleted and no CCR was present in the impoundment. The embankment appeared to be supported on natural ground at the site.

Did the dam assessor meet with, or have documentation from, the design Engineer-of-Record concerning the foundation preparation?

From the site visit or from photographic documentation, was there evidence of prior releases, failures, or patchwork on the dikes?

No.

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