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Coal Combustion Residue Impoundment

Round 11 - Dam Assessment Report

Watts Bar Fossil Plant (Site #12)

Old Ash Pond and Stilling Basin
Tennessee Valley Authority
Spring City, TN

Prepared for:

United States Environmental Protection Agency
Office of Resource Conservation and Recovery

Prepared by:

Dewberry & Davis LLC
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INTRODUCTION, SUMMARY CONCLUSIONS AND RECOMMENDATIONS

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

This assessment of the stability and functionality of the Watts Bar Old Ash Pond and Stilling Basin (inactive for at least the last 25 years) is based on a review of available documents and on the site assessment conducted by Dewberry personnel on September 15, 2011. We found the supporting technical documentation inadequate (Section 1.1.3). As detailed in Section 1.2, there are 3 recommendations based on field observations that may help to maintain a safe and trouble-free operation.

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

In February 2009, the EPA sent letters to coal-fired electric utilities seeking information on the safety of surface impoundments and similar facilities that receive liquid-borne material that store or dispose of coal combustion residue. This letter was issued under the authority of the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) Section 104(e), to assist the Agency in assessing the structural stability and functionality of such management unit(s), 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 unit(s) including surface impoundments or similar diked or bermed management unit(s) or management unit(s) 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 the Watts Bar Fossil Plant Old Ash Pond and Stilling Basin that were reviewed and used in preparation of this report.

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

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

LIMITATIONS

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

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1.0 CONCLUSIONS AND RECOMMENDATIONS

1.1 CONCLUSIONS

Conclusions are based on visual observations from a one-day site visit, September 15, 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)

Based upon the Dewberry site visit the facility's embankments appear to be structurally sound. Supporting technical documentation demonstrate the management unit is structurally sound.

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

A hydrologic and hydraulic analysis was not provided to Dewberry. A conclusion regarding the hydrologic/hydraulic safety of the management unit cannot be made at this time.

1.1.3 Conclusions Regarding the Adequacy of Supporting Technical Documentation

The supporting technical documentation is inadequate due to the lack of a hydraulic/hydrologic analysis. 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(s) 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 field 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. The pond embankment appears structurally sound. There are no apparent indications of unsafe conditions.

1.1.6 Conclusions Regarding the Adequacy of Maintenance and Methods of Operation

The current maintenance and methods of operation appear to be adequate for the old fly ash management unit because the power plant is no longer functional. The field inspection revealed no evidence of significant embankment repairs or prior releases. There are a few areas of tree growth and excessive vegetation on the embankment.

1.1.7 Conclusions Regarding the Adequacy of the Surveillance and Monitoring Program

The surveillance and monitoring program that is in place is adequate since the power plant has been inactive for over 25 years.

1.1.8 Classification Regarding Suitability for Continued Safe and Reliable Operation

The facility is rated FAIR 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 rating.

1.2 RECOMMENDATIONS

1.2.1 Recommendations Regarding the Structural Stability

It is recommended that the banks of the Tennessee River which are adjacent to the ash pond be laid back and lined with rip-rap to prevent future erosion due to wear action along the banks.

It is also recommended that frequent inspections of the management unit embankment be completed until final closure is complete to visibly assess whether existing conditions are altered, helping to ensure structural stability.

1.2.2 Recommendations Regarding the Hydrologic/Hydraulic Safety

It is recommended that a hydraulic/hydrologic analysis be performed to demonstrate that ash is not released to the Tennessee River during the design storm event. Receipt of this analysis could lead to a change in the rating to Satisfactory.

1.2.3 Recommendations Regarding the Maintenance and Methods of Operation

Tree growth was observed along the pond's embankment. The field report notes 3 trees with a maximum diameter of 3-4 inches. It is recommended that the embankment be properly maintained to remove existing trees, remove excess vegetation, and prevent future growth.

1.3 PARTICIPANTS AND ACKNOWLEDGEMENT

1.3.1 List of Participants

Robert Fuller, Stantec
Jimmy Mullins, Tennessee Valley Authority
Paul Pearman, Tennessee Valley Authority
Bronson L. Reed, Tennessee Valley Authority
Brett Whatt, Tennessee Valley Authority
Bill Roddy, Tennessee Valley Authority
Marty Helton, Tennessee Valley Authority
Stanley Nixon, Tennessee Valley Authority
Stanley W. Notestine, Dewberry
James Filson, Dewberry

1.3.2 Acknowledgement and Signature

We acknowledge that the management unit referenced herein has been assessed on September 15, 2011.

Stanley W. Notestine, P.E.   James Filson, P.E.

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

2.1 LOCATION AND GENERAL DESCRIPTION

The Watts Bar Fossil Plant, owned by the Tennessee Valley Authority, was completed in 1942, and operated until it was decommissioned in 1983. Watts Bar Fossil Plant was a coal-fired power plant and produced fly ash and bottom ash as byproducts of coal combustion. The ash pond for this facility was completed in 1974, see Appendix A, Document 2. An expansion took place in 1977 that added a bottom ash dike to form the stilling basin. The pond and stilling basin are now used for stormwater management for surface runoff.

The Watts Bar Fossil Plant is located near Spring City, Tennessee. The plant is just south of Watts Bar Reservoir on the Tennessee River. See Figure 2.1-1 Location Map and Figure 2.1-2 Aerial Photograph.

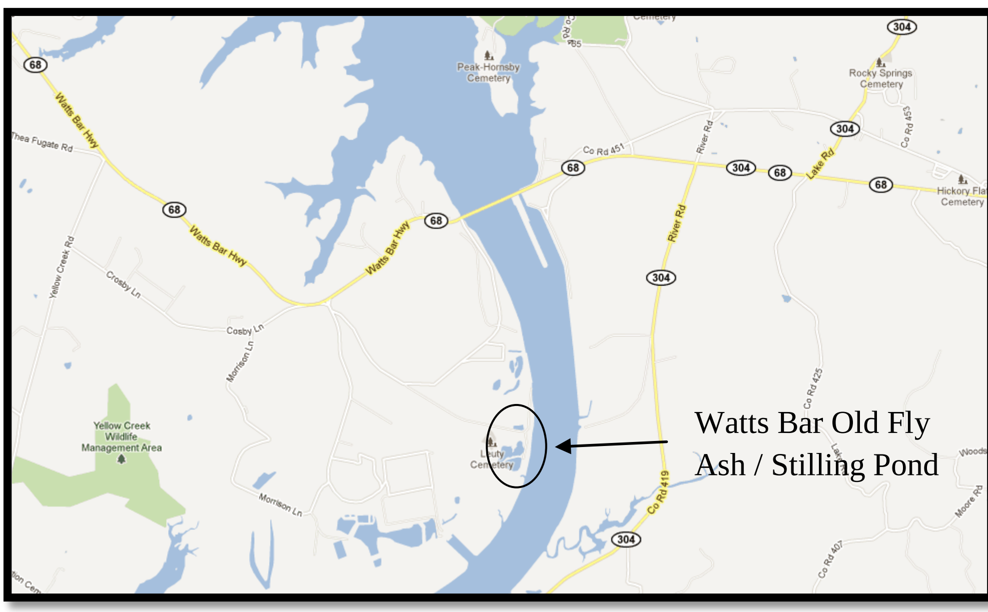

Figure 2.1-1: Location Map

Figure 2.1-2: Aerial Photograph

Table 2.1: Summary of Embankment Dimensions and Size	
	Ash Pond
Embankment Height (ft)	11
Crest Width (ft)	30-35
Length (ft)	2000
Side Slopes (upstream) H:V	1.5:1
Side Slopes (downstream) H:V	1.5:1

2.2 COAL COMBUSTION RESIDUE HANDLING

Fly ash and bottom ash are not currently being added to the ash pond. The Watts Bar Fossil Plant was decommissioned in 1983 and the pond currently acts only as a stormwater management facility for surface runoff from the Watts Bar site.

2.3 SIZE AND HAZARD CLASSIFICATION

The total storage capacity of the ash pond is 230,000 cubic feet or 5.28 acre-feet, see Appendix A, Document 2. The embankment has a height of 11 feet. These two values lead to the impoundment being classified as small based on 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	50 and < 1,000	25 and < 40
Intermediate	1,000 and < 50,000	40 and < 100
Large	> 50,000	> 100

The ash pond has a hazard classification of low based on the guidelines in table 2.2b. The economic and environmental losses due to a dam failure would be relatively low and would be limited to the site owned by the Tennessee Valley Authority. There would be no expected loss of life as a result of a failure of the ash pond embankment.

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)

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

Fly ash and bottom ash have not been produced by the Watts Bar Fossil Plant since 1983. No additional ash has been added to the pond since that time and therefore the level of fly and bottom ash residuals has not increased. The pond is currently only used as a stormwater management facility for surface runoff. There is currently 9 feet of freeboard in the pond. Table 2.3 summarizes the storage capacity of the pond.

Table 2.3: Maximum Capacity of Unit	
Watts Bar Fossil Plant Ash Pond	
Surface Area (acre)[1]	15
Current Storage Capacity (cubic yards)[1]	80,000
Current Storage Capacity (acre-feet)	50
Total Storage Capacity (cubic yards)[1]	230,000
Total Storage Capacity (acre-feet)	143
Crest Elevation (feet)	713
Normal Pond Level (feet)	704

[1]Appendix A, Document 2.

2.5 PRINCIPAL PROJECT STRUCTURES

2.5.1 Earth Embankment

An earth embankment was constructed to form two sides of the pond and to impound water, fly ash, and bottom ash. This embankment lies between the pond and the Tennessee River which is located to the east of the pond. The embankment was designed to use a side-hill configuration to contain water and ash. This configuration utilizes a hill's slope and allows for the embankment to only be necessary on two of the pond's sides. The embankment is 11 feet high and between 30 and 35 feet wide.

2.5.2 Outlet Structures

There are several outlet structures within the old ash pond. An open trapezoidal channel comes off of the pond and acts as a spillway for the facility. The old ash pond has 3 outlet pipes. Each pipe is concrete and has an inside diameter of 36 inches. Clear water was flowing through the pipes at the time of the site visit. These pipes discharge from the stilling basin and into the Tennessee River.

2.6 CRITICAL INFRASTRUCTURE WITHIN FIVE MILES DOWN GRADIENT

There is no critical infrastructure located immediately downstream of the ash pond and stilling basin. The nearest downstream town is Dayton, Tennessee at a distance of approximately 20 miles.

3.0 SUMMARY OF RELEVANT REPORTS, PERMITS, AND INCIDENTS

3.1 SUMMARY OF LOCAL, STATE, AND FEDERAL ENVIRONMENTAL PERMITS

Discharge from the impoundment is regulated by the Tennessee Department of Environment and Conservation and the impoundment has been issued a National Pollutant Discharge Elimination System Permit. Permit No. TN0005461 was issued June 30, 2011 (See Appendix A, Document 1).

3.2 SUMMARY OF SPILL/RELEASE INCIDENTS

Data reviewed by Dewberry did not indicate any spills or unpermitted releases over the last 10 years. There has been at least one incident of minor seepage along the toe of the embankment previously observed by the Tennessee Valley Authority, see Appendix A, Document 3.

4.0 SUMMARY OF HISTORY OF CONSTRUCTION AND OPERATION

4.1 SUMMARY OF CONSTRUCTION HISTORY

4.1.1 Original Construction

The Watts Bar Fossil Plant was completed in 1942. The facility's ash pond was completed in 1974. The pond was constructed to contain fly and bottom ash from the coal power plant.

4.1.2 Significant Changes/Modifications in Design since Original Construction

A bottom ash dike was added in 1977 to form the stilling basin portion of the facility.

4.1.3 Significant Repairs/Rehabilitation since Original Construction

Data reviewed by Dewberry does not show any significant repairs or rehabilitation since the original construction.

4.2 SUMMARY OF OPERATIONAL PROCEDURES

4.2.1 Original Operational Procedures

Data reviewed by Dewberry did not contain the original operational procedures for the ash pond and stilling basin.

4.2.2 Significant Changes in Operational Procedures and Original Startup

The Watts Bar Fossil Plant was closed in 1983 and since that time the ash pond has not been used to store fly ash or bottom ash. Since 1983, the pond and stilling basin have been used as a stormwater management facility for surface runoff.

4.2.3 Current Operational Procedures

The old ash pond and stilling basin still serve as a stormwater management facility. Inspections are accomplished regularly to maintain the facility and prevent any possible safety problems.

4.2.4 Other Notable Events since Original Startup

No information was provided to Dewberry concerning notable events impacting the operation of the ash pond and stilling basin.

5.0 FIELD OBSERVATIONS

5.1 PROJECT OVERVIEW AND SIGNIFICANT FINDINGS

Dewberry personnel Stanley W. Notestine, P.E. and Jim Filson, P.E. performed a site visit on Thursday, September 15, 2011, in company with the participants.

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

The overall assessment of the dam was that it was in fair condition and no significant findings were noted.

5.2 ASH POND AND STILLING BASIN

5.2.1 Crest

The crest of the embankment had no signs of significant depressions, tension cracks or other indications of settlement or shear failure. Figures 5.2.1-1 and 5.2.1-2 show the typical crest conditions along the embankment.

Figure 5.2.1-1: Crest of northeastern portion of embankment. The ash pond is to the right and the Tennessee River is to the left.

Figure 5.2.1-2: Crest of northern portion of embankment. The ash pond is to the left and the Watts Bar Fossil Plant is to the right.

5.2.2 Upstream/Inside Slope

The inside slopes of the embankment had a well maintained cover of grasses/weeds. There were no observed scarps, sloughs, bulging, cracks, depressions or other indications of slope instability. Figures 5.2.2-1 and 5.2.2-2 show different sections of the inside slopes.

Figure 5.2.2-1: Inside slope of southern portion of embankment.

Figure 5.2.2-2: Inside slope of northeast portion of embankment.

5.2.3 Downstream/Outside Slope and Toe

The outside slopes of the embankment to the north and east had a well maintained cover of grasses/weeds. The outside slopes to the south had significant vegetative cover. There were some areas of sloughing at the toe of the embankment. Some woody vegetation was noted to have grown on the outside slope of the pond. Figures 5.2.3-1, 5.2.3-2, and 5.2.3-3 show different sections of the outside slopes.

Figure 5.2.3-1: Outside slope of northeast portion of embankment. Note tree on embankment slope.

Figure 5.2.3-2: Outside toe of northeast portion of embankment. Note erosion.

Figure 5.2.3-3: Outside slope of southern portion of embankment. Note tree growth and excess vegetation.

5.2.4 Abutments and Groin Areas

All abutments had a maintained cover of grasses/weeds. There were no observed scarps, sloughs, bulging, cracks, depressions or other indications of instability.

5.3 OUTLET STRUCTURES

5.3.1 Riser Structures

The pond's riser structures were in good condition. There was no apparent blockage or damage to any of the structures. Figures 5.3.1-1 and 5.3.1-2 show different riser structures.

Figure 5.3.1-1: Riser structures. Picture taken from eastern portion of embankment.

Figure 5.3.1-2: Riser structure.

5.3.2 Outlet Conduit

All of the 3 outlet pipes were unclogged and in good condition. Clear water was flowing through each of the 3 pipes. Figure 5.3.2-1 shows all 3 outlet pipes.

Figure 5.3.2-1: 3 - 36 inch outlet pipes discharging towards the Tennessee River.

5.3.3 Emergency Spillway

The emergency spillway was unclogged and in good condition.

6.0 HYDROLOGIC/HYDRAULIC SAFETY

6.1 SUPPORTING TECHNICAL DOCUMENTATION

6.1.1 Flood of Record

No documentation was provided to Dewberry regarding local flood records. USGS river gage 03543005 is located along the Tennessee River downstream of the Watts Bar Dam. During the recorded period, the largest peak flow occurred in 1984, see Exhibit 1. This peak flow is comparable to the Tennessee River 1% annual chance (100 year) peak discharge found in the Rhea County FIS Study, see Exhibit 2. The Rhea County FIRM dated November 5, 2008, (Map Number 47143C0260D) shows that the ash pond is in an area determined to be above the 0.2% annual chance flood elevation, see Exhibit 3. Since the 0.2% annual chance flood is larger than the 1% annual chance flood, the ash pond and its embankments are safely above the elevation for the flood of record.

A large storm recently dropped 9-11 inches of rainfall on the area of the Watts Bar Fossil Plant. The Tennessee Valley Authority reported that this large storm only caused the water surface elevation of the old ash pond to increase by approximately 1-1.5 feet. This small increase for such a large storm indicates the drainage area of the old ash pond is relatively small.

6.1.2 Inflow Design Flood

Data reviewed by Dewberry did not contain inflow design flood information.

6.1.3 Spillway Rating

Data reviewed by Dewberry did not contain spillway rating information.

6.1.4 Downstream Flood Analysis

Data reviewed by Dewberry did not contain downstream flood analysis information.

6.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Supporting documentation reviewed by Dewberry is inadequate. Dewberry was not provided with a hydrologic/hydraulic report.

6.3 ASSESSMENT OF HYDROLOGIC/HYDRAULIC SAFETY

Based on the site visit, pond response to a large rainfall event, and Dewberry's evaluation, the facility has adequate hydrologic/hydraulic safety. A hydrologic/hydraulic report is recommended to support the visual assessment made during the site visit.

7.0 STRUCTURAL STABILITY

7.1 SUPPORTING TECHNICAL DOCUMENTATION

7.1.1 Stability Analyses and Load Cases Analyzed

Dewberry was provided CDM Smith's *Report Existing Conditions Stability Analyses*, dated January 31, 2012 (see Appendix A, Document 4) and the Revision 1 report, dated April 30, 2012 (see Appendix A, Doc 5). These documents summarize the slope stability of the disposal facility's embankments under static and seismic conditions. The stability analysis is based on previously available site information as well as recent information acquired by CDM Smith, including August 2011 bulk sampling & laboratory testing and January 2012 subsurface exploration. The slope stability of the embankment considered static and seismic conditions under steady-state seepage.

7.1.2 Design Parameters and Dam Materials

Design parameters considered in the slope stability analysis are provided in the CDM Smith report.

A total of 3 boring logs were completed in January 2012 by CDM Smith. Embankment material was characterized based on these boring logs. Embankment material is described to be layered in fill, medium stiff to stiff clay, soft clay and silt, sand, weathered rock and gravel, and inter-bedded shale and limestone bedrock.

7.1.3 Uplift and/or Phreatic Surface Assumptions

Along with the 2012 borings, 2 new groundwater observation wells were installed. Well readings were used to determine phreatic surface elevations along the embankment for use in the slope stability analysis. In addition to consideration of groundwater elevations, water levels observed in the Ash/Stilling Pond as well as water levels of the Tennessee River were used in developing normal pool condition of 705-ft.

7.1.4 Factors of Safety and Base Stresses

Two critical sections were used in the slope stability analysis, one through the wet pond area (Section A-A') and a second through the dry ash area along the embankment (Section B-B'). Each section was analyzed considering steady state seepage using effective engineering parameters such as unit weight and shear strength properties of the subsurface materials.

CDM Smith's 2012 stability analysis referenced seismic forces for the Watts Bar Fossil Plant location that correspond to an approximate exceedance probability of 2-percent for 50 years. This return period is consistent with the seismic stability analysis guidance provided by the US Army Corp of Engineers. The horizontal seismic coefficient considered a peak ground acceleration with 2-percent probability of exceedance in 50-years of 0.116g.

A summary of the computed safety factors is included in Table 7.1.4.

Table 7.1.4 Factors of Safety for Watts Bar Fossil Plant

	Section A-A' (Wet Area)	Section B-B' (Dry Ash Area)
Calculated Static Loading Safety Factor Inboard Slope	1.9	2.4
Calculated Static Loading Safety Factor Outboard Slope	1.8	1.5
Required Safety Factor (US Army Corp of Engineers	1.5	1.5
Calculated Seismic Loading Safety Factor Inboard Slope	1.4	1.3
Calculated Seismic Loading Safety Factor Outboard Slope	1.1	1.0
Required Safety Factor (US Army Corp of Engineers	>1.0	>1.0

7.1.5 Liquefaction Potential

No assessment of liquefaction potential was performed for the existing conditions.

7.1.6 Critical Geological Conditions

Data reviewed by Dewberry did not contain geological information.

7.2 ADEQUACY OF SUPPORTING TECHNICAL DOCUMENTATION

Slope stability documentation provided for the existing conditions is adequate.

7.3 ASSESSMENT OF STRUCTURAL STABILITY

Overall, the structural stability of the Watts Bar embankment appears to be **Satisfactory** based on the following observations:

- Safety factors for static stability and seismic stability meet the minimum required by the US Army Corp of Engineers guidance.

8.0 ADEQUACY OF MAINTENANCE AND METHODS OF OPERATION

8.1 OPERATING PROCEDURES

The plant has been closed for more than 25 years. There is currently no fly ash being added to the Old Ash Pond and Stilling Basin. The pond currently serves only as a stormwater management facility for site runoff. The facility has both an emergency action plan and a seepage action plan in place.

8.2 MAINTENANCE OF THE DAM AND PROJECT FACILITIES

The site visit showed no signs of damage or lack of maintenance. An adequate maintenance plan was provided to Dewberry by Tennessee Valley Authority. Mowing is done along some of the pond embankment in an effort to prevent excess vegetation. Mowing is not done along the portion of the embankment located on the nuclear power plant site.

8.3 ASSESSMENT OF MAINTENANCE AND METHODS OF OPERATIONS

8.3.1 Adequacy of Operating Procedures

The facility's operating procedures are adequate based on the power plant's closure and the Dewberry site visit.

8.3.2 Adequacy of Maintenance

Based on the assessments of this report, operation procedures seemed to be adequate.

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

9.0 ADEQUACY OF SURVEILLANCE AND MONITORING PROGRAM

9.1 SURVEILLANCE PROCEDURES

The facility is being inspected annually to identify maintenance issues and prevent future safety problems. See Appendix A, Document 3 for the 2009 facility assessment completed by Stantec.

9.2 INSTRUMENTATION MONITORING

The ash pond has monitoring wells and piezometers but instrument readings are not being taken and recorded.

9.3 ASSESSMENT OF SURVEILLANCE AND MONITORING PROGRAM

9.3.1 Adequacy of Inspection Program

The inspection program is deemed adequate based on observations during the site visit and a review of the 2009 facility assessment. The current inspection program is adequate because the facility is inspected annually by Stantec and shows no major signs of a lack of maintenance.

9.3.2 Adequacy of Instrumentation Monitoring Program

Based on the data reviewed by Dewberry, the instrumentation monitoring program is adequate. The program is adequate because the Watts Bar Fossil Plant has been closed for more than 25 years.

Exhibit 1: USGS Peak Streamflow, USGS 03543005 Tennessee River at Watts Bar Dam

Peak Streamflow for the Nation

USGS 03543005 TENNESSEE RIVER AT WATTS BAR DAM (TAILWATER), TN

Available data for this site: Surface-water: Peak streamflow GO

Rhea County, Tennessee
Hydrologic Unit Code 06010201
Latitude 35°37'13", Longitude 84°47'00" NAD27
Drainage area 17,310.00 square miles

Output formats

- Table
- Graph
- Tab-separated file
- peakfq (watstore) format
- Reselect output format

Water Year	Date	Gage Height (feet)	Stream-flow (cfs)
1936	Mar. 28, 1936		202,000[1,6]
1975	Mar. 14, 1975		136,000[1,6]
1976	Jan. 10, 1976		45,400[1,6]
1977	Apr. 05, 1977		161,000[1,6]
1978	Jan. 28, 1978		79,500[1,6]
1979	Mar. 09, 1979		111,000[1,6]
1980	Mar. 21, 1980		79,200[1,6]
1981	Aug. 14, 1981		41,200[1,6]
1982	Feb. 12, 1982		77,100[1,6]
1983	May 23, 1983		84,600[1,6]
1984	May 08, 1984		208,000[1,6]
1985	Feb. 02, 1985		54,900[1,6]
1986	Dec. 04, 1985		35,400[1,6]

Peak Streamflow Qualification Codes.

- 1 -- Discharge is a Maximum Daily Average
- 6 -- Discharge affected by Regulation or Diversion

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Exhibit 2: FEMA Rhea County FIS Study, Table 2-Summary of Discharges

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. mi.)	PEAK DISCHARGES (cfs) 10% annual chance	2% annual chance	1% annual chance	0.2% annual chance
LITTLE RICHLAND CREEK (Cont'd)					
just upstream of confluence of Yarborough Branch	3.30	660	1,040	1,200	1,650
at Norfolk Southern Railway	2.48	530	850	1,000	1,350
MCGILL CREEK					
at mouth	13.3	N/A	N/A	6,200	N/A
about 0.9 mile upstream of mouth	13.0	N/A	N/A	6,200	N/A
PINEY RIVER					
just downstream of confluence of Vans Creek	101.3	14,200	22,000	26,100	37,300
at J. Lon Foust Highway	95.9	13,600	21,200	25,100	35,900
just downstream of confluence of Soak Creek	92.9	13,300	20,800	24,500	35,100
RICHLAND CREEK					
at U.S. Route 27	54.1	9,985	15,775	18,565	25,655
ROARING CREEK					
at Harrison Street	27.7	6,200	9,700	10,600	14,800
just downstream of confluence of Flora Branch	26.3	5,900	9,300	10,200	14,200
SALE CREEK					
just upstream of confluence of Roaring Creek	14.0	N/A	N/A	6,400	8,900
at Burnett Street	13.2	N/A	N/A	6,000	8,300
about 0.5 mile upstream of confluence of Hickman Branch	9.5	N/A	N/A	4,900	6,800
TENNESSEE RIVER					
about 36 miles downstream of Watts Bar Dam	N/A	N/A	212,000	224,000	292,000
just downstream of Watts Bar Dam	17,310	168,000	198,000	214,000	273,000
TOWN CREEK					
just downstream of confluence of Ford Branch	5.56	906	1,500	1,770	2,370

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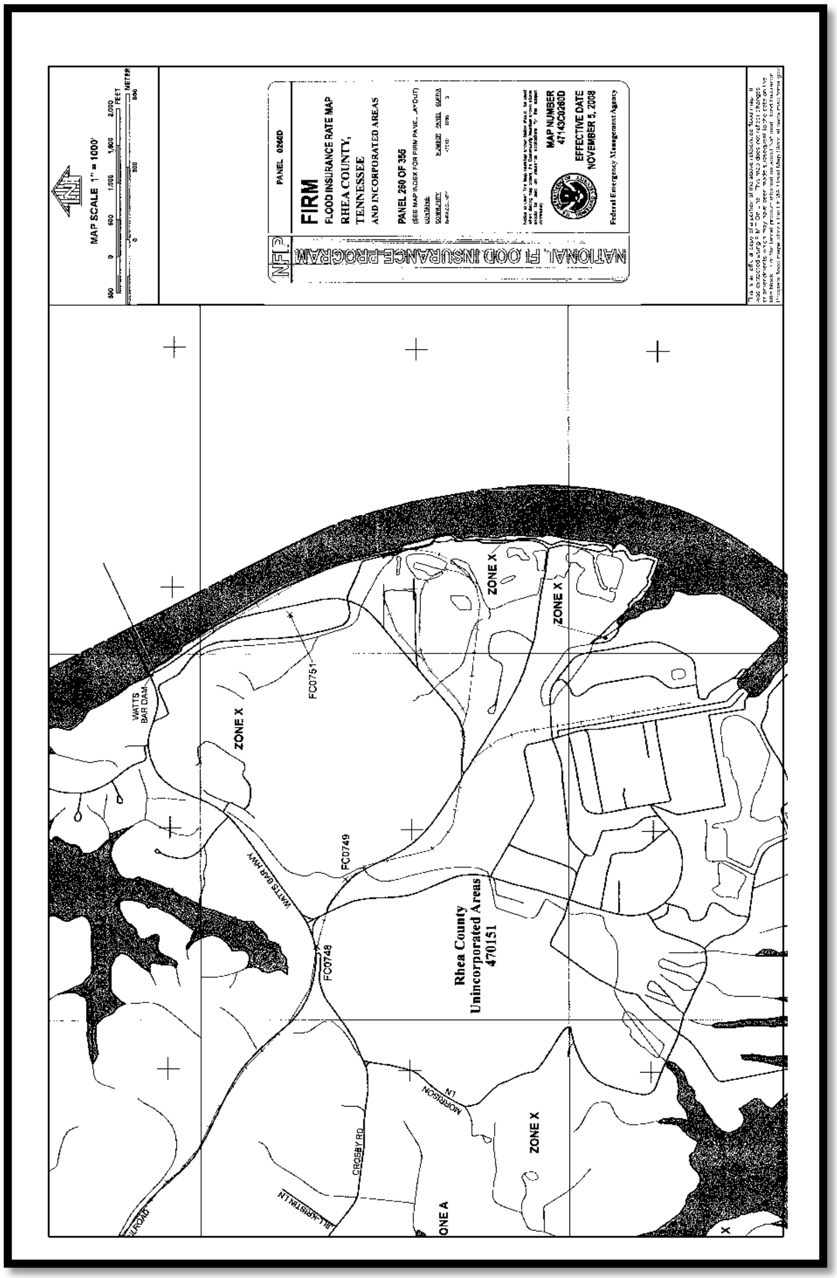
Exhibit 3: FEMA Rhea County FIRM, Map Number 47143C0260D



APPENDIX A

Document 1

State of Tennessee NPDES Permit

STATE OF TENNESSEE

NPDES PERMIT

MODIFIED
No. TN0005461

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 - Watts Bar Fossil Plant**
is authorized to discharge:	**combined process and non-process wastewater and storm water runoff from Outfall 002**
from a facility located:	**in Spring City, Rhea County, Tennessee**
to receiving waters named:	**Tennessee River between river miles 528 and 530**

in accordance with effluent limitations, monitoring requirements and other conditions set forth herein.

This permit shall become effective on:	**September 1, 2011**
This permit shall expire on:	**August 31, 2016**
Issuance date:	**June 30, 2011**

For

Paul E. Davis, Director
Division of Water Pollution Control

CN-0759

RDAs 2352 and 2366

APPENDIX A

Document 2

Tennessee Valley Authority Response to Environmental Protection Agency Request for Information

Tennessee Valley Authority
400 West Summit Hill Drive
Knoxville, Tennessee 37902-1401

Anda A. Ray
Senior Vice President
Office of Environment and Research

March 25, 2009

Mr. Richard Kinch
U.S. Environmental Protection Agency
Two Potomac Yard
2733 South Crystal Drive
5th Floor: N-5783
Arlington, Virginia 22202-2733

Dear Mr. Kinch:

Enclosed is the Tennessee Valley Authority's (TVA) response to your requests for information about coal-combustion by-product management impoundments and our signed authorized certification. Your requests were received at TVA's plant sites on March 12 and March 13. Enclosed is the consolidated response from TVA for all of our fossil plants. We have also included in our response two plants (Watts Bar Fossil Plant, inactive and Cumberland Fossil Plant) for which we did not receive a request for information.

Sincerely,

Anda A. Ray

Enclosures: 2007-2008 Annual Inspection Reports of Waste Disposal Areas for all TVA fossil plants.
TVA Responses to EPA Information Request.
Ash Storage Summary.
Certification Form.

EPA believes that the information requested is essential to an evaluation of the threat of releases of pollutants or contaminants from these units. The provisions of Section 104 of CERCLA authorize EPA to pursue penalties for failure to comply with or respond adequately to an information request under Section 104(e). In addition, providing false, fictitious or fraudulent statements or representations may subject you to criminal penalties under 18 U.S.C. 1001.

Your response must include the following certification signed and dated by an authorized representative of Tennessee Valley Authority.

> I certify that the information contained in this response to EPA's request for information and the accompanying documents is true, accurate, and complete. As to the identified portions of this response for which I cannot personally verify their accuracy, I certify under penalty of law that this response and all attachments were prepared in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fines and imprisonment for knowing violations.

Signature: [signature]
Name: John C. Kammeyer
Title: VP, Engineering

This request has been reviewed and approved by the Office of Management and Budget pursuant to the Paperwork Reduction Act, 44 U.S.C., 3501-3520.

Please send your reply to:

Mr. Richard Kinch
US Environmental Protection Agency (5306P)
1200 Pennsylvania Avenue, NW
Washington, DC 20460

If you are using overnight or hand delivery mail, please use the following address:

Mr. Richard Kinch
US Environmental Protection Agency
Two Potomac Yard
2733 S. Crystal Dr.
5th Floor; N-5783
Arlington, VA 22202 2733

Tennessee Valley Authority Response to Environmental Protection Agency Request for Information

1. **Relative to the National Inventory of Dams criteria for High, Significant, Low, or Less-than-Low, please provide the potential hazard rating for each management unit and indicate who established the rating, what the basis of the rating is, and what federal or state agency regulates the unit(s). If unit(s) does not have a rating, please note that fact.**

   The dam safety hazard potential rating for each management unit is identified on the attached table. The current hazard potential ratings were assigned by TVA using the National Inventory of Dams criteria as a guideline. Hazard classifications have not been assigned to dry disposal management units. The list is updated by TVA every 2 years. No other agencies, federal or state, regulate these facilities from a dam safety perspective.

   Currently, TVA has secured the services of a third party consultant to review the conditions at our coal combustion storage facilities and provide opinions relative to hazard potential. These opinions will be based on the National Inventory of Dams criteria, as well as dam safety regulations of the states in which each unit is located.

2. **What year was each management unit commissioned and expanded?**

   The year each management unit was commissioned and expanded is identified in the attached table.

3. **What materials are temporarily or permanently contained in the unit? Use the following categories to respond to the question: (1) fly ash; (2) bottom ash; (3) boiler slag; (4) flue gas emission control residuals; (5) other. If the management unit contains more than one type of material, please identify all that apply. Also, if you identify "other", please specify, the other types of materials that are temporarily or permanently contained in the unit(s)**

   The coal-combustion byproduct materials contained in each unit are identified in the attached table. Impoundments at units are also routinely used to combine and treat a variety of runoff and low volume water wastes prior to discharge.

4. **Was the management unit(s) designed by a Professional Engineer? Is or was the construction of the waste management unit(s) under the supervision of a Professional Engineer? Is inspection and monitoring of the safety of the waste management unit(s) under the supervision of a Professional Engineer?**

   Permitted solid waste landfill design documents were prepared under the supervision of a registered professional engineer, with design documents stamped by the responsible engineer. In general, for non-permitted management units, the design and construction, along with the inspection and monitoring of all management units, were performed under the supervision of professional engineers.

   TVA is currently revising our program to ensure that the supervision of all design, construction, and monitoring elements for all management units will be performed by professional engineers properly licensed in the states where the project is located and that have specific experience in dam design and operation.

5. **When did the company last assess or evaluate the safety (i.e., structural integrity) of the management unit(s)? Briefly describe the credentials of those conducting the structural integrity assessments/evaluations. Identify actions taken or planned by facility personnel as a result of these assessments or evaluations. If corrective actions were taken, briefly describe the credentials of those performing the corrective actions, whether they were company employees or contractors. If the company plans an assessment or evaluation in the future, when is it expected to occur?**

   Dates of the most recent facility inspection performed by the company or its consultant are listed in the attached table. These inspections were limited to surface observations. No intrusive sampling or testing, or engineering analyses were involved. Enclosed are the 2007-2008 inspection reports which were performed by TVA staff. All 2009 inspection reports are currently under review. These 2009 inspections were performed by TVA staff (who are experienced, degreed Civil Engineers, under the supervision of a registered professional engineer), with the exception of Cumberland, Shawnee, and Watts Bar (inactive) Fossil Plants, which were performed by Stantec.

   The most recent reviews at the Cumberland and Shawnee Fossil Plants were performed by Stantec. Stan Harris, PE, led those reviews. Mr. Harris has over 25 years experience in dam design, construction, and monitoring. In addition, Mr. Harris has experience leading dam safety training initiatives for the United States Army Corps of Engineers.

   Recommended corrective actions resulting from these evaluations are listed in the attached table. The corrective actions have been assigned to TVA staff or contractors experienced in general earth work construction and operation/construction of coal combustion disposal facilities.

TVA has retained the services of a third party consultant, Stantec, to assess each coal combustion byproducts storage facility at the eleven (11) active and one (1) inactive fossil plant. The assessments include field reconnaissance and records review for each facility. Reports will include recommendations and a priority list for additional geotechnical and engineering evaluations, if necessary. The study is on-going with results expected by the end of April 2009.

As a part of this study, TVA has initiated geotechnical explorations of the gypsum stack at our Paradise Fossil Plant, the ash pond at our Johnsonville Fossil Plant, the gypsum stack and ash dredge cell at our Widows Creek Fossil Plant, the ash disposal facility at our John Sevier Fossil Plant, and the gypsum stack and ash stack at our Cumberland Fossil Plant.

6. **When did a State or Federal regulatory official inspect or evaluate the safety (structural integrity) of the management unit(s)? If you are aware of a planned state or federal inspection or evaluation in the future, when is it expected to occur? Please identify the Federal or State regulatory agency or department which conducted or is planning the inspection or evaluation. Please provide a copy of the most recent official inspection report or evaluation.**

   TVA facilities are subject to regulation by state agencies responsible for permitting solid waste disposal and discharging of process or storm water flows. These state agencies do perform field reviews; however TVA facilities are not subject to regulation by state agencies relative to dam safety permitting and have not been subject to review or inspections by any federal regulatory agency. Copies of the most recent issued inspection report are enclosed for the 2007-2008 time period.

7. **Have assessments or evaluations, or inspections conducted by Federal regulatory officials conducted within the past year uncovered a safety issue(s) with the management unit(s), and, if so, describe the actions that have been or are being taken to deal with the issue or issues. Please provide any documentation that you have for these actions.**

   TVA facilities are subject to regulation by state agencies responsible for permitting solid waste disposal and discharging of process or storm water flows. These state agencies do perform field reviews however; TVA facilities are not subject to regulation by state or federal regulatory agencies relative to dam safety permitting and have not been subject to review or inspections. Copies of the most recent issued inspection report are enclosed for the 2007-2008 time period.

   Primarily maintenance issues were identified *during the most recent inspections. A summary of items identified are provided in the attached table. TVA is currently preparing work orders to address these items. The work will be performed by TVA staff or contractors experienced in earth work and the operation of coal combustion product disposal facilities.*

8. **What is the surface area (acres) and total storage capacity of each of the management units? What is the volume of materials currently stored in each of the management unit(s)? Please provide the date that the volume measurement(s) was taken. Please provide the maximum height of the management unit(s). The basis for determining maximum height is explained later in this Enclosure.**

   The surface area, total storage capacity, volume of materials currently stored, and date of last volume measurement for each management unit are provided in the attached table. Data based on 2006 long-range plans of the projected remaining capacities ending at Fiscal Year 2008.

9. **Please provide a brief history of known spills or unpermitted releases from the unit within the last ten years, whether or not these were reported to State or federal regulatory agencies. For purposes of this question, please include only releases to surface water or to the land (do not include releases to groundwater).**

   A history of known spills or unpermitted releases from each unit within the last ten (10) years, if applicable, is listed in the attached table. All spills and unpermitted releases were reported to the appropriate state or federal agencies as required by regulation or law.

10. **Please identify all current legal owner(s) and operator(s) at the facility.**

    The United States is the owner of TVA facilities, and TVA is the operator of each facility listed in the attached table.

PLANT	FACILITY	HAZARD POTENTIAL CLASSIFICATION (See footnote [illegible])	Hazard Rating Performed By	YR MGT UNIT COMMISSIONED (See footnote [illegible])	YR MGT UNIT EXPANDED	MATERIALS CONTAINED IN UNIT	LAST TVA ASSESSMENT	NEXT SCHEDULED TVA ANNUAL INSPECTION	ACTIONS TAKEN OR PLANNED RESULTING FROM LAST ANNUAL INSPECTION	ISSUES REPORTED BY STATE OR FEDERAL ASSESSMENTS AND ACTIONS TAKEN (See footnote [illegible])	SURFACE AREA (ACRES)	TOTAL STORAGE CAPACITY (Cubic Yards)	CURRENT VOLUME OF MATERIAL (Cubic Yards)	REMAINING CAPACITY (Cubic Yards)	DATE VOLUME TAKEN	CURRENT HEIGHT (FT)	FUTURE MAX. HEIGHT (FT)	KNOWN SPILLS OR UNPERMITTED RELEASES (SURFACE WATER/LAND) (See footnote [illegible])	CURRENT LEGAL OWNER(S) & OPERATOR(S) AT FACILITY
Allen Fossil Plant	EAST ASH DISPOSAL	LOW	TVA	1967	1978	FLY ASH	Nov-08	2009	Maintenance concerns such as rutting, erosion, vegetation, etc., were noted; a seep was noted north of the plant - TVA has an independent consultant evaluating the seep.	NR	70	1,775,000	1,029,000	746,000	2008	20	20	NR	Owner - United States, Operator - TVA
	EAST ASH STILLING POND	Not Rated		1978	Not Expanded	Fly ash, bottom ash					23	290,000	INCLUDED IN EAST ASH DISPOSAL AREA	INCLUDED IN EAST ASH DISPOSAL AREA	2008	20	20		
Bull Run Fossil Plant	DRY FLY ASH DISPOSAL AREA	Not Rated		1982 - Phase 1	1990 - Phase II	FLY ASH	Nov-08	2009	(1) Work order written to regrade top of Bottom Ash Stack, (2) work order for regrading and placement of rip rap below drainage pipe erosion on east side of Bottom Ash Stack, (3) work orders written for numerous animal paths and burrows noted around Bottom Ash Stack and Active Fly Ash Pond Area 2, (4) work order for repair of erosion areas along the bank of Bull Run Creek on south side of Active Fly Ash Pond Area 2, (5) removal of fallen trees on west side of Area 2 Stilling Pond and north side of Gypsum Disposal Area 2A, (6) work order written to repair eroded area on south slope of Gypsum Disposal Area 2A.	NR	17 (Phase II)	4,800,000.00	3,803,000	897000	2008	80	84	NR	Owner - United States, Operator - [illegible]
	FLY ASH POND AND STILLING BASIN AREA 2	LOW	TVA	1967	1976 - divider dike constructed to form Stilling Pond 1981 - dike constructed to form Pond 2A	Fly ash, bottom ash					49	2,700,000	2,332,600	367,400	2008	20	20		
	BOTTOM ASH DISPOSAL AREA 1	Not Rated		1967	1980 - Dike constructed to form stacking area within former pond	BOTTOM ASH (flows to Fly Ash Pond)					32	876,500	627,000	250,000	2008	52	85		
	GYPSUM DISPOSAL AREA 2A	Not Rated		1981 (originally fly ash settlement pond) 2008 (Gypsum Disposal Area)	Not Expanded	FLUE GAS EMISSION CONTROL RESIDUALS (Flows to fly ash pond)					42	2,743,000	896,000	1,847,000	2006	46	165		
Colbert Fossil Plant	DISPOSAL AREA 5	LOW	TVA	1983	1990 - converted to dry ash operation	FLY ASH, potentially ammonated.	Mar-08	2009	Disposal Area 5 - reported annual maintenance items include: cover and vegetate stack slopes semi-annually, repair erosion as needed, regrade perimeter ditch as needed. Ash Pond 4 - joint sealant applied to RCP spillway riser joints annually, semi-annual mowing of dike slopes, reportedly applied tree killer substance to sparse trees on west side of pond last year (trees not yet removed though), weekly monitoring of seepage areas.	NR	75	8,800,000	6,765,000	2,035,000	2006	120	135	NR	Owner - United States, Operator - TVA
	ASH POND 4	LOW	TVA	1972	1984	Bottom ash, fly ash (historical)					45	2,200,000	1,159,000	1,041,000	2006	40	40		
	DISPOSAL AREA 5 BASIN	Not Rated		1983	N/A	Fly Ash					12	600,000	150,000	450,000	2006	17	17		
Cumberland Fossil Plant	DRY ASH STACK	Not Rated		1968	Dry Ash stacking began in mid- 1990s over old pond	FLY ASH/BOTTOM ASH	Feb-09	2009	Maintenance activities needed include repairs for erosion, monitoring seepage, tree removal, clearing and cleaning inner slopes and perimeter ditches, repair of animal burrows, establishing vegetation in exposed areas, and recommendations for construction of the current gypsum dikes.	NR	110	12,600,000	4,781,000	7,818,000	2008	35	200	NR	Owner - United States, Operator - TVA
	ASH POND	LOW	TVA	1968	Dikes raised in 1979	Bottom ash, fly ash (historical)					50	2,000,000	1,305,000	695,000	2006	35	35		
	GYPSUM STORAGE AREA	LOW	TVA	1968	Gypsum area constructed over old pond in mid 1990s	FLUE GAS EMISSION CONTROL RESIDUALS					170	20,000,000	1,826,000	18,174,000	2006	60	140		
allatin Fossil Plant	FLY ASH POND E	LOW	TVA	1970	1986 - Divider Dike Constructed Forming Ponds A and E, 2008 - Pond E Expanded	FLY ASH, bottom ash, E flows to C.	2008	2009	Annual maintenance items reported by GAF include: annual seeding of new dikes for Pond E, mow along Pond E dike slopes beneath power lines along river.	NR	157	7,100,000	4,968,000	2,132,000	2008	30	35	NR	Owner - United States, Operator - TVA
	BOTTOM ASH POND A	LOW	TVA	1970	1986 - Divider Dike Constructed Forming Ponds A and E, 1994 - Divider dike raised	BOTTOM ASH, A flows to B					269	7,083,000	4,951,409	2,131,591	2008	25	25		
	STILLING POND B, C & D	Not Rated		1970	1986 - Ponds B and C formed when divider dike constructed to form Ash Ponds A and E	FLY ASH & BOTTOM ASH and other listed in E.					55	600,000	400,000	200,000	2008	10	10		
ohn Sevier Fossil Plant	DRY ASH STACK	Not Rated		1955 (former ash ponds)	1979 - all sluicing stopped, designated for dry ash disposal	FLY ASH	Nov-07	2009	(A) to monitor the exterior dikes slopes and toe areas of all disposal areas for surface sloughs, new seepage area, changes in existing seeps, or movements; (B) continuation of mowing program and prevention of tree growth on dikes; (C) cover exposed slopes with earth, seed, fertilize and mulch as described in the operations manual, (D) removal of sediment from Coal Yard Drainage basin; (E) reclaim animal burrows.	NR	84	3,800,000	2,098,000	1,702,000	2006	101	143	NR	Owner - United States, Operator - TVA
	BOTTOM ASH POND	LOW	TVA	1979	Not Expanded	BOTTOM ASH, FLY ASH					26 (pond area only) 41 (total area)	1,200,000	1,035,293	165,000	2008	25	25		

PLANT	FACILITY	HAZARD POTENTIAL CLASSIFICATION (See footnote #1)	Hazard Rating Performed By	YR MGT UNIT COMMISSIONED (See footnote #2)	YR MGT UNIT EXPANDED	MATERIALS CONTAINED IN UNIT	LAST TVA ASSESSMENT	NEXT SCHEDULED TVA ANNUAL INSPECTION	ACTIONS TAKEN OR PLANNED RESULTING FROM LAST ANNUAL INSPECTION	ISSUES REPORTED BY STATE OR FEDERAL ASSESSMENTS AND ACTIONS TAKEN (See footnote #3)	SURFACE AREA (ACRES)	TOTAL STORAGE CAPACITY (Cubic Yards)	CURRENT VOLUME OF MATERIAL (Cubic Yards)	REMAINING CAPACITY (Cubic Yards)	DATE VOLUME TAKEN	CURRENT HEIGHT (FT)	FUTURE MAX. HEIGHT (FT)	KNOWN SPILLS OR UNPERMITTED RELEASES (SURFACE WATER/LAND) (See footnote #4)	CURRENT LEGAL OWNER(S) & OPERATOR(S) AT FACILITY
Johnsonville Fossil Plant	ASH DISPOSAL AREA 2	LOW	TVA	1970	1978	FLY ASH & BOTTOM ASH	Nov-07	2009	Recommendations include maintenance activities: filling animal burrows, repairing erosion, filling in depressed areas, clearing heavy vegetation, and tree removal. Additionally, also monitoring seepage.	NR	97	4,360,000	4,164,000	196000	2006	30	30	Reported release of small quantity of cenospheres on March 27, 2004 when discharge structure was disturbed during maintenance.	Owner - United States, Operator - TVA
Kingston Fossil Plant	MAIN ASH POND	LOW	TVA	1951	1968 - raised dike	FLY ASH & BOTTOM ASH	Oct-08	2009	Standard recommendations were to repair all erosion ditches, repair wheel ruts, remove floating ash from the pond to prevent a permit violation, remove trees from dikes and mow the dikes regularly to control the growth of vegetation. Repair broken monitoring wells along Swan Pond Road, monitor seeps and under drains.	NR	92	14,370,000	UNKNOWN	UNKNOWN	NA	50	UNKNOWN	November 7, 2003 and November 1, 2006, an ash release occurred to land from a slough in the Dredge Cell embankment. A release into the Emory River occurred on December 22, 2008 from the Dredge Cell embankment failure. No reports found of releases from the Main Ash Pond or Stilling Basin.	Owner - United States, Operator - TVA
	STILLING POND	LOW See footnote 1	TVA	1978	Not Expanded	Materials from main ash pond					29	468,000	260,000	208,000	2006	50	50		
Paradise Fossil Plant	SCRUBBER SLUDGE COMPLEX (Gypsum Stack and Scrubber Sludge Stilling Pond)	LOW	TVA	1986	Not Expanded	FLY ASH, FLUE GAS EMISSION CONTROL RESIDUALS	Oct-08	2009	With respect to dam safety, primarily minor concerns (rutting, erosion, vegetation, etc.) were identified in the report. The under drain ditch at the Gypsum Stack needs to be cleaned out to prevent flow over the road. Several seeps were noted at the Daniel Run Pond 3, but were not flowing. Recommended removal of fines from the Coal Yard Runoff Ponds and all of the Red Water Ponds.	NR	255	858,000	11,783,000	35,074,000	2006	62	270	NR	Owner - United States, Operator - TVA
	FLY ASH EXTENSION AREA POND (Peabody Ash and Stilling Pond and Jacob's Creek Fly Ash and Stilling Pond)	LOW	TVA	1971	1997	FLY ASH					203	6,348,000	2,956,000	3,392,000	2006	34	34		
	SLAG AREAS 2A & 2B	LOW	TVA	1967	1970	BOTTOM ASH. A portion of the flow is routed to the fly ash extension area pond.					27	968,000	752,000	216,000	2006	24	24		
Shawnee Fossil Plant	CONSOLIDATED WASTE DRY STACK			1984	Horizontal expansion design prepared in 2000	FLY ASH/BOTTOM ASH, Drains to ash pond	Feb-09	2009	Maintenance activities needed include repairs for erosion, monitoring seepage, tree removal, clearing and clearing inner slopes, repair of animal burrows, establishing vegetation in exposed areas, monitoring animal paths, repairing leaking raw water line, removing sediment build-up, and recommendations for regrading intake channel dredge cell.	NR	200	33,194,000	22,811,000	10,382,000	2006	100	270	NR	Owner - United States, Operator - TVA
	ASH POND	LOW	TVA	1952	Area 2 was constructed in 1971 and the dikes were raised in 1979	FLY ASH/BOTTOM ASH					180	5,000,000	4,712,000	287,000	2006	25	25		
Widows Creek Fossil Plant	ASH POND (Complex consists of Bottom Ash Stack, Iron Pond, Cooper Pond, Old Scrubber Sludge Pond (Dredge Cell), Asbestos Waste Disposal Area, Pump Pond, Upper and Lower Stilling Ponds)	LOW	TVA	1950	During 2005, a dredge cell was constructed over the old scrubber sludge pond area. During 2007 dredging ceased.	FLUE GAS EMISSION CONTROL RESIDUALS, FLY ASH & BOTTOM ASH	Oct-08	2009	Review with the Constructer the Gypsum Stack operations manual and drawings to ensure the operations continue in accordance with the current stacking plan, monitor the wet area along the southern lower perimeter dike, rework a portion of the west slope next to the Stilling Pond, install sub drains on the west slope adjacent to the Gypsum Stilling Pond, uncover the slope drains on the 650/655 bench and grade per design drawings. In regards to the Wet Gypsum Stacking Stilling Pond, the planned actions are to consider and alternate skimmer design on TVA drawing 10W235-19. In regards to the Pump Pond, the planned actions are to monitor the seep in the dike between the Stilling Pond and the Pump Pond. In regards to the Active Ash Pond, the planned actions are to monitor the seepage along the south Perimeter dike next to the stilling pond.	NR	310	18,890,000	1,856,000	17,034,000	2006	50	115	Reported release of small quantity of cenospheres from the Ash Pond which occurred on December 10, 2004 due to intense precipitation. Reported release of small quantity of cenospheres from the Ash Pond which occurred on January 30, 2006. An abandoned decant weir in Pond 2B of the Gypsum Stack failed on January 9, 2009.	Owner - United States, Operator - TVA
	GYPSUM STACK (Wet Stacking Area)	LOW	TVA	1986	Phase I vertical expansion occurred from 1986 to 1992. Phase II horizontal expansion began in 1992.	FLUE GAS EMISSION CONTROL RESIDUALS, FLY ASH & BOTTOM ASH					110	17,683,000	7,892,000	9,791,000	2006	75	150		
Watts Bar Fossil Plant (Inactive)	ASH POND and STILLING BASIN	LOW	TVA	1974	1977	Previous fly ash, bottom ash	Feb-09	2009	Complete Closure Plan - currently construction is approximately 95 percent complete.	NR	14	230,000	150000	80,000	2006	30	30	NR	Owner - United States, Operator - TVA

Notes: 1. Hazard Potential listed for those facilities previously rated by TVA, all facilities are currently under evaluation. Based on hindsight at Kingston Fossil Plant, the ranking did not adequately represent the actual risk experienced on 12/22/2008.
2. Year Management Unit Commissioned approximated from available reports, drawings, or permit documents.
3. NR - None Reported
4. Does not include NPDES permit exceedences

Evaluation of Fossil Coal Combustion Products (CCP) Facilities for Dam Safety Hazard Classification

TVA has performed a preliminary evaluation to classify coal combustion storage facilities in accordance with FEMA's Hazard Potential Classification System for Dams. These guidelines evaluate the consequences of a potential failure not the likelihood of a failure. Guidelines that were developed and utilized are included below. These results have been reviewed with Stantec, who have been contracted by TVA to assess all of the coal combustion storage areas. Stantec's detailed analysis could change these preliminary conclusions.

<u>Definitions of a Dam - Federal Guidelines for Dam Safety</u> (FEMA 93 issued June 1979)

Any artificial barrier, including appurtenant works, which impounds or diverts water, and which (1) is twenty-five feet or more in height from the natural bed of the stream or watercourse measured at the downstream toe of the barrier or from the lowest elevation of the outside limit of the barrier if it is not across a stream channel or watercourse, to the maximum water storage elevation or (2) has an impounding capacity at maximum water storage elevation of fifty acre-feet or more. These guidelines do not apply to any such barrier which is not in excess of six feet in height regardless of storage capacity, or which has a storage capacity at maximum water storage elevation not in excess of fifteen acre-feet regardless of height. This lower size limitation should be waived if there is a potentially significant downstream hazard.

In addition to conventional structures, this definition of "dam" specifically includes "tailings dams," embankments built by waste products disposal and retaining a disposal pond.

<u>TVA notes</u>: Expand "tailings dams" definition to include wet coal-combustion by-product storage facilities. Dry stack storage areas are classified as a "dry stack" and not evaluated because they do not have dikes or impound water. Classifications of active structures will be based on current conditions (height/storage). Inactive ash ponds/dredge cells will not be reviewed since they are either inactive or closed, and they are no longer impounding water since the impounded water decreases every year once they are inactive. Classifications will be re-evaluated every five years or sooner if conditions change.

<u>Hazard Potential Classification Systems for Dams</u> (FEMA 333 Issued April 2004)

1. **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.
2. **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.
3. **High Hazard Potential**
   Dams assigned the high hazard potential classification are those where failure or misoperation will probably cause loss of human life.

Hazard Potential Classification	Potential Loss of Human Life	Potential 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 this classification)

Guidelines for TVA Evaluating FEMA Criteria below:

Hazard Potential Classification	Potential Loss of Human Life	Potential Environmental Impact	Potential Economic and Infrastructure Lifeline Losses
Low	0	Contained on TVA property or minimal off-property impact	No expected damages to public roads, powerlines, etc.
Significant	0	Off TVA property, may enter waters of the U.S.	Expected damages to public roads, powerlines, etc.
High	1 or more		

Facility	Overall Rating
Allen	
East Ash Disposal & East Ash Stilling Pond	Significant
Bull Run	
Dry Fly Ash Stack (Not Rated)	N/A
Fly Ash Pond and Stilling Basin Area 2	High (Impact: Housing)
Bottom Ash Disposal Area 1	Significant
Gypsum Disposal Area 2A	Low
Colbert	
Disposal Area 5 (Not Rated)	N/A
Ash Pond 4	High (Impact: Highway/Housing)
Disposal Area 5 Basin	Significant

Facility	Overall Rating
Cumberland	
Dry Ash Stack (Not Rated)	N/A
Ash Pond	High (Impact: Highway)
Gypsum Storage Area	High (Impact: Industrial)
Gallatin	
Fly Ash Pond E	Significant
Bottom Ash Pond A	Significant
Stilling Pond B, C & D	Significant
John Sevier	
Dry Ash Stack (Not Rated)	N/A
Bottom Ash Pond	Significant
Johnsonville	
Ash Disposal Area 2	Significant
Kingston	
Main Ash Pond	Significant
Stilling Pond	Significant
Paradise	
Scrubber Sludge Complex	Low
Fly Ash Extension Area	Low
Slag Areas 2A & 2B	Low
Shawnee	
Consolidated Waste Dry Stack (Not Rated)	N/A
Ash Pond	Significant
Widows Creek	
Ash Pond	Significant
Gypsum Stack	High (Impact: Housing)
Watts Bar	
Ash Pond and Stilling Basin	Significant

EPA believes that the information requested is essential to an evaluation of the threat of releases of pollutants or contaminants from the units. The provisions of Section 104 of CERCLA authorize EPA to pursue penalties for failure to comply with or respond adequately to an information request under Section 104(c). In addition, providing false, fictitious or fraudulent statements or representations may subject you to criminal penalties under 18 U.S.C. 1001.

Your response must include the following certification signed and dated by an authorized representative of Tennessee Valley Authority.

I certify that the information contained in this response to EPA's request for information and the accompanying documents is true, accurate, and complete. As to the identified portions of this response for which I cannot personally verify their accuracy, I certify under penalty of law that this response and all attachments were prepared in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fines and imprisonment for knowing violations.

Signature: [signature]

Name: John C Kammeyer

Title: VP, CCP Engineering & Projects

This request has been reviewed and approved by the Office of Management and Budget pursuant to the Paperwork Reduction Act, 44 U.S.C., 3501-3520.

Please send your reply to:

Mr. Richard Kinch
U.S. Environmental Protection Agency (5306P)
1200 Pennsylvania Avenue Southwest
Washington, DC 20460

If you are using overnight or hand-delivery mail, please use the following address:

Mr. Richard Kinch
U.S. Environmental Protection Agency
Two Potomac Yard
2733 South Crystal Drive
5th Floor, N-5783
Arlington, Virginia 22202-2733

Tennessee Valley Authority, 400 W. Summit Hill Drive, Knoxville, Tennessee 37902

Anda A. Ray
Senior Vice President, Environment and Technology
and Environmental Executive

October 22, 2010

Mr. Richard Kinch
U.S. Environmental Protection Agency
Two Potomac Yard
2733 South Crystal Drive
5th Floor: N-5783
Arlington, Virginia 22202-2733

Dear Mr. Kinch:

On July 16, 2009, the Tennessee Valley Authority (TVA) provided preliminary hazard ratings for our wet coal combustion impoundments. We also indicated that we had hired an engineering firm, Stantec, to inspect, test and make recommendations. Stantec has completed a more detailed assessment of our impoundments designated as "high hazard". As a result, four of the five impoundments classified as "high hazard" have been reduced to "significant hazard". Accordingly, we have amended our previous information and enclosed the recent reevaluation information.

If you have questions, please do not hesitate to contact me at (865) 632-8511.

Sincerely,

Anda A. Ray

Enclosures

Evaluation of Fossil Coal Combustion Products (CCP) Facilities for Dam Safety Hazard Classification

TVA performed a preliminary evaluation to classify coal combustion storage facilities in accordance with FEMA's Hazard Potential Classification System for Dams in 2009. These guidelines evaluated the consequences of a potential failure not the likelihood of a failure. Guidelines that were developed and utilized are included below. The preliminary results were reviewed, updated and finalized by Stantec, who was contracted by TVA to assess all of the coal combustion storage areas. In 2010, Stantec performed a more detailed analysis that changed some of the initial classifications. The Facility Ratings listed at the end of this document, reflect the changed classificiations.

<u>Definitions of a Dam - Federal Guidelines for Dam Safety</u> (FEMA 93 issued June 1979)

Any artificial barrier, including appurtenant works, which impounds or diverts water, and which (1) is twenty-five feet or more in height from the natural bed of the stream or watercourse measured at the downstream toe of the barrier or from the lowest elevation of the outside limit of the barrier if it is not across a stream channel or watercourse, to the maximum water storage elevation or (2) has an impounding capacity at maximum water storage elevation of fifty acre-feet or more. These guidelines do not apply to any such barrier which is not in excess of six feet in height regardless of storage capacity, or which has a storage capacity at maximum water storage elevation not in excess of fifteen acre-feet regardless of height. This lower size limitation should be waived if there is a potentially significant downstream hazard.

In addition to conventional structures, this definition of "dam" specifically includes "tailings dams," embankments built by waste products disposal and retaining a disposal pond.

<u>TVA notes</u>: Expand "tailings dams" definition to include wet coal combustion byproduct storage facilities. Dry stack storage areas are classified as a "dry stack" and not evaluated because they do not have dikes or impound water. Classifications of active structures will be based on current conditions (height/storage). Inactive ash ponds/dredge cells will not be reviewed since they are either inactive or closed, and they are no longer impounding water since the impounded water decreases every year once they are inactive. Classifications will be re-evaluated every five years or sooner if conditions change.

<u>Hazard Potential Classification Systems for Dams</u> (FEMA 333 issued April 2004)

1. **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.
2. **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.

3. **High Hazard Potential**
Dams assigned the high hazard potential classification are those where failure or misoperation will probably cause loss of human life

Hazard Potential Classification	Potential Loss of Human Life	Potential 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 this classification)

Guidelines for TVA Evaluating FEMA Criteria below:

Hazard Potential Classification	Potential Loss of Human Life	Potential Environmental Impact	Potential Economic and Infrastructure Lifeline Losses
Low	0	Contained on TVA property or minimal off-property impact	No expected damages to public roads, powerlines, etc.
Significant	0	Off TVA property, may enter waters of the U.S.	Expected damages to public roads, powerlines, etc.
High	1 or more		

Facility	Overall Rating
Allen	
East Ash Disposal & East Ash Stilling Pond	Significant
Bull Run	
Dry Fly Ash Stack (Not Rated)	N/A
Fly Ash Pond and Stilling Basin Area 2	Significant
Bottom Ash Disposal Area 1	Significant
Gypsum Disposal Area 2A	Low
Colbert	
Disposal Area 5 (Not Rated)	N/A
Ash Pond 4	Significant
Disposal Area 5 Basin	Significant

Facility	Overall Rating
Cumberland	
Dry Ash Stack (Not Rated)	N/A
Ash Pond	High (Impact: Highway)
Gypsum Storage Area	Significant
Gallatin	
Fly Ash Pond E	Significant
Bottom Ash Pond A	Significant
Stilling Pond B, C & D	Significant
John Sevier	
Dry Ash Stack (Not Rated)	N/A
Bottom Ash Pond	Significant
Johnsonville	
Ash Disposal Area 2	Significant
Kingston	
Main Ash Pond	Significant
Stilling Pond	Significant
Paradise	
Scrubber Sludge Complex	Low
Fly Ash Extension Area	Low
Slag Areas 2A & 2B	Low
Shawnee	
Consolidated Waste Dry Stack (Not Rated)	N/A
Ash Pond	Significant
Widows Creek	
Ash Pond	Significant
Gypsum Stack	Significant
Watts Bar	
Ash Pond and Stilling Basin	Significant

APPENDIX A

Document 3

Stantec 2009 TVA Disposal Facility Assessment

1. General Facility Information

Facility Status:	Inactive	**NID Identification:**	Not Available
Surface Area (inside dikes)	14.3 Acres	**Maximum Height (toe to top of dike):**	30
Free Water Volume:	5 Acre-feet	**Maximum Water Storage:**	N/A
Estimated CCB Storage:	N/A	**Dike Length:**	N/A
Plant Discharge to Facility:	N/A	**Current Pool Elevation:**	N/A

2. Site Visit Information

Stantec Assessment Team: Robert Fuller, PE
Benjamin Phillips, EIT

TVA Staff Present: Steve Williams (Exit Interview)

Field Assessment Dates: January 16, 2009.

Weather/Site Conditions: Sunny, Cold.

3. History/Description of Usage

History and Operation: Built in 1974 for sluiced disposal. Bottom ash dike constructed in 1976 to form stilling pond portion.

Past Failures/Releases: None documented.

4. Owner's Operations, Maintenance and Inspection Information

Emergency Action Plan: N/A

Operations Manual: N/A

TVA Maintenance: Mowing.

TVA Inspections: Annually.

Problems Previously Identified During Past TVA Inspections:	Minor seepage along toe of embankment. Erosion along riverbank adjacent to toe and at spillway outfall.

5. Documents Reviewed

See attached Document Log for complete list of documents provided by TVA for review. In particular, the following provided pertinent information for the assessment of this facility:

TVA Design Drawings:	10N245
TVA As-Built Drawings:	No drawings were identified as "As-Built".
TVA Construction Testing Records:	N/A
TVA Annual Inspection Reports:	1974 -2008
Geotechnical Data:	N/A

6. Stantec Field Observations

See attached Concerns/Photo Log, Photos, and Site Plan Drawing.

6.1. Interior Slopes

Vegetation:	None.
Trees:	None.
Wave Wash Protection:	N/A
Erosion:	None observed.
Instabilities:	None observed.
Animal Burrows:	None observed.
Freeboard:	**Measured:** 5 feet **Design:** N/A
Encroachments:	None
Slope:	**Measured:** N/A **Design:** N/A

6.2. Crest

Crest Cover and Slope:	Grass, bottom ash, gravel, and asphalt pavement.
Erosion:	None observed.
Alignment:	Consistent.
Settlement/Cracking:	None observed.
Bare Spots/Rutting:	Minor.
Width:	**Measured:** N/A **Design:** N/A

6.3. Exterior Slopes

Vegetation:	Grass and some woody growth.
Trees:	Yes.
Erosion:	Minor.
Instabilities:	None observed.
Uniform Appearance:	Yes.
Seepage:	None observed
Benches:	None observed.
Foundations, Drains, Relief Wells, Instrumentation:	Well covers were observed on eastern side of the embankment.
Animal Burrows:	None observed.
Slope:	**Measured:** N/A **Design:** N/A
Height:	**Measured:** N/A **Design:** N/A

6.4. Spillway Weirs/Riser Inlets

Number:	3
Size, Type and Material:	CMP

Height of Riser Inlets:	N/A
Access:	Deteriorated metal access bridge.
Joints:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	N/A

6.5. Outlet Pipes

Number:	3
Size, Type and Material:	Concrete.
Headwall:	Yes
Joint Separations:	N/A
Mis-Alignment:	N/A
Closed/Abandoned Conduits:	N/A

7. Notable Observations and Concerns

- Excess vegetation, some woody, on the embankment.
- Steep slopes on the embankment.
- Erosion on the riverbank below the embankment toe and at the spillway discharge into the river.

8. Recommendations

8.1. Phase 2 Engineering and Programmatic Recommendations

- It is recommended that a capacity analysis be performed for the Stilling Pond and Ash Pond to check if sufficient volume capacity exists for storm water runoff. Further hydraulic/hydrologic analysis may be needed pending outcome of capacity analysis.
- It is recommended that the submitted closure plan be completed.

8.2. Maintenance Recommendations

- Cut and maintain heavy/tall cattail and grass growth on interior slopes of ponds to allow better observation.
- Remove trees/brush from downstream dike slope of Stilling Pond and Ash Pond and outlet area.
- Continue to monitor areas associated with the closure plan.
- Continue annual dike and facility inspections.

Drawing Mark P-1-1 Excess vegetation on embankment crest and slopes.

Stantec

Concerns/Photo Log

Drawing Mark	Comments	Photo/GPS ID
P-1-1	Excess vegetation on embankment crest and slopes.	WBF-P-1-1

APPENDIX A

Document 4

CDM Smith January 2012 Report, Existing Conditions Stability Analysis

CDM Smith
5400 Glenwood Ave, Suite 300
Raleigh, North Carolina 27612
tel: 919 325-3500
fax: 919 781-5730

January 31, 2012

Mr. James D. Mullins, P.E.
Senior Program Manager
Tennessee Valley Authority
CCP Engineering
1101 Market Street, LP 5E-C
Chattanooga, TN 37402

Subject: Report
Existing Conditions Stability Analyses
Ash Pond Area at Watts Bar Fossil Plant

Dear Mr. Mullins:

The purpose of this letter report is to present the results of the existing conditions stability analyses performed by CDM Smith for the Ash/Stilling Pond area at the Watts Bar Fossil (WBF) plant near Spring City, Tennessee. These analyses were performed to support the U.S. Environmental Protection Agency's assessment of the Tennessee Valley Authority's (TVA) Coal Combustion Products (CCP) disposal facilities.

Project Background

The WBF plant is directly downstream of the Watts Bar Dam and Lock and abuts the west bank of the Tennessee River. Currently, the WBF plant is not operational and decommissioning is underway. The WBF plant was a coal-fired power plant built by TVA between 1940 and 1945. The plant was operated in two stages, from end of construction to 1957 and from 1970 to 1982. During the plant operation, ash and boiler slag generated by the plant were stockpiled and stored on-site.

In 2010, TVA contracted with CDM Smith to perform Phase I preliminary design services to support final closure of the WBF plant as part of the WBF Plant Coal Combustion Products Closure Project. The final closure encompasses multiple areas which include disposal facilities, impoundments, and stormwater ponds permitted in accordance with multiple regulations. The project includes closure of five (5) main areas: (i) the Borrow Source Area, (ii), Slag Processing Area (iii), Chemical Pond Area, (iv) Ash/Stilling Pond Area, and (v) Riverbank Area, as shown on **Figure 1**.

As part of this work, TVA requested that CDM Smith provide an existing condition evaluation for the stability of the Ash/Stilling Pond Area. This evaluation considered stability of the Ash/Stilling

Pond Area under static and seismic loading conditions based upon available data, as described herein.

Available Information

During the preliminary design phase for the closure, CDM Smith reviewed the following available information provided by TVA:

- MACTEC geotechnical report at Borrow Area
- QA/QC reports for closure construction at Chemical Pond and Slag Disposal Area during 2006 to 2009.
- TVA Disposal Facility Assessment, Phase I Plant Summary, Watts Bar Fossil Plant (WBF), by Stantec, 2009.
- Final Report – Development of Hazard Deaggregation Inputs for Use in Risk Analysis of Fossil Plants, by AMEC GeoMatrix, March 2010.
- Watts Bar Fossil Plant, Annual Inspection Report for Ash/Waste Disposal Areas, from 1967 through 2008.
- Watts Bar Fossil Plant, Slag Disposal Area Closure Plan, Project Planning Document, approved by TVA in January 2007.
- Fly Ash, Bottom Ash, and Scrubber Gypsum Study, by Law Engineering, November 1995.

In addition, CDM Smith performed the following site-specific investigations to supplement the available data:

- Site walk and surficial soil sampling in the Borrow Area and Slag Disposal Area in August 2011.
- Bulk sampling and laboratory testing of underwater ash samples from the Ash/Stilling Pond Area in August 2011.
- Site survey of the Slag Disposal Area and Ash/Stilling Pond Area in December of 2011. Survey was performed by TVA at the request of CDM Smith.

- Subsurface exploration program along west bank of Tennessee River consisting of three geotechnical borings and installation of two groundwater observation wells.

The subsurface exploration program was completed in January 2012 and draft boring logs and water level readings from the wells are currently available and included as **Attachment A**. Laboratory testing results for disturbed and undisturbed samples collected in these borings are not available at the time of this report.

Existing Conditions Evaluation

The existing conditions stability analyses for the Ash/Stilling Pond Area were performed at two critical cross-sections, as shown on **Figure 2**. The two critical cross-sections were selected at locations exhibiting the steepest exterior embankment slopes and riverbank slopes. The locations of the geotechnical borings completed in January 2012 are also shown on this figure. Cross-section A-A' extends through the Wet Ash Pond Area and Cross-Section B-B' extends through the Dry Ash Area, as shown on **Figures 3A** and **3B**, respectively. The cross-sections were developed based upon available topographic survey, design plans for the ponds, and the subsurface conditions encountered in the test borings. Currently there are no bathymetric survey data available for the river bank slope below normal water level. For this evaluation, the river bank slope was assumed to follow the same natural slope above water level and extend to the top of the bedrock at the river bed.

Selection of Design Parameters

The engineering design properties of the ash and soil layers for the seepage and stability analysis of the cross-sections are summarized in **Tables 1a and 1b**. The basis for selection of the design properties is also listed in the tables. In general, ash properties were estimated based upon available data from similar TVA facilities and soil properties were estimated based upon the field investigation data, empirical correlations, and experience in similar geologic conditions.

Table 1a: Parameters used in SEEP/W Seepage Analyses

Layer	Material	k_h		k_h / k_v	Basis of Parameter Selection
		ft/day	cm/sec		
	Ash Material	0.45	1.6E-04	20	Based on comparison between laboratory testing data for existing ash material and TVA's CCP material database
1	Fill	0.0028	1.0E-06	10	From Peck[(1)]; typical value for mixture of sand, clay, and silt.
2A	Medium Stiff to Stiff Clay	0.0014	5.0E-07	15	From Peck; typical value for low-permeability soil.
2B	Soft Clay and Silt	0.0014	5.0E-07	15	From Peck; typical value for low-permeability soil.
3	Sand	2.83	1.0E-03	4	From Peck; typical value for sand.
4	Weathered Rock and Gravel	28.35	1.0E-02	4	From Peck; typical value for sand and gravel mixtures.
5	Interbedded Shale and Limestone Bedrock	0.0006	2.0E-07	1	From Domenico[(2)]; page 39; high-end value for Shale bedrock.

Reference:

1. Ralph B. Peck, 'Foundation Engineering', 2nd edition, 1974; page 43.

2. Patrick A. Domenico, 'Physical and Chemical Hydrogeology', 2nd edition, 1997.

Table 1b: Strength Parameters used in SLOPE/W Stability Analyses

Layer	Material	Unit Weight, pcf	Friction Angle, degrees	Undrained Shear Strength, psf	Basis of Parameter Selection(1)
	Ash Material (wet)	70	20	-	Comparison between laboratory testing data for existing ash material and TVA's CCP material database
	Ash Material (dry)	85	25	-	
1	Fill	120/115(2)	32	-	Selected based on lower 1/3 N-values(3) from B-2 and B-3
2A	Medium Stiff to Stiff Clay	110/105(2)	29	1000	Selected based on lower 1/3 N-values(3) and pocket penetrometer readings(4) from B-2 and B-3
2B	Soft Clay and Silt	110	28	500	Selected based upon N-values and pocket penetrometer readings(4) from B-2 and B-3
3	Sand	120	30	-	Selected based on Lower 1/3 N-values(3) from B-2 and B-3
4	Weathered Rock and Gravel	125	40	-	Based upon experience in similar geologic conditions
5	Bedrock	Impenetrable			Assumed

Notes:

1. Correlation of N-value and friction angle from Ralph B. Peck, 'Foundation Engineering', 2nd edition, 1974; page 310.
2. Values listed are saturated/moist unit weights.
3. Lower 1/3 value is defined as the value where at least 2/3 of all the readings are greater or equal. N-value is defined as the sum of the blows to drive the 2nd and 3rd 6-inch-increments of each split spoon sample.
4. Pocket penetrometer readings were performed on split spoon samples and Shelby tube sample during drilling.

For stability analyses under seismic conditions, peak ground acceleration for the WBF plant site was selected based upon a review of the "*Final Report - Development of Hazard Deaggregation Inputs for Use in Risk Analysis of Fossil Plants",* by AMEC GeoMatrix, dated March 28, 2010 and the USGS 2008 Hazards Map available at http://earthquake.usgs.gov. **Table 2** summarizes the data from the three TVA plants closest to WBF and the USGS Hazard Map values for the WBF. Based upon these data, the peak ground acceleration on hard rock for WBF was interpolated to be 0.042g for a 500-year return period and a 0.116g for a 2500-year return period. Ground motion corresponding to a 500-year return period is consistent with seismic stability guidance provided by Tennessee's dam safety regulations Chapter 1200-5-7 for design of new dams. However, these regulations further recommend that facilities constructed prior to 2008 be designed to withstand seismic accelerations according to zones indicated on the "Geologic Hazards Map of Tennessee" by Robert A. Miller (1978). The WBF facility is located in Seismic Risk Zone 2, which corresponds to an acceleration of 0.05g. Since the 500-year return peak ground acceleration of 0.042g estimated for WBF is less than the value recommended based upon the Seismic Risk Zone, a seismic acceleration of 0.05g was used in the stability analyses.

Table 2: Summary of Available Seismic Hazards Results (AMEC Report and USGS)

Plant	Latitude	Longitude	Return Period (years)	Probability of Exceedance	PGA, (g)	
					AMEC Report	USGS
Bull Run	36.00	-84.15	2500	2% in 50 years	0.131	0.155
			500	10% in 50 years	0.043	0.044
Kingston	35.90	-84.51	2500	2% in 50 years	0.115	0.134
			500	10% in 50 years	0.041	0.041
Watts Bar	35.61	-84.78	2500	2% in 50 years	***0.116***	0.135
			500	10% in 50 years	***0.042***	0.042
Widows Creek	34.90	-85.75	2500	2% in 50 years	0.1	0.115
			500	10% in 50 years	0.038	0.038

Bolded values were interpolated from tabulated data.

Seepage Analyses and Results

The phreatic surface for each stability analysis was developed from seepage analyses performed with the SEEP/W 2007 software package by GEO-SLOPE International, Ltd. This computer program uses the inputted geometry, soil, rock, and ash properties, and boundary conditions (surface water and groundwater conditions) to develop a steady-state seepage profile. For our analyses, the model was calibrated using the field data gathered during the recent geotechnical investigations by CDM Smith including:

- Water levels observed in the Ash/Stilling Pond
- Groundwater levels measured in the observation wells
- River level elevation data available at "http://www.tva.gov/lakes/wbh_o.htm" for the dates/times.

Once the model was calibrated, the steady-state phreatic surface was developed for normal pool conditions in the Ash/Stilling Pond (EL. 705).

Static Slope Stability Analyses and Results

Analyses for overall (global) stability under static conditions were performed using the SLOPE/W 2007 modeling software package from GEO-SLOPE. This computer program uses the inputted slope geometry, soil, rock, and ash properties, and phreatic surface and calculates the factors of safety against deep-seated circular failures. Phreatic surfaces generated by SEEP/W were imported to SLOPE/W for the static and seismic slope stability analyses. The Spencer method was selected for the slope stability analyses. The minimum acceptable static factor of safety against overall slope failure is 1.5 for normal pool conditions.

Effective stress strength parameters were used for all materials in static analyses. The stability analyses are included in **Attachment B** and the minimum factors of safety for deep-seated circular failure surfaces are presented in **Table 3**. Failure surfaces less than 5 feet deep are considered to be sloughing/surficial failures. The stability analyses did exhibit some lower factors of safety for sloughing/surficial failures along the river bank, but these failure surfaces did not extend into the pond berm such that the global stability of the ash pond would be impacted. Results presented herein considered the deep-seated failures that extend into the ash pond areas only. All factors of safety for static conditions equal or exceed the minimum required.

Table 3: Results of Slope Stability Analyses – Static Conditions

Run #	Modeling Scenario	Calculated Factor of Safety	
		Inboard Slope	Outboard Slope
A-1	Static Slope Stability at Wet Pond	1.9	1.8
B-1	Static Slope Stability at Dry Ash Area	2.4	1.5

Seismic Slope Stability Analyses and Results

The stability analyses under seismic loading conditions were performed using a pseudostatic method, where the added inertial load from an earthquake is represented by a horizontal pseudostatic coefficient. Based upon the Standard Penetration Test N-values and fines content of the subsurface soils, the soils at the site are not considered to be susceptible to liquefaction. The analyses assumed no liquefaction of the subsurface soils and undrained shear strength parameters were used for the natural clay soils (Layer 2A and 2B). Peak ground surface acceleration was estimated as 0.05g. Tolerable deformations were assumed for cases where the pseudostatic factor of safety is greater than 1.0. Normal pool conditions were assumed (El. 705).

The stability analyses are included in Attachment B and the minimum factors of safety for deep-seated circular surfaces are presented in **Table 4**. All factors of safety for seismic (pseudostatic) conditions equal or exceed the minimum required.

Table 4: Results of Slope Stability Analyses – Seismic Conditions

Run #	Modeling Scenario	Calculated Factor of Safety	
		Inboard Slope	Outboard Slope
A-2	Seismic Conditions at Wet Pond	1.8	1.3
B-2	Seismic Conditions at Dry Ash Pond	2.2	1.1

Conclusions

The results of the analyses indicate acceptable factors of safety for both cross-sections through the Ash/Stilling Pond Area for static and seismic slope stability under existing conditions. The seismic slope stability analyses presented in this letter use a pseudostatic approach to represent existing conditions. For seismic assessment of the closure design, TVA will employ a comprehensive risk-based approach, with design and mitigation decisions based upon the probability and consequences of failure. This approach is outlined in the document *"Seismic Risk Assessment, Closed CCP Storage Facilities, Tennessee Valley Authority"* dated March, 2010 and included as **Attachment C**.

Limitations

This letter report has been prepared for specific application to the subject project in accordance with generally accepted geotechnical engineering practices. No other warranty, express or implied, is made. In the event that any changes occur, the conclusions and recommendations presented in this memorandum should not be considered valid, unless changes are reviewed and conclusions of this memorandum are modified or verified in writing.

Very truly yours,

Stephen L. Whiteside

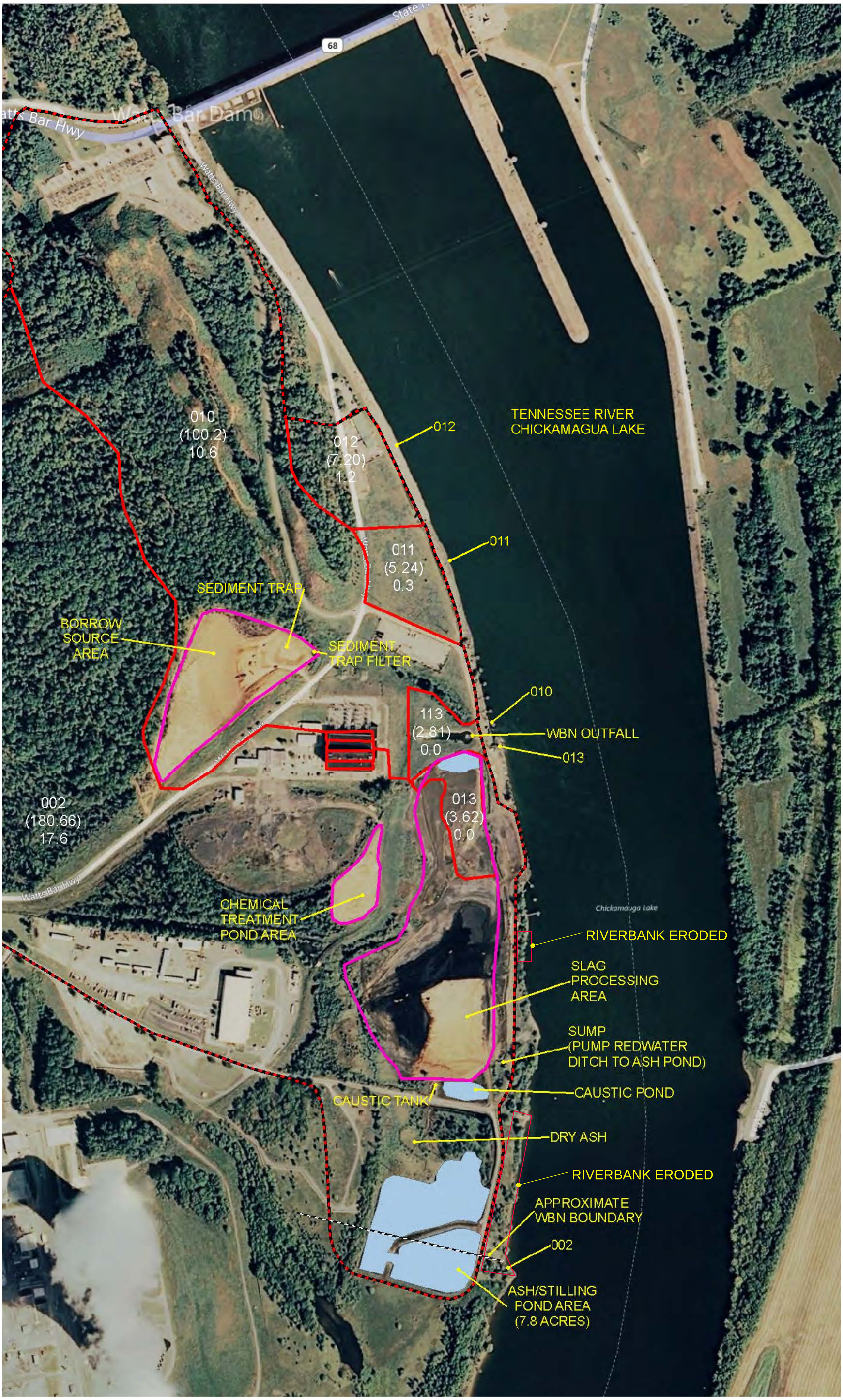
Stephen L. Whiteside, P.E.
Vice-President
CDM Smith Inc.

cc: Michael Bachand, CDM Smith
Jintao Wen, CDM Smith
Danielle Neamtu, CDM Smith

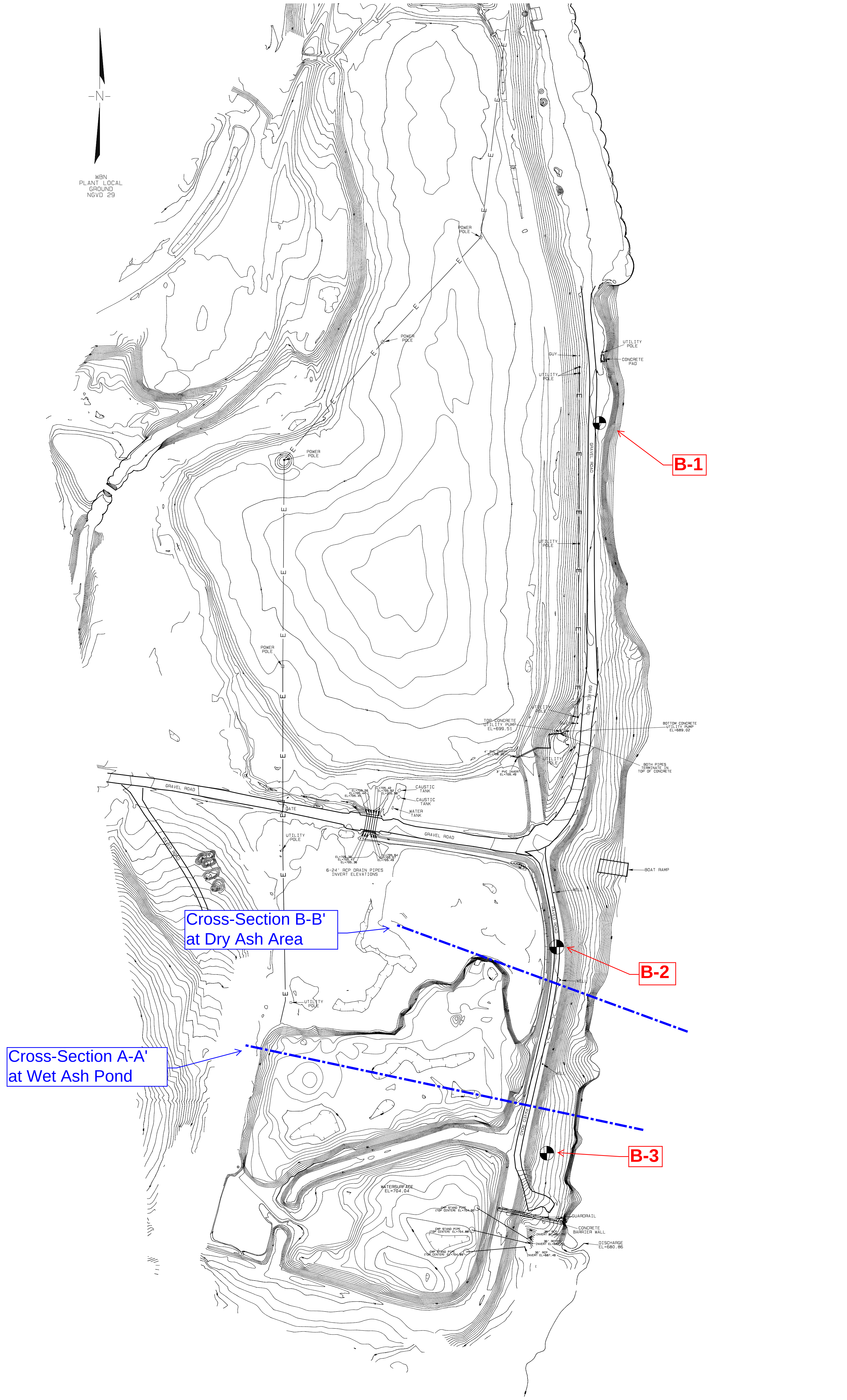
STEPHEN L. WHITESIDE
REGISTERED ENGINEER
AGRICULTURE
COMMERCE
No. 108556
STATE OF TENNESSEE

1/31/12

FIGURES



CDM Smith	Watts Bar Fossil Plant Tennessee Valley Authority Coal Combustion Products Closure Project	Locations of Closure Areas Figure 1

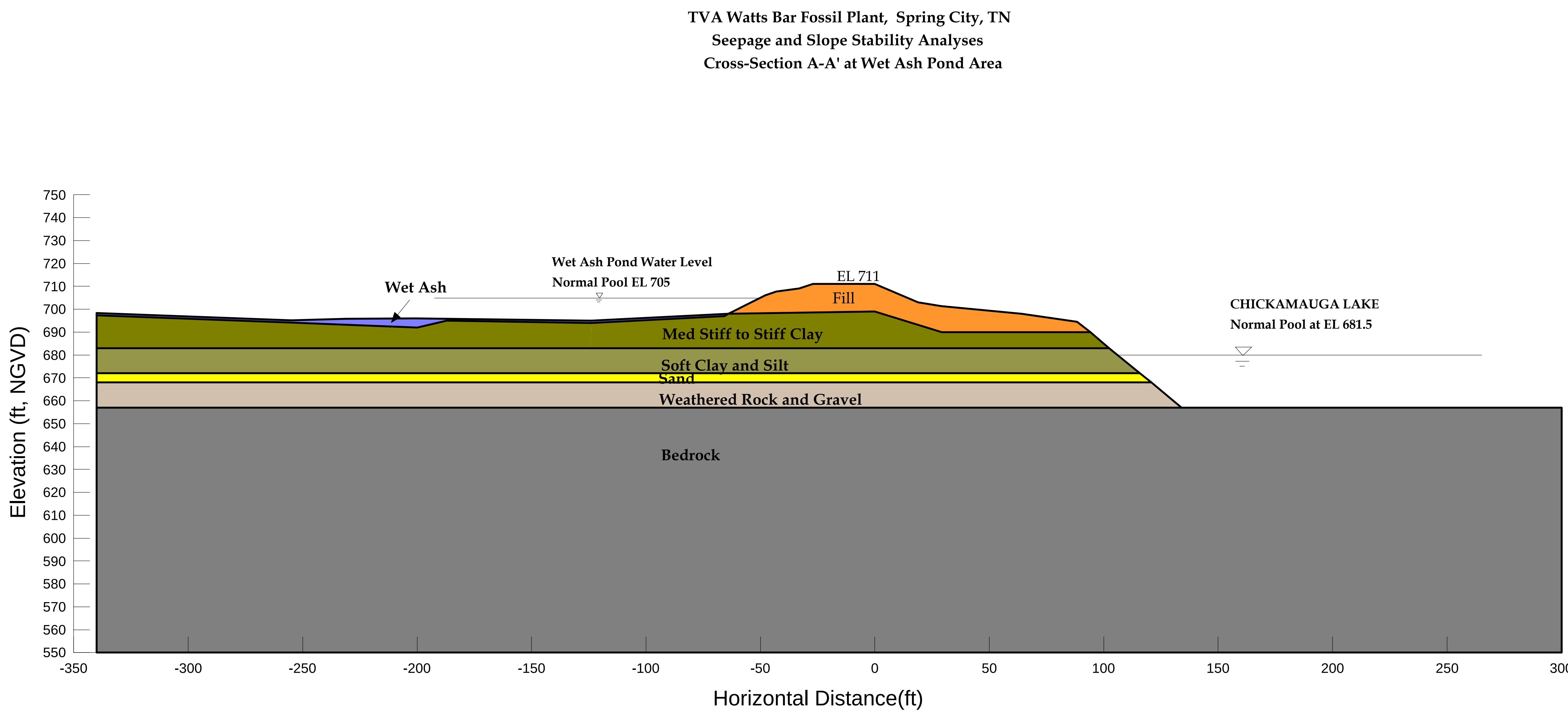


CDM Smith

Watts Bar Fossil Plant
Tennessee Valley Authority
Coal Combustion Products Closure Project

Borings and Cross-Section Locations

Figure 2



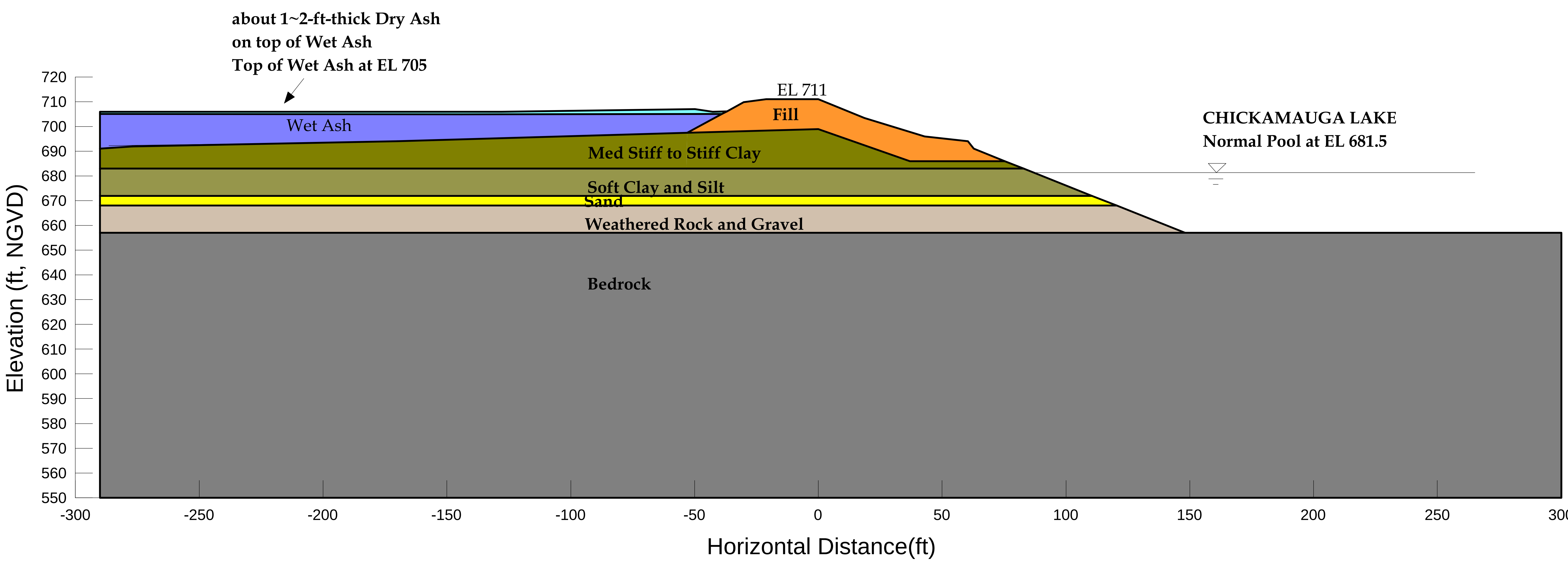
CDM Smith

Watts Bar Fossil Plant
Tennessee Valley Authority
Coal Combustion Products Closure Project

Cross-Section A-A' at Wet Ash Pond Area

Figure 3A

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section B-B' at Dry Ash Area



CDM Smith

Watts Bar Fossil Plant
Tennessee Valley Authority
Coal Combustion Products Closure Project

Cross-Section B-B' at Dry Ash Area

Figure 3B

ATTACHMENT A

CAMP DRESSER & McKEE

CDM Smith

BOREHOLE LOG
B-1

Client: TVA
Project Location: Spring City, TN
Project Name: TVA Watts Bar Fossil Plant
Project Number: 83529

Drilling Contractor: Total Depth Drilling
Drilling Method/Rig: 3.25" HSA/CME-55
Drillers: Tim Hall
Drilling Date: Start: 11-16-11 **End:** 11-17-11
Borehole Coordinates:
N 466,232.90 E 2,331,561.10

Surface Elevation (ft.): 699
Total Depth (ft.): 44.6
Depth to Initial Water Level (ft-bgs): 9.32
Abandonment Method: Converted to observation well
Field Screening Instrument:
Logged By: M. Howe

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			699.0 / 0				GW	GRAVEL - 2.0 Inches.
SS	S-1	12/11			10, 40		FILL	Moist to wet, dense, tan-brown and gray, GRAVEL and SILT, **-FILL-**
SS	S-2	24/22			12, 17, 27, 22			Moist, dense, dark brown and yellow-brown, SAND and SILT, trace gravel.
SS	S-3	24/18	694.0	0.25	13, 19, 21, 11			Moist, hard, orange-brown to blue-gray and tan, SILT, some sand.
SS	S-4	24/20	5	1.0	3, 8, 5, 6			Moist, stiff, tan to blue-gray mottling, CLAY, trace silt, sand, and wood fragments.
SS	S-5	24/16		0.75	2, 3, 2, 2			Moist, medium stiff, tan to blue-gray, CLAY, trace silt, sand, and gravel.
SS	S-6	24/18	689.0 / 10	0.5	3, 3, 4, 6			Moist, medium stiff, medium brown to tan-brown, SILT, some sand, trace gravel.
SS	S-7	24/18	684.0		2, 2, 2, 2		SC/CL	Moist to wet, very loose to loose, SAND and CLAY, little silt. **- ALLUVIAL SOIL -**

EXPLANATION OF ABBREVIATIONS

DRILLING METHODS:
HSA - Hollow Stem Auger
SSA - Solid Stem Auger
HA - Hand Auger
AR - Air Rotary
DTR - Dual Tube Rotary
FR - Foam Rotary
MR - Mud Rotary
RC - Reverse Circulation
CT - Cable Tool
JET - Jetting
D - Driving
DTC - Drill Through Casing

SAMPLING TYPES:
AS - Auger/Grab Sample
CS - California Sampler
BX - 1.5" Rock Core
NX - 2.1" Rock Core
GP - Geoprobe
HP - Hydro Punch
SS - Split Spoon
ST - Shelby Tube
WS - Wash Sample
OTHER:
AGS - Above Ground Surface

REMARKS

Hammer weight = 140 pounds, drop height = 30 inches
Split spoon = 2 inches OD, 24 inches long

Borehole coordinates are approximate based upon handheld GPS and elevations are estimated by overlaying coordinates with the survey.

Boring logs are draft and will be finalized upon receiving laboratory test results.

Reviewed by: **Date:**

CAMP DRESSER & McKEE

CDM Smith

BOREHOLE LOG
B-1

Client: TVA

Project Location: Spring City, TN

Project Name: TVA Watts Bar Fossil Plant

Project Number: 83529

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			684.0 15				SC/CL	
SS	S-8	24/24		0.75	2 3 4 5		CL	Moist to wet, medium stiff, red-brown to tan-brown, CLAY, little to some sand.
			679.0 20					
SS	S-9	24/24		0.5	2 4 4 5			Moist to wet, medium stiff to stiff, orange-brown to gray-tan, CLAY, some silt, trace to little sand.
			674.0 25					
SS	S-10	24/24			1 2 3 7		SM	Wet, loose, gray to tan, SAND, some silt.
			669.0 30					
SS	S-11	15/10			11 67 100/3"		ML/GM	Moist to dry, hard, gray, SILT and WEATHERED SHALE. **-WEATHERED ROCK-** Auger refusal at 33.0 feet below ground surface.
NQ	C-1	63/6	664.0 35		2:45 1:45 2:15 8:30 3:15		GW	Split-spoon refusal at 34.3 feet below ground surface. **RUN 1: 34.3 to 39.6 feet-bgs** REC = 9.5%, RQD = 0% Moderately hard, highly weathered, green and brown to gray, aphanitic, INTERBEDDED SHALE, LIMESTONE, and RIVER ROCK; extremely thin bedding, low angle jointing, very close spacing, rough, discolored, open, quartz vugs.

CAMP DRESSER & McKEE

BOREHOLE LOG
B-1

Client: TVA

Project Location: Spring City, TN

Project Name: TVA Watts Bar Fossil Plant

Project Number: 83529

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
					3:00		GW	
NQ	C-2	60/7.5	659.0 40		8:15 5:00 4:15 6:45 3:30	SHALE/LIMESTONE		**RUN 2: 39.6 to 44.6 feet-bgs** REC = 12.5%, RQD = 0% Moderately hard to hard, highly weathered, gray, aphanitic, INTERBEDDED SHALE and LIMESTONE; very thin to extremely thin bedding, low angle jointing, very close to close spacing, rough, discolored, open, calcite veins.
			654.0 45					Boring terminated at 44.6 feet below ground surface.
			649.0 50					
			644.0 55					
			639.0 60					

CAMP DRESSER & McKEE

CDM Smith

BOREHOLE LOG
B-2

Client: TVA
Project Name: TVA Watts Bar Fossil Plant
Project Location: Spring City, TN
Project Number: 83529

Drilling Contractor: Total Depth Drilling
Surface Elevation (ft.): 711
Drilling Method/Rig: 3.25" HSA/CME-55
Total Depth (ft.): 46.1
Drillers: Allan Fowler
Depth to Initial Water Level (ft-bgs): 27.4
Drilling Date: Start: 1-10-12 **End:** 1-10-12
Abandonment Method: Grouted to ground surface
Borehole Coordinates:
N 465,036.40 E 2,331,471.00
Field Screening Instrument:
Logged By: M. Howe

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			711.0 / 0				ASPHALT	ASPHALT - 3.0 Inches.
							GRAVEL BASE	GRAVEL BASE - 8.0 Inches.
SS	1	24/23		3.0	5, 6, 8, 9		FILL	Moist, stiff, orange brown, CLAY, **-FILL-**.
SS	2	24/24		>4.5	3, 6, 10, 14			Moist, very stiff, orange brown, CLAY, some silt, trace gravel. Moist, very stiff, dark brown, CLAY, some silt, trace gravel.
SS	3	24/24	706.0 / 5	>4.5	5, 8, 11, 12			Moist, very stiff, dark brown with gray mottling, CLAY , some silt.
SS	4	24/24		4.0	5, 8, 10, 11			Moist, very stiff, dark brown with light brown and gray mottling, CLAY, some silt.
SS	5	24/24	701.0 / 10	4.5	4, 6, 7, 9			Moist, stiff, dark brown with gray and light brown mottling, CLAY, some silt.
							CL	
SS	6	24/14	696.0	2.0	3, 6, 7			Moist, stiff, orange to yellow brown, CLAY, little sand (in lenses). **- ALLUVIAL SOIL -**

EXPLANATION OF ABBREVIATIONS

DRILLING METHODS:
HSA - Hollow Stem Auger
SSA - Solid Stem Auger
HA - Hand Auger
AR - Air Rotary
DTR - Dual Tube Rotary
FR - Foam Rotary
MR - Mud Rotary
RC - Reverse Circulation
CT - Cable Tool
JET - Jetting
D - Driving
DTC - Drill Through Casing

SAMPLING TYPES:
AS - Auger/Grab Sample
CS - California Sampler
BX - 1.5" Rock Core
NX - 2.1" Rock Core
GP - Geoprobe
HP - Hydro Punch
SS - Split Spoon
ST - Shelby Tube
WS - Wash Sample
OTHER:
AGS - Above Ground Surface

REMARKS

Hammer weight = 140 pounds, drop height = 30 inches
Split spoon = 2 inches OD, 24 inches long

Borehole coordinates are approximate based upon handheld GPS and elevations are estimated by overlaying coordinates with the survey.

Boring logs are draft and will be finalized upon receiving laboratory test results.

Reviewed by: **Date:**

CAMP DRESSER & McKEE

CDM Smith

BOREHOLE LOG
B-2

Client: TVA
Project Name: TVA Watts Bar Fossil Plant
Project Location: Spring City, TN
Project Number: 83529

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			696.0 15		8		CL	
							CL-CH	Moist, medium stiff to stiff, medium brown to tan, CLAY, trace to little sand.
SS	7	24/25	691.0 20	2.3	3 3 5 5			
ST	1	24/26		1.0				Shelby tube sample collected from 20.5 to 22.5 feet below ground surface. Moist to wet, medium brown, CLAY, little Silt, trace sand.
SS	8	24/19		0.8	2 3 3 3			Moist to wet, medium stiff, medium brown, CLAY , trace to little silt.
			686.0 25				CH	Moist to wet, medium stiff, medium brown, high plasticity CLAY, little silt, trace sand.
SS	9	24/24	681.0 30	1.0	1 2 3 3			
SS	10	24/26	676.0 35	0.5	1 2 2 2			Wet, soft to medium stiff, medium brown, high plasticity CLAY, little to some Silt, little Sand.
							SP	Wet, loose, medium brown, fine to medium SAND, trace to little silt.
					1			

BOREHOLE-PP READINGS/NO ROCK TVA WATTS BAR FOSSIL PLANT.GPJ CDM_CORP.GDT 1/31/12

CAMP DRESSER & McKEE

BOREHOLE LOG
B-2

Client: TVA
Project Name: TVA Watts Bar Fossil Plant
Project Location: Spring City, TN
Project Number: 83529

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
SS	11	24/25	671.0 40		2 4 6		SP	
SS	12	23/24	666.0 45		1 11 13 100/5"		SW/GW	Wet, medium dense to very dense, gray, fine to coarse SAND and GRAVEL, trace silt. <<-WEATHERED ROCK-.
SS	13	1/1			100/1"			Auger refusal at 46.0 feet below ground surface. Split-spoon refusal at 46.1 feet below ground surface.
			661.0 50					
			656.0 55					
			651.0 60					

CAMP DRESSER & McKEE

CDM Smith

B O R E H O L E L O G
B-3

Client: TVA

Project Name: TVA Watts Bar Fossil Plant

Project Location: Spring City, TN

Project Number: 83529

Drilling Contractor: Total Depth Drilling

Surface Elevation (ft.): 701

Drilling Method/Rig: 3.25" HSA/CME-55

Total Depth (ft.): 54.8

Drillers: Tim Hall

Depth to Initial Water Level (ft-bgs): 18.11

Drilling Date: Start: 11-15-11 **End:** 11-16-11

Abandonment Method: Converted to observation well

Borehole Coordinates:

N 464,593.80 E 2,331,431.10

Field Screening Instrument:

Logged By: M. Howe

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			701.0 / 0		2		TOPSOIL	TOPSOIL - 2-Inches.
SS	S-1	24/18		3.5	4, 5, 6		FILL	Moist, stiff, medium brown to dark brown, CLAY, trace sand, **-FILL-**
SS	S-2	24/24		1.0	4, 7, 12, 9			Moist, very stiff, medium brown to dark brown with orange, CLAY, trace sand.
SS	S-3	24/20	696.0 / 5	2.0	4, 6, 6, 5			Moist, medium dense, medium brown with orange, SILT, some sand.
SS	S-4	24/22		1.0	6, 5, 7, 5			Moist, medium dense, medium brown to orange-brown, SAND, little silt.
SS	S-5	24/19		1.0	4, 4, 7, 5			Moist, stiff, medium brown to orange-brown, CLAY, little sand. Moist, medium dense, medium brown to orange-brown, SAND, little silt.
			691.0 / 10					
SS	S-6	24/22	686.0	1.0	3, 4, 5, 5		CL	Moist to wet, stiff, medium brown, CLAY, little silt. **- ALLUVIAL SOIL -**

EXPLANATION OF ABBREVIATIONS

DRILLING METHODS:
- HSA - Hollow Stem Auger
- SSA - Solid Stem Auger
- HA - Hand Auger
- AR - Air Rotary
- DTR - Dual Tube Rotary
- FR - Foam Rotary
- MR - Mud Rotary
- RC - Reverse Circulation
- CT - Cable Tool
- JET - Jetting
- D - Driving
- DTC - Drill Through Casing

SAMPLING TYPES:
- AS - Auger/Grab Sample
- CS - California Sampler
- BX - 1.5" Rock Core
- NX - 2.1" Rock Core
- GP - Geoprobe
- HP - Hydro Punch
- SS - Split Spoon
- ST - Shelby Tube
- WS - Wash Sample

OTHER:
- AGS - Above Ground Surface

REMARKS

Hammer weight = 140 pounds, drop height = 30 inches
Split spoon = 2 inches OD, 24 inches long

Borehole coordinates are approximate based upon handheld GPS and elevations are estimated by overlaying coordinates with the survey.

Boring logs are draft and will be finalized upon receiving laboratory test results.

Reviewed by: **Date:**

CAMP DRESSER & McKEE

BOREHOLE LOG
B-3

Client: TVA
Project Location: Spring City, TN
Project Name: TVA Watts Bar Fossil Plant
Project Number: 83529

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			686.0 / 15				CL	
SS	S-7	24/10	681.0 / 20	0.3	1 1 1 2		MH	Wet, very soft to soft, medium brown to tan-brown, ELASTIC SILT, little sand.
SS	S-8	24/24	676.0 / 25	0.5	2 1 2 2		CL	Wet, soft, medium brown to tan-brown, CLAY, some silt, trace sand.
SS	S-9	24/24	671.0 / 30		2 1 2 3		SP-SM	Wet, very loose, medium brown to gray-brown, fine SAND, some to little silt.
SS	S-10	24/15	666.0 / 35		2 8 11 12			Wet, medium dense, tan to gray, fine to medium SAND, little silt, trace gravel.
SS	S-11	8/8			48 100/2"		CL/GC	Moist to wet, hard, gray, CLAY and WEATHERED SHALE. **-WEATHERED ROCK-**

CAMP DRESSER & McKEE

BOREHOLE LOG
B-3

Client: TVA **Project Name:** TVA Watts Bar Fossil Plant
Project Location: Spring City, TN **Project Number:** 83529

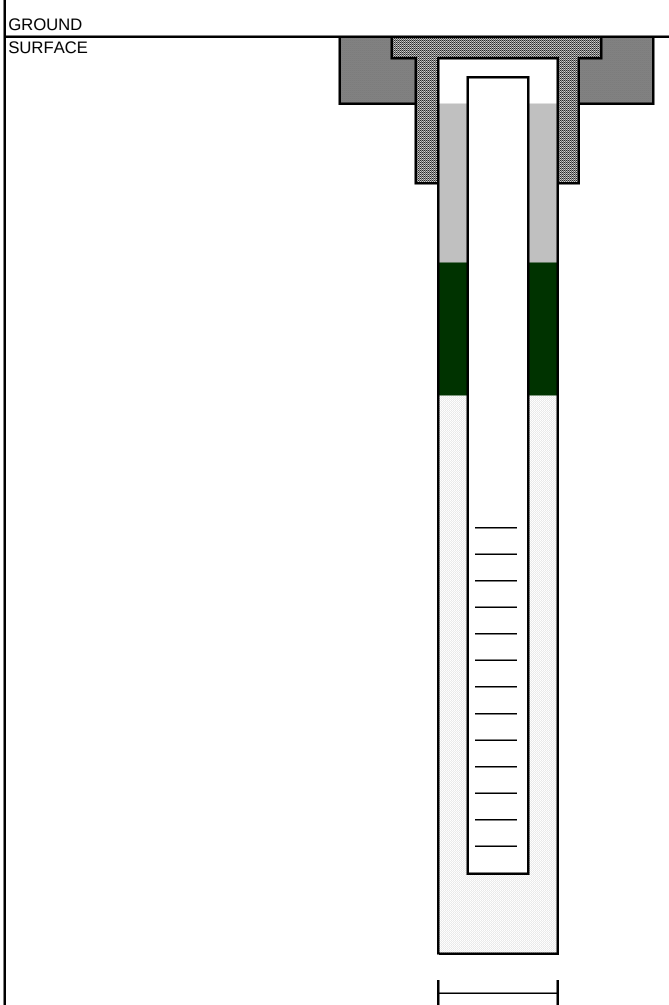
Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			661.0 40				CL/GC	Split-spoon refusal at 38.7 feet below ground surface. Auger refusal at 40.4 feet below ground surface.
NQ	C-1	52.8/6			7:30 6:00 6:00 5:15 2:00		GW	**RUN 1: 40.4 to 44.8 feet-bgs** REC = 9%, RQD = 0% Moderately hard to hard, highly weathered, brown and orange to gray, aphanitic, interbedded SHALE, LIMESTONE, and RIVER ROCK; extremely thin bedding, low angle jointing, very close spacing, rough, discolored, open, calcite veins.
NQ	C-2	60/14	656.0 45		4:30 7:00 6:00 7:15 8:15		SHALE/LS	**RUN 2: 44.8 to 49.8 feet-bgs** REC = 23%, RQD = 0% Moderately hard to hard, highly weathered,gray, aphanitic, interbedded LIMESTONE and SHALE; very thin bedding, low angle jointing, very close spacing, rough, discolored, open, calcite veins.
NQ	C-3	60/9.5	651.0 50		9:45 16:15 7:30 8:15 6:45		SHALE/LS	**RUN 3: 49.8 to 54.8 feet-bgs** REC = 16%, RQD = 0% Moderately hard, highly weathered,gray, aphanitic, interbedded LIMESTONE and SHALE; extremely thin to very thin bedding, low angle jointing, very close spacing, rough, discolored, open.
			646.0 55					Boring terminated at 54.8 feet below ground surface.
			641.0 60					

CDM Smith

5400 Glenwood Ave
Suite 300
Raleigh, NC 27612
(919) 787-5620

Monitoring Well Installation Log

Client:	TVA	Contractor:	Total Depth Drilling	Boring/Well No.:	B-1/MW-1
Project Name:	Watts Bar Fossil Plant	Driller:	Tim Hall	Date Installed:	11/17/11 - 01/11/12
Project Location:	Watts Bar (Rhea Co.), TN	Ground EL:	699.0 ft	Logged By:	MRH
Project Number:	83529	Riser EL:		Page:	1 of 1



SURFACE SEAL: (Thickness & Type) — 1 ft - Portland Cement

BACKFILL MATERIAL: (Type) — Soil sloughed into hole

TOP OF SEAL: — 16 ft

SEAL CONSTRUCTION: (Thickness & Type) — 7 ft - Bentonite

TOP OF SANDPACK: — 23 ft

RISER CONSTRUCTION: (Type, Diameter Material) — Schedule 40 PVC, 2 - Inch

TOP OF SCREEN: — 25 ft

SANDPACK TYPE: — Filter Sand - DSI Well Gravel Pack

SCREEN MATERIAL: (Type, Slot, Diameter Material) — Schedule 40 PVC, 0.10, 2-Inch

BOTTOM OF SCREEN: — 35 ft

BOTTOM OF BOREHOLE: — 44.6 ft

BOREHOLE DIAMETER: — 0.75 ft - soil/0.24 ft - rock

NOTE: All depths are in feet below ground surface, unless noted otherwise.

Remarks:

Updated On: 04/09/01

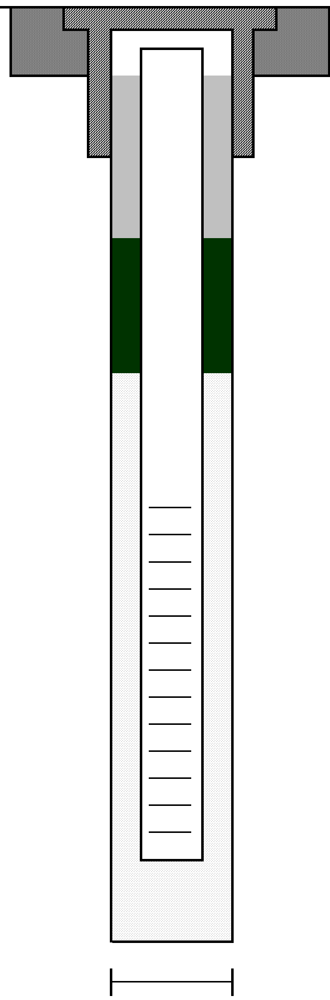
CDM Smith
5400 Glenwood Ave
Suite 300
Raleigh, NC 27612
(919) 787-5620

Monitoring Well Installation Log

Client:	TVA	Contractor:	Total Depth Drilling	Boring/Well No.:	B-3/MW-3
Project Name:	Watts Bar Fossil Plant	Driller:	Tim Hall	Date Installed:	11/16/2011
Project Location:	Watts Bar (Rhea Co.), TN	Ground EL:	701.0 ft	Logged By:	MRH
Project Number:	83529	Riser EL:		Page:	1 of 1

GROUND SURFACE

ROADWAY BOX



SURFACE SEAL: (Thickness & Type)	3 ft - Portland Cement
BACKFILL MATERIAL: (Type)	Filter Sand (DSI gravel pack)
TOP OF SEAL:	24 ft
SEAL CONSTRUCTION: (Thickness & Type)	4 ft - Bentonite
TOP OF SANDPACK:	28 ft
RISER CONSTRUCTION: (Type, Diameter Material)	Schedule 40 PVC, 2-Inch
TOP OF SCREEN:	30 ft
SANDPACK TYPE:	:Filter Sand - DSI Well Gravel Pack
SCREEN MATERIAL: (Type, Slot, Diameter Material)	Schedule 40 PVC, 0.10, 2-Inch
BOTTOM OF SCREEN:	40 ft
BOTTOM OF BOREHOLE:	54.8 ft
BOREHOLE DIAMETER:	0.75 ft - soil/0.24 ft - rock

NOTE: All depths are in feet below ground surface, unless noted otherwise.

Remarks:

Updated On: 04/09/01

Summary of Groundwater Level Readings
TVA WBF CCP Closure
Spring City, TN

Location	Ground Surface Elevation	Groundwater Level Readings		Date	Time (24 hr)
		in feet below ground surface	Elevation, ft		
B-1	699	12.1	686.9	11/16/2011	17:15
		13.1	685.9	11/16/2011	17:40
		9.32	689.7	1/11/2012	10:40
B-2	711	37.1	673.9	1/10/2012	13:05
		27.4	683.6	1/10/2012	14:50
B-3	701	31.15	669.9	11/15/2011	10:20
		15.70	685.3	11/16/2011	11:00
		19.00	682.0	1/10/2012	15:10
		18.11	682.9	1/11/2012	11:10

Note: Elevations & locations based on estimated distance to existing features.

ATTACHMENT B

Client: TVA
Project: WBF CCP CLOSURE

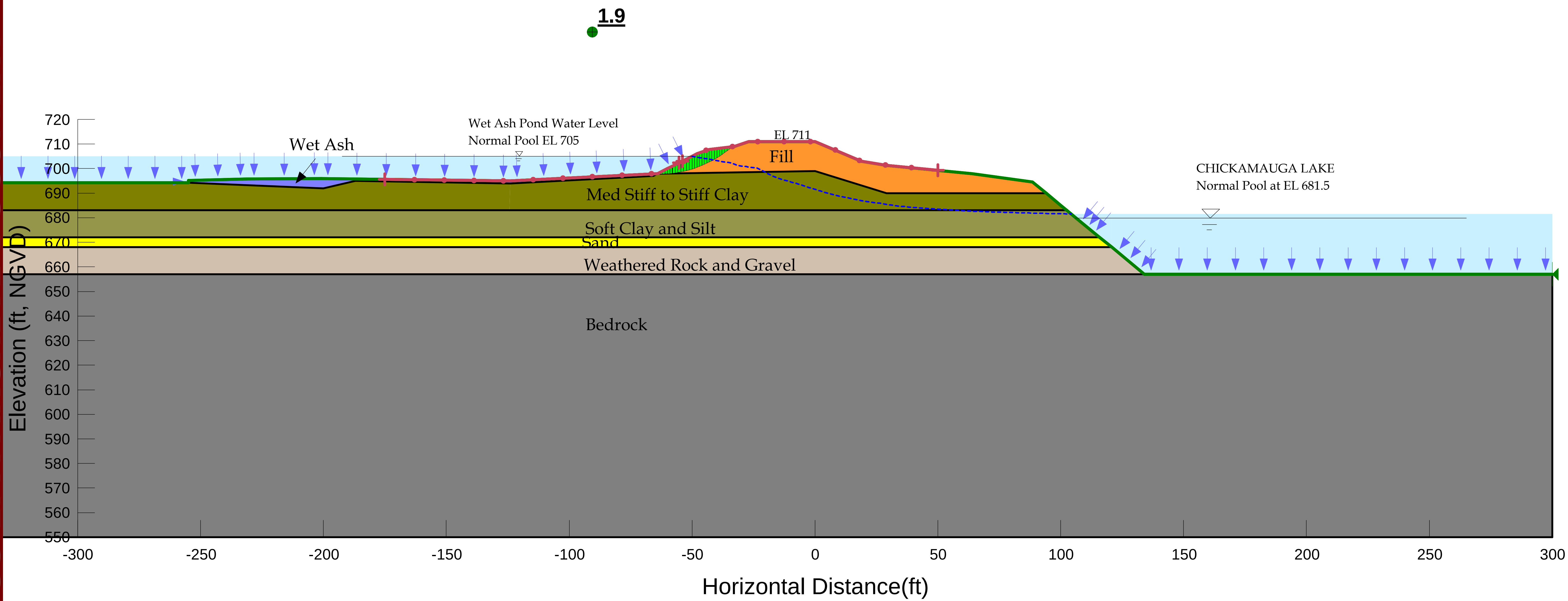
Computed By: Wen, Jintao
Date & Time: 1/20/2012 10:03:52 AM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section A-A' at Wet Ash Pond Area

Case Number: A-1
Location: Section A-A'

Model Scenario:
Existing Conditon at Wet Ash Pond Area
Static Analysis

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 0 psf 29 °
Layer 2B: Soft Clay and Silt 110 pcf 0 psf 28 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

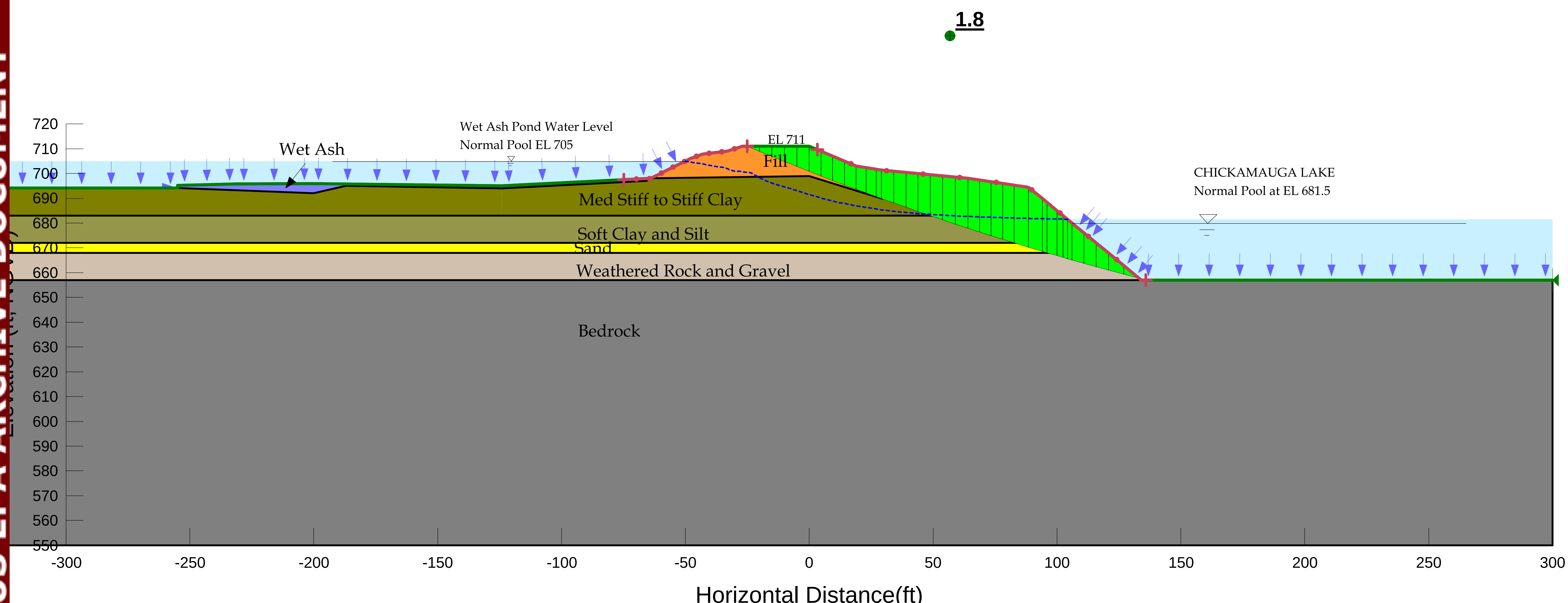
Computed By: Wen, Jintao
Date & Time: 1/20/2012 10:03:52 AM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section A-A' at Wet Ash Pond Area

Case Number: A-1
Location: Section A-A'

Model Scenario:
Existing Conditon at Wet Ash Pond Area
Static Analysis

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 0 psf 29 °
Layer 2B: Soft Clay and Silt 110 pcf 0 psf 28 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

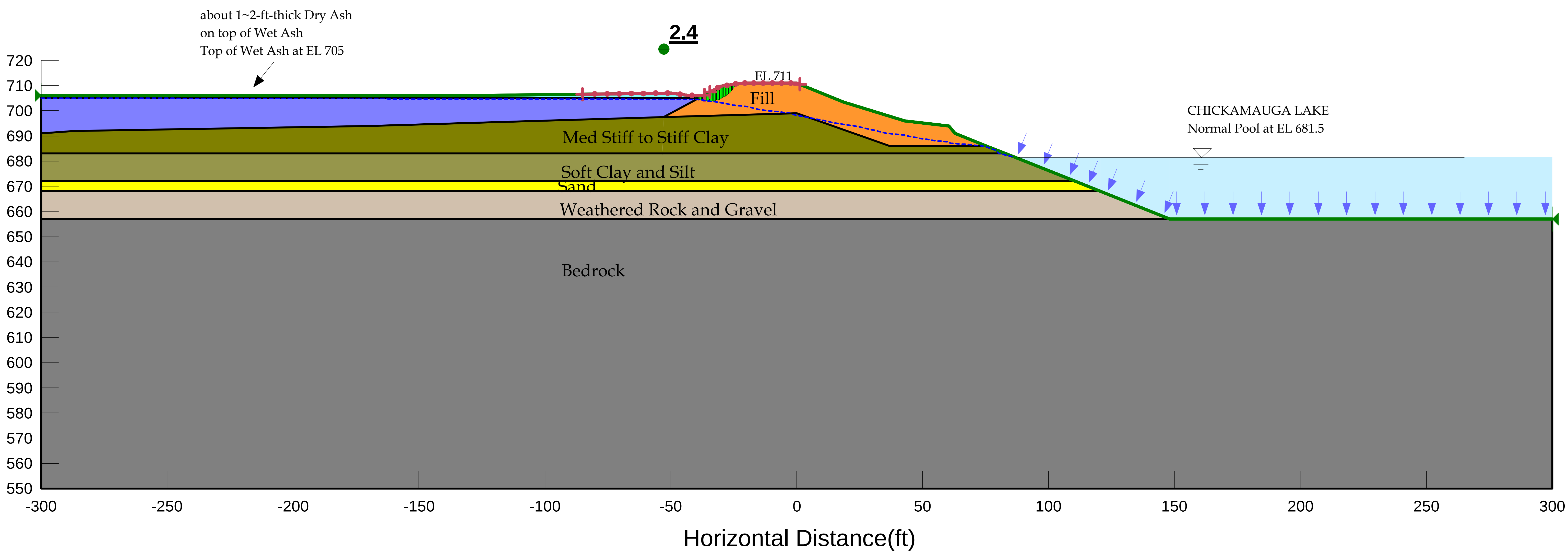
Computed By: Wen, Jintao
Date & Time: 1/20/2012 9:58:09 AM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section B-B' at Dry Ash Area

Case Number: B-1
Location: Section B-B'

Model Scenario:
Existing Conditon at Dry Ash Area.
Static Analsyes

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Dry Ash 85 pcf 0 psf 25 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 0 psf 29 °
Layer 2B: Soft Clay and Silt 110 pcf 0 psf 28 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

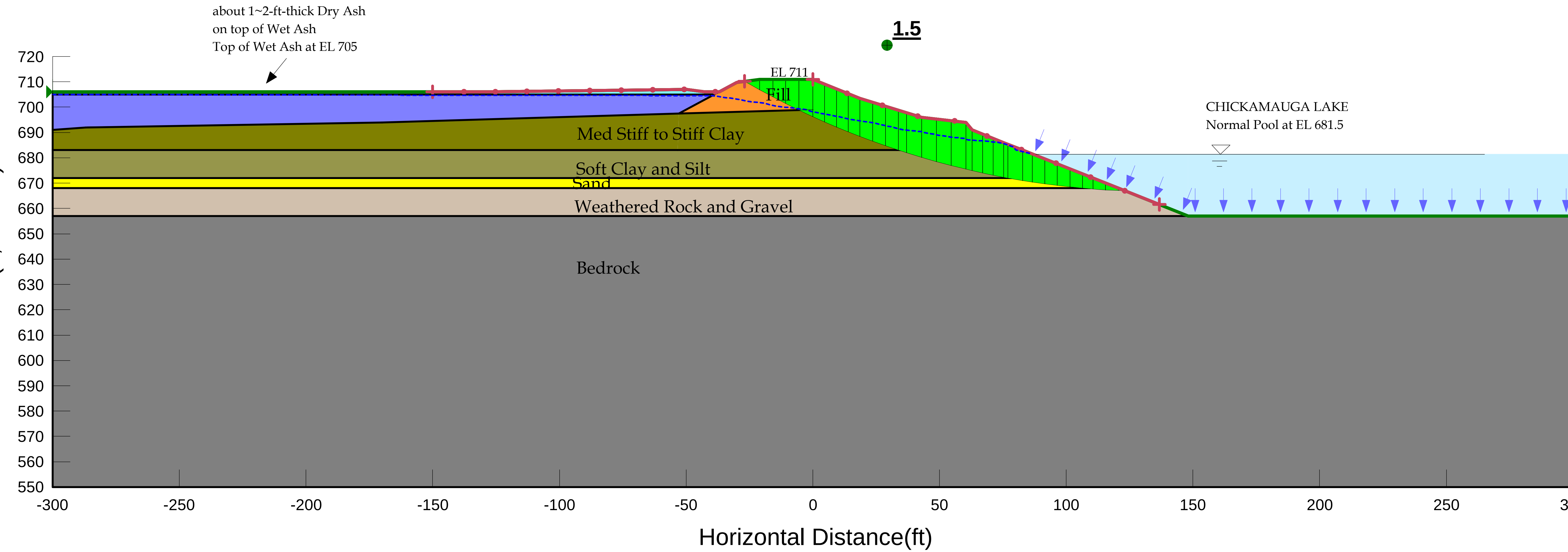
Computed By: Wen, Jintao
Date & Time: 1/20/2012 10:02:17 AM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section B-B' at Dry Ash Area

Case Number: B-1
Location: Section B-B'

Model Scenario:
Existing Conditon at Dry Ash Area.
Static Analsyes

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Dry Ash 85 pcf 0 psf 25 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 0 psf 29 °
Layer 2B: Soft Clay and Silt 110 pcf 0 psf 28 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

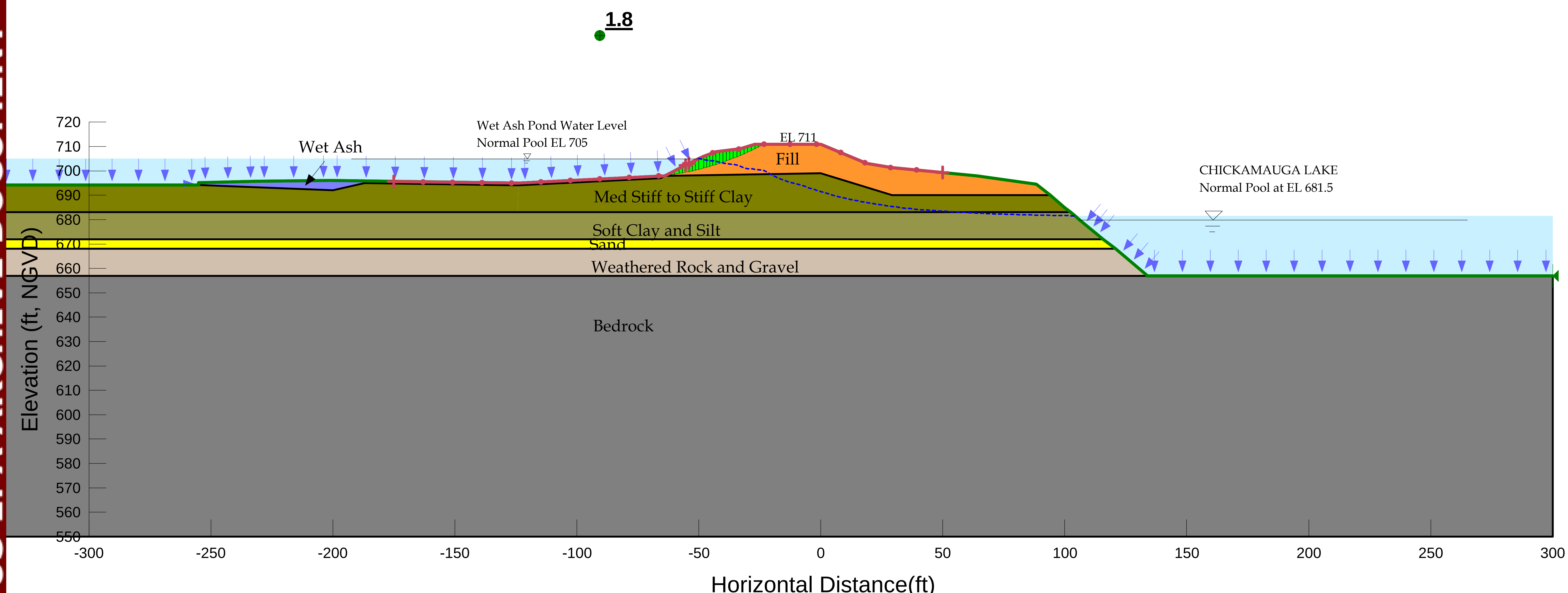
Computed By: Wen, Jintao
Date & Time: 1/19/2012 5:19:20 PM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section A-A' at Wet Ash Pond Area

Case Number: A-2
Location: Section A-A'

Model Scenario:
Existing Conditon at Wet Ash Pond
Seismic Analysis PHA=0.05g

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 1000 psf 0 °
Layer 2B: Soft Clay and Silt 110 pcf 500 psf 0 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

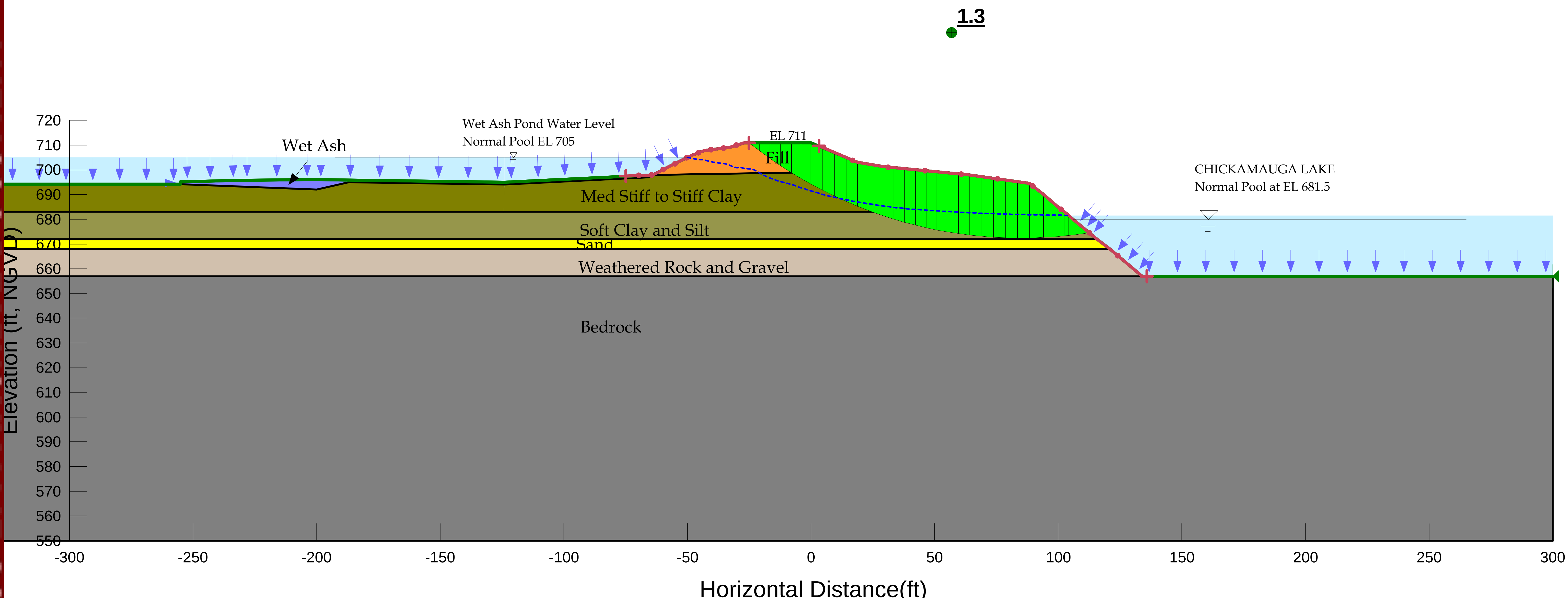
Computed By: Wen, Jintao
Date & Time: 1/19/2012 5:19:20 PM

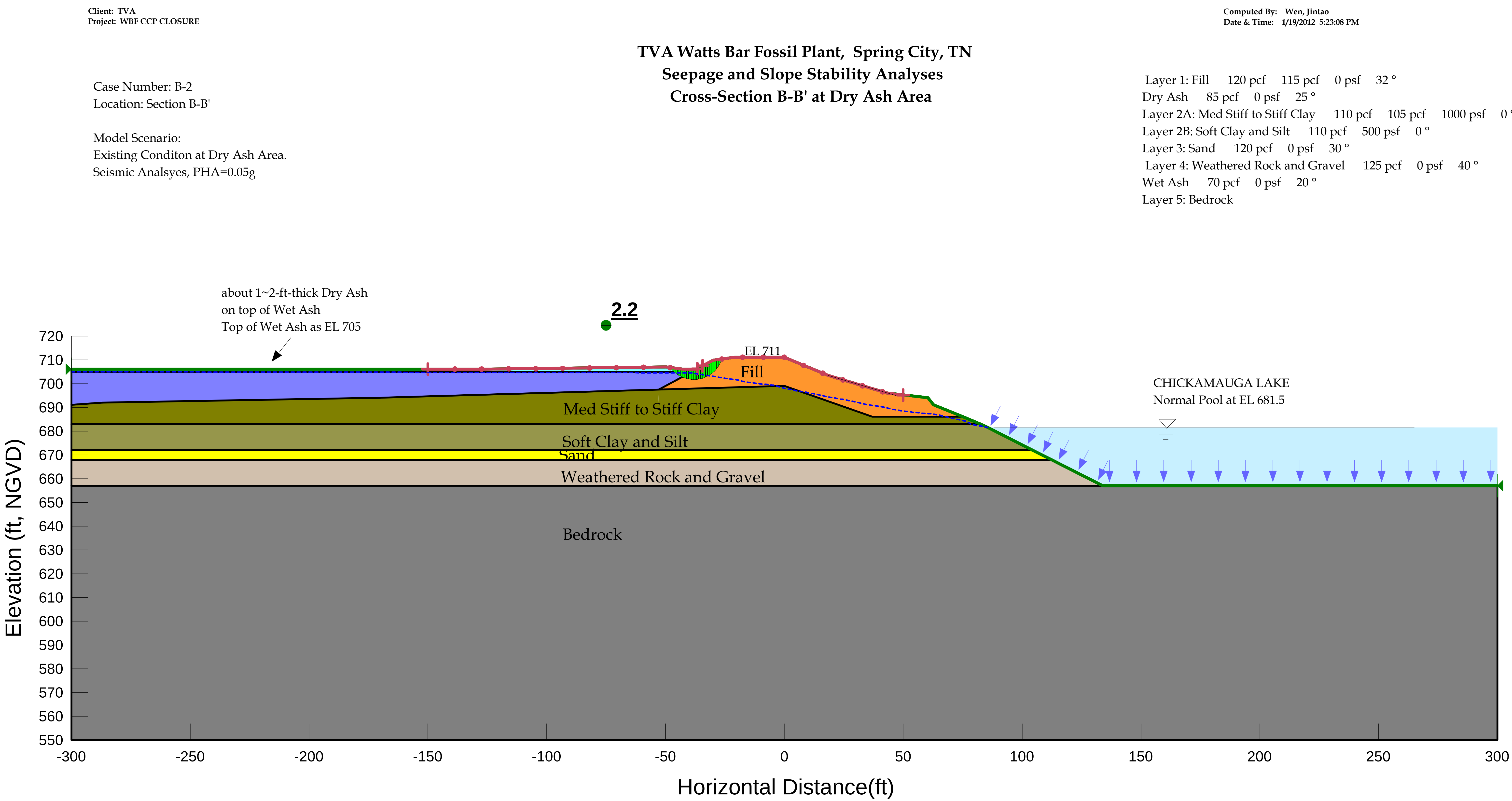
TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section A-A' at Wet Ash Pond Area

Case Number: A-2
Location: Section A-A'

Model Scenario:
Existing Conditon at Wet Ash Pond
Seismic Analysis PHA=0.05g

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 1000 psf 0 °
Layer 2B: Soft Clay and Silt 110 pcf 500 psf 0 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock




Client: TVA
Project: WBF CCP CLOSURE
Computed By: Wen, Jintao
Date & Time: 1/19/2012 5:23:08 PM
TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section B-B' at Dry Ash Area
Case Number: B-2
Location: Section B-B'
Model Scenario:
Existing Conditon at Dry Ash Area.
Seismic Analsyes, PHA=0.05g
Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Dry Ash 85 pcf 0 psf 25 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 1000 psf 0 °
Layer 2B: Soft Clay and Silt 110 pcf 500 psf 0 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock
about 1~2-ft-thick Dry Ash
on top of Wet Ash
Top of Wet Ash as EL 705
2.2
EL 711
Fill
Med Stiff to Stiff Clay
Soft Clay and Silt
Sand
Weathered Rock and Gravel
Bedrock
CHICKAMAUGA LAKE
Normal Pool at EL 681.5
Elevation (ft, NGVD)
720
710
700
690
680
670
660
650
640
630
620
610
600
590
580
570
560
550
-300
-250
-200
-150
-100
-50
0
50
100
150
200
250
300
Horizontal Distance(ft)

Client: TVA
Project: WBF CCP CLOSURE

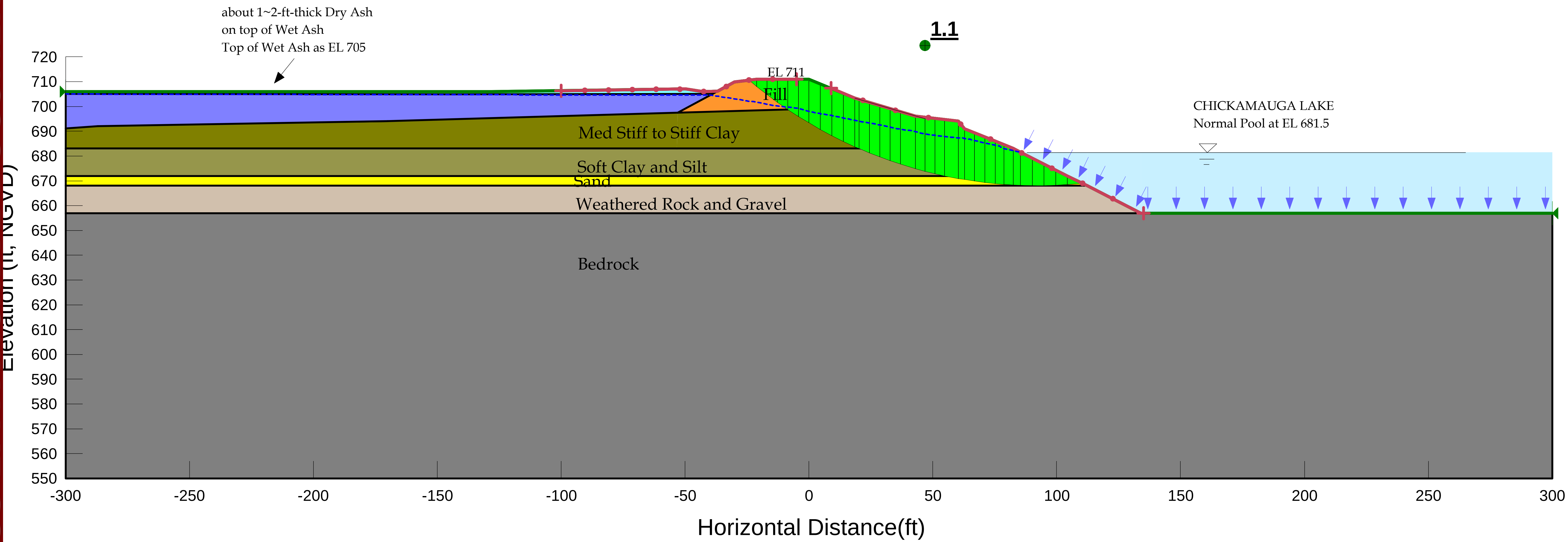
Computed By: Wen, Jintao
Date & Time: 1/19/2012 5:23:08 PM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section B-B' at Dry Ash Area

Case Number: B-2
Location: Section B-B'

Model Scenario:
Existing Conditon at Dry Ash Area.
Seismic Analsyes, PHA=0.05g

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Dry Ash 85 pcf 0 psf 25 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 1000 psf 0 °
Layer 2B: Soft Clay and Silt 110 pcf 500 psf 0 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



ATTACHMENT C

Seismic Risk Assessment
Closed CCP Storage Facilities
Tennessee Valley Authority Fossil Plants

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Seismic Risk Assessment
Closed CCP Storage Facilities
Tennessee Valley Authority Fossil Plants

This document outlines proposed engineering analyses to estimate seismic failure risks at wet storage facilities for coal combustion products, following closure, at various TVA fossil power plants. The specific details outlined in this document are subject to future discussion and modification by the project team.

OVERVIEW

Tennessee Valley Authority (TVA) operates storage facilities for coal combustion products (CCPs) at eleven fossil power generating stations. As TVA transitions to dry systems for handling these materials, 18 to 25 wet storage facilities (CCP ponds, impoundments, dredge cells, etc.) will be closed (drained and capped). The CCP storage facilities are currently operated in accordance with state and federal regulations, but previously issued permits have not required evaluations for seismic performance. Moreover, the existing permits do not require seismic qualification for the storage facilities in their closed configurations.

TVA recognizes there is a potential for strong earthquakes to occur within the region, and there is a tangible risk for seismic failure at each closed CCP facility. These risks, including both the likelihood of failure and the consequences, must be understood to effectively manage TVA's portfolio of byproduct storage sites. This white paper summarizes the methodology that will be used to estimate these risks at the CCP storage facilities following closure.

Seismicity in the TVA service area is attributed to the New Madrid fault and smaller, less concentrated crustal faults. These two earthquake scenarios generate significantly different seismic hazards at each locality and will be considered independently within the risk assessment. At each closed byproduct facility, potential seismic failure modes will be evaluated in sequence. Instability due to soil liquefaction, slope instability due to inertial loading, and other potential failure mechanisms will be addressed. Seismic performance will be evaluated for differing earthquake return periods until a limiting (lowest return period) event that would cause failure is obtained. The probability of seismic failure will then correspond to the probability of this limiting earthquake event. The assessment of risk will also include estimates of potential consequences, as well as costs to mitigate the risks, that reflects the unique setting of the individual storage facilities after closure.

Following the same general methodology, seismic risks will be estimated in two phases. The near-term "Portfolio Seismic Assessment" will provide a rough estimate of seismic risks. The likely performance of each facility will be evaluated using simplified analyses, empirical methods, and the judgment of experienced engineers. The results will establish a ranking of the relative risks across the closure portfolio and also provide a preliminary picture of overall seismic risk. For the subsequent "Facility Seismic Assessments", seismic performance will be judged on the basis of site-specific data and detailed engineering analyses, which will be completed during the closure design process for individual facilities.

SEISMIC RISKS

This white paper provides an overview of the engineering methods proposed by Stantec for estimating seismic risks at TVA's closed byproduct storage sites. For each facility, four specific questions must be answered quantitatively:

(1) What is the approximate probability that a strong earthquake will occur?

Several seismic source zones could produce earthquakes large enough to impact these TVA sites. Very large magnitude earthquakes have occurred within the New Madrid seismic zone, which is located along the western boundaries of Tennessee and Kentucky. Because of their observed large magnitude and frequency of occurrence, New Madrid events contribute substantially to the seismic risks at all TVA sites. Ground motions from a New Madrid earthquake would attenuate with distance toward the east, such that local area sources also contribute significantly to site-specific seismic hazards.

Seismicity across the Tennessee Valley was previously characterized by AMEC/Geomatrix (2004), in a probabilistic study that focused on TVA dam sites. The same seismogenic model can be applied in evaluating earthquakes that would impact other TVA sites. Accordingly, probabilistic seismic hazards obtained from the 2004 AMEC/Geomatrix model will be used in the seismic risk assessment of the closed CCP storage facilities.

(2) Will a given earthquake cause failure in the closed facility?

Many of the TVA byproduct storage facilities are underlain by a substantial thickness of loose, saturated, alluvial soils (silts and sands). Some facilities will have layers of ash or other uncemented CCPs that remain saturated following closure. These materials, especially sluiced fly ash, are prone to liquefaction in a strong earthquake, as cyclic motions cause a build up of pore water pressure and a consequent loss of effective stress and shearing resistance. Extensive liquefaction in a foundation or CCP deposit under a storage facility would be expected, in most cases, to result in lateral spreading and massive slope movements (failure). Even without liquefaction, large slope deformations or failures may be triggered by lateral inertial loads during an earthquake. Liquefaction and dynamic loading of slopes are the most likely failure mechanisms, but other seismic failure modes, which may be unique to a particular closed storage facility, must also be evaluated.

(3) What are the potential consequences of a failure?

In addition to understanding the probability of failure, a risk assessment should consider the potential consequences. A failure is likely to have economic costs associated with clean-up and restoration of the site. Depending on the local site conditions, failure of a closed CCP facility may or may not cause significant impacts on the environment, waterways, transportation routes, buried or overhead utilities, or other infrastructure. Substantial economic costs would result if power generation is interrupted. Failure consequences may also include the potential loss of human life at some sites.

In this proposed seismic risk assessment, the definition of "failure" will be constrained to mean the displacement of stored materials to a distance beyond the permitted boundary of the facility. While smaller deformations in a closed storage facility could cause economic damages, the resulting consequences for TVA should be manageable. Hence, this risk assessment will focus on potential "failures" where stored materials could move past the permitted boundary.

(4) What are the approximate costs to mitigate the risks of a seismic failure?

With an understanding of the probability and consequences of failure, the potential risks can be quantified and understood, possibly leading to decisions to mitigate seismic risks in the closure of certain facilities. Mitigation measures might include ground improvement to reduce liquefaction potential (stone columns, deep soil mixing, jet grouting, or other appropriate technology), stabilization of slopes by flattening or buttressing, enhanced drainage features, or some other engineered solution. The potential cost of these risk mitigation strategies are needed to make appropriate management decisions.

PORTFOLIO AND FACILITY ASSESSMENTS

Seismic evaluations will be completed for each of the CCP storage facilities that TVA has slated for closure; a tentative list is given in Table 1. The assessment of seismic risks will be accomplished in two phases:

A. Portfolio Seismic Assessment

In this first phase, the seismic risk assessment will be carried out using general site information, simplified analyses, empirical methods, and the judgment of experienced engineers. A team of four to five engineers will complete this evaluation for the entire portfolio, with assistance from the engineering teams currently working on each facility. After the probabilistic seismic hazards are defined, this phase of the work can be completed in a relatively short timeframe.

Given the level of effort and the simplified engineering analyses to be employed, the seismic risk estimates from the Phase A assessment will be approximate. Rather than attempting to compute precise risk numbers, Phase A will focus on capturing the relative risks between the different closed facilities. The key to successfully meeting this objective will be the consistent application of the assessment process across the portfolio.

This effort will result in a ranked list of sites that can be used to illustrate where seismic risks are greatest within the portfolio. The results will also provide some insight for understanding and communicating the magnitude of potential risks associated with seismic loading of the closed CCP facilities.

As a secondary objective, the Phase A assessment team will also consider the potential for failure of the active storage facilities, due to an earthquake occurring prior to closure. The seismic risks associated with the operating facility will not be estimated, but the Phase A assessment process provides an opportunity to identify potential failure mechanisms that should be addressed in the short term. This information may suggest the need to re-prioritize the closure schedule. Prior to closure, many of the wet CCP storage facilities retain large pools of water and are thus more susceptible to uncontrolled releases in an earthquake. TVA has already made the decision to close these wet storage facilities to manage these risks, so the effort in Phase A will focus on identifying sites that may have unusually high seismic risks and deserve more study or higher priority in the closure program.

B. Facility Seismic Assessment

In this subsequent phase of work, more detailed engineering analyses will be carried out using site-specific geometry, subsurface conditions, material parameters, and results from static slope stability analyses. Simplified, state-of-the-practice methods of engineering analysis will be used; more complex analytical methods will be generally impractical for this risk assessment.

This phase of the work will be accomplished for individual facilities as part of the closure design, after the completion of other engineering analyses. The risks will be quantified by the design team, with assistance from the portfolio seismic assessment team. Significant, detailed effort will be required to assess each closed facility.

Compared to Phase A, the risk estimates obtained at this stage will be more reliable and better represent the actual risks for seismic failure. While it will be impossible to know how accurately the risks have been characterized at the completion of Phase B, the objective is to obtain results that are within perhaps ± 30% of the "actual" risk numbers. TVA expects to use the Phase B results to decide if the risks are acceptable, or if the closure design should be modified to mitigate risks for a seismic failure.

The engineering methodology (described below) to be followed in the Phase A and B evaluations will not characterize all of the uncertainties with respect to seismic performance. The uncertainties in the soil parameters and in the liquefaction, stability, and deformation analyses will not be quantified and carried through the risk assessment. Consequently, the estimated risk numbers will be approximate, but the results will be sufficiently accurate to support TVA decisions regarding prioritization for closure or the need for seismic mitigation. At most sites, the risks are expected to be high enough or low enough that further refinement in the risk numbers would not change these decisions. More detailed analysis beyond Phase B would be unjustified in these cases.

This assessment plan does not preclude the possibility that more detailed risk evaluations could be undertaken in subsequent phases of work. The Phase B results might reveal a subset of closed facilities with marginal risks, where a more rigorous and complete calculation of the risks would be needed to support a management decision. Hence, at the conclusion of the Phase B assessments, a "Phase C" evaluation may be needed for select sites and facilities, wherein uncertainties in the soil parameters and performance analyses would be quantified and carried through the risk assessment.

RESULTS AND APPLICATION

The results from the Phase A Portfolio Assessment will be presented in a table, like Table 1. For each facility evaluated, the estimated annual probability of failure due to a seismic event, the expected consequences (economic costs and potential loss of life), and the mitigation costs (design features to reduce risks) will be tabulated. The same parameters, but more accurate numbers, will be reported from the more in-depth Phase B assessments. A qualitative description of the data quality (based on the number of borings, test data on key soil properties, etc.) will also be included, to indicate how well the site conditions were characterized at the time of the Phase A or B assessment.

In both Phase A and B, the evaluation teams will prepare a discussion of significant issues driving the seismic risks at each site. This summary will include knowledge gaps, likely failure mechanisms, unique consequences, suggested approaches for risk mitigation, and other key information. The Phase A evaluation of a facility may point out the need for additional data to support later seismic analyses in Phase B; needed field or laboratory testing could then be accomplished and documented as part of the facility closure design effort.

In the short term, TVA will utilize the Phase A results to better plan budgets and schedules for managing the closure process over the next several years. The Phase A assessment will also be used as an opportunity to identify operating facilities with especially high seismic risks. While these risks will not be quantified for conditions prior to closure, the consideration of potential seismic failure modes may prompt additional study and reconsideration of priorities. Where justified, the priorities for closure may be changed to more quickly address sites with higher seismic risks.

More accurate risk estimates will be obtained from the Phase B assessments, which will be completed as part of the closure design process. Those results will be used, within TVA's existing decision making framework, to judge if seismic mitigation is needed. For context, the criteria in Tables 2 and 3 represent the risk-based framework TVA uses to guide enterprise-level decisions. This framework relies upon broad, qualitative scoring of consequences and risks for the organization. For managing the seismic risks at the closed CCP facilities, complete probabilistic calculations of risk are not needed; approximate estimates of seismic risk will be sufficient to support TVA decisions.

The risks computed in Phase A and B will not be compared to a prescribed threshold or design risk level. Criteria for tolerable seismic risk in these closed CCP storage facilities has not been defined in the existing permits, in TVA policy, or in TVA design guidance.

METHODOLOGY

The same general methodology, outlined in ten steps below and in Figures 1 through 4, will be used to evaluate seismic risk in both the Phase A Portfolio Assessments and the Phase B Facility Assessments. While advanced engineering analyses may be required to demonstrate acceptable seismic performance in a design situation, simplified analyses will be used here, consistent with the goal of estimating the probability of failure.

In Step 1, seismic hazard parameters will be defined for each site; the results will be used as inputs for both the Phase A and Phase B assessments. Then, the evaluation of a particular facility will begin with a review of existing site information (Step 2), followed by engineering analyses for seismic performance. As described in Steps 3 through 7 below, the engineering analyses in Phase B will be more detailed than the simplified estimates in Phase A. The analyses will commence with an initial selection of an earthquake return period and evaluation for seismic performance. Steps 3 through 7 will be repeated until the limiting (lowest) earthquake return period expected to cause failure is obtained. Flowcharts summarizing Steps 1 through 7 in the Phase A and B seismic performance assessments are given in Figures 3 and 4, respectively. The earthquake event with the lowest return period that causes failure will then be used to compute the probability of failure in Step 8. The potential consequences and mitigation costs will be estimated in Steps 9 and 10.

Step 1 – Define Seismic Input Parameters

Seismic hazards at TVA dam sites were quantified in a 2004 study by AMEC/Geomatrix. The New Madrid fault zone and several area source zones contribute to the seismicity of the region, as represented schematically in Figure 1. The New Madrid seismic zone is characterized by a large linear, combined reverse/strike-slip fault. Earthquakes in the area source zones are more diffuse (less concentrated in clusters) and tend to occur in zones of weakness of large crustal extent rather than along narrow, well-defined faults. Earthquakes occurring within the New Madrid Seismic Zone and in area sources outside of it will be considered in developing seismic input parameters for each CCP facility. However, only seismic source zones that contribute significantly to the ground motion hazard at a particular site will be used to develop seismic input parameters.

The national USGS seismic hazard model will not be used in these seismic risk assessments; instead, TVA will ask AMEC/Geomatrix to compute the site-specific seismic hazards for each closed CCP facility. The needed information can be obtained from the existing seismogenic model, but will need to separately consider the hazards associated with the New Madrid events and all other seismic sources (Figure 2), hereafter referred to in this white paper as the "earthquake scenarios". The following parameters are needed for each earthquake scenario:

- Uniform hazard spectra for frequencies from 0.25 to 100 Hz (100 Hz value is equivalent to peak ground acceleration, PGA) at the top of rock for a range of return periods from 100 to 2,500 years.
- De-aggregation for relevant ground motion frequencies (one or more of the following: 0.5, 1.0, 2.5, 5.0, and 100 Hz) at each return period. The de-aggregation results will be used to select appropriate, representative earthquake parameters (magnitude and distance from the site), from which inputs needed for liquefaction analyses can be developed.

In the Phase A effort, the project team (including seismologists designated by TVA) will meet to consider the earthquake hazard data produced by the AMEC/Geomatrix model for each site. The team will reach consensus on the appropriate parameters (return period, earthquake magnitude, and peak ground acceleration) to be used in evaluating each facility, before proceeding with work on subsequent steps of the analysis. The seismic parameters to be tabulated (Table 4) will then be used in both the Phase A and Phase B assessments.

Ground motion time histories will be needed for the detailed Phase B calculations, and TVA will need to ask AMEC/Geomatrix to provide:

- Representative acceleration time histories (two orthogonal components), representing ground motions at the top of the rock profile for the specified earthquake return periods.

Given the results of the Phase A assessment, the Phase B analyses will focus on a narrower range of possible earthquakes. Hence, acceleration time histories will not be needed for every seismic event listed in Table 4.

Step 2 – Review Site and Facility Information

To meet the requirements for closure of TVA ash storage facilities, the closed condition may involve placement of compacted ash behind a strengthened dike, drainage of pond water to the levels of the surrounding groundwater table, and capping of the area with native soils. The collection of available site information for each facility will be reviewed from a seismic performance perspective. For the Phase B assessment, this information will be augmented with new data that becomes available during the closure design process.

The project information needed for each storage facility includes:

- Planned geometry of the closed storage facility, as needed to meet current design criteria and regulatory requirements.
- Geologic mapping and related information about the site geology.
- Historical records and other information related to site development.
- Boring logs, SPT data, CPT data, shear wave velocities, etc. from field explorations.
- Laboratory data from testing of site materials, including classification, Atterberg limits, moisture content, particle size, specific gravity, unit weight, compaction tests, and other relevant test data.
- Laboratory data on measured strength properties, for both drained and undrained conditions.
- Previously completed slope stability analyses, where available, will be modified for calculations in the risk assessments.

Step 3 - Evaluate Potential for Soil Liquefaction

The potential for soil liquefaction may be the greatest contributor to failure risk at many of the TVA storage sites. Liquefaction will thus be considered first in the assessment of seismic performance at each closed facility (Figures 3 and 4).

The Phase A assessment will utilize empirical charts and back-of-the-envelope calculations to judge if liquefaction would be likely for a given earthquake scenario. For example, Ambraseys (1988) compiled magnitude, epicentral distance, and whether or not liquefaction was observed in past earthquakes, and then suggested a threshold boundary (in terms of magnitude and epicentral distance) where liquefaction might occur in natural soil deposits. Selected, parametric calculations with the simplified procedure outlined by Youd et al (2001) will also be useful in judging what earthquakes would cause liquefaction in the Phase A Portfolio Assessments. These empirical methods may be unconservative for evaluating saturated CCPs, which are often more prone to liquefaction than a sandy soil, but the results will still provide useful guidance in the Phase A assessment.

For the Phase B liquefaction evaluations, detailed engineering analyses will be undertaken to obtain estimates of cyclic loading, soil resistance, and factor of safety as described below. Potentially liquefiable soils include saturated alluvial soils, loose granular fills, and sluiced ash. The detailed analyses will focus on critical cross sections of the closed facilities; liquefaction safety factors will not be computed for all boring locations at a site.

(a) Soil Loading from Earthquake Motions

The magnitude of the cyclic shear stresses induced by an earthquake are represented by the cyclic stress ratio (CSR). The simplified method proposed by Seed and Idriss (1971) will be used to estimate CSR in the Phase A parametric analyses (ground response analyses will not be completed in Phase A).

In Phase B, the CSR at specific locations (borings and depths where in situ penetration resistance are measured) will be computed using one-dimensional, equivalent-linear elastic methods as implemented in the ProSHAKE software. Using an acceleration time history at the top of rock (obtained from the seismic hazards study in Step 1), the computer program will model the upward propagation of the ground motions through a one-dimensional soil profile. For cases where the one-dimensional assumption is inadequate, the calculations can be accomplished using QUAKE, a two-dimensional finite element program that implements the same dynamic modulus reduction curves and damping relationships as used in ProSHAKE.

The cyclic stresses imparted to the soil will be estimated from the earthquake parameters described in Step 1, representing earthquakes on the New Madrid fault and local crustal events.

(b) Soil Resistance from Correlations with Penetration Resistance

The resistance to soil liquefaction, expressed in terms of the cyclic resistance ratio (CRR), will be assessed using the NCEER empirical methodology (Youd et al. 2001). Updates to the procedure from recently published research will be used where warranted. The analyses will be based on the blowcount value (N) measured in the Standard Penetration Test (SPT) or the tip resistance (q_c) measured in the Cone Penetration Test (CPT). In Phase A, typical or representative values will be used in parametric hand calculations; detailed data from site-specific explorations will be analyzed in Phase B.

The NCEER procedure involves a large number of correction factors. Based on the site-specific conditions and soil characteristics, engineering judgment will be used to select appropriate correction factors consistent with the consensus recommendations of the NCEER panel (Youd et al. 2001). To avoid inappropriately inflating the CRR, the NCEER fines content adjustment will not be applied where zero blowcounts ("weight of hammer" or "weight of rod") are recorded. The magnitude scaling factor (MSF) is used in the empirical liquefaction procedure to normalize the representative earthquake magnitude to a baseline 7.5M earthquake. The earthquake magnitude (M) considered to be most representative of the liquefaction risk will be determined by applying the MSF to the de-aggregation data (from Step 1) for each selected earthquake return period.

Saturated fly ash, where it remains following closure, is likely to be more susceptible to liquefaction than indicated by these empirical methods. Values of CRR determined via the NCEER procedure are related to the observation of liquefaction in natural soils, mostly silty sands. Given the spherical particle shape and uniform, small grain size of fly ash, the NCEER procedure may give CRR values that are too high for saturated fly ash.

Lacking better methods of analysis, the lower-bound, "clean sand" base curve (Youd et al. 2001) will be assumed to apply for fly ash in the Phase A assessment. Within the liquefaction calculations, this will be accomplished for these materials by neglecting the fines content adjustment to the normalized penetration resistance. For Phase B, published and unpublished data from cyclic laboratory testing on similar materials will be sought to augment the indications of liquefaction resistance obtained from in situ penetration tests.

(c) Factor of Safety Against Liquefaction

The factor of safety against liquefaction (FS_{liq}) is defined as the ratio of the liquefaction resistance (CRR) over the earthquake load (CSR). Following TVA design guidance and the precedent set by Seed and Harder (1990), FS_{liq} is interpreted as follows:

- Soil will liquefy where $FS_{liq} \leq 1.1$.
- Expect substantial soil softening where $1.1 < FS_{liq} \leq 1.4$.
- Soil does not liquefy where $FS_{liq} > 1.4$.

Using this criteria for guidance, values of FS_{liq} computed throughout a soil deposit or cross section (at specific CPT-q_c and SPT-N locations) will be reviewed in aggregate. Occasional pockets of liquefied material in isolated locations are unlikely to induce a larger failure, and are typically considered tolerable. Instead, problems associated with soil liquefaction are indicated where continuous zones of significant lateral extent exhibit low values of FS_{liq}. Engineering judgment, including consideration for the likely performance in critical areas, will be used for the overall assessment of each facility. A determination of "extensive" or "insignificant" liquefaction will then lead to the appropriate stability analyses in the next stage of the evaluation, as indicated in Figures 3 and 4.

Step 4 – Characterize Post-Earthquake Soil Strengths

The post-earthquake shearing resistance of each soil and CCP will be estimated, with consideration for the specific characteristics of that material. The full, static shear strength will be assigned to unsaturated soils. Excess pore pressures will not develop in an unsaturated soil during seismic loading, so drained strength parameters can be used. The undrained strengths of saturated soils will be decreased to account for the softening effects of pore pressure buildup during the earthquake. Specifically:

- In saturated clays and soils with $FS_{liq} > 1.4$, 80% of the static undrained strength will be assumed.
- In saturated, low-plasticity, granular soils with $1.1 < FS_{liq} \leq 1.4$, a reduced strength will be assigned, based on the excess pore pressure ratio, r_u (Seed and Harder 1990).

Typical relationships between FS_{liq} and r_u have been published by Marcuson and Hynes (1989).

- In saturated, low-plasticity, granular soils with $FS_{liq} \leq 1.1$, a residual (steady state) strength (S_{us}) will be estimated for the liquefied soil. Values of S_{us} can be obtained from the empirical correlations published by Seed and Harder (1990), Castro (1995), Olson and Stark (2002), Seed et al. (2003), and Idriss and Boulanger (2008).

Subsequent stability and deformation analyses will be accomplished using these reduced strength parameters. No attempt will be made to model the cyclic reduction in soil shear strength during an earthquake. In the deformation analyses, the fully reduced strengths will be assumed at the start of cyclic loading, which will yield conservative estimates of slope displacements.

Step 5 – Analyze Slope Stability

The next step in the performance evaluation (Figures 3 and 4) will consider slope stability, for conditions with or without significant liquefaction. Slope stability will be evaluated using two-dimensional, limit equilibrium, slope stability methods. Reduced soil strengths (from Step 4), conservatively representing the loss of shearing resistance due to cyclic pore pressure generation during the earthquake, will be used in the stability calculations. The analyses will be accomplished using Spencer's method of analysis, as implemented in the SLOPE/W software, considering both circular and translational slip mechanisms.

Input files for static stability calculations, where previously completed for a particular facility, will be updated to represent seismic conditions. These stability analyses may be not available, or the closure geometry may be undefined, for the Phase A assessment of some sites. In those cases, simplified or approximate geometries will be developed for approximate analysis in Phase A. Engineering experience will also be useful in judging likely seismic stability. For example, a complete failure is likely if liquefaction undermines the foundation of the outslope. In the absence of liquefaction, a slope that exhibits adequate safety factors under static conditions is unlikely to fail in an earthquake. Back-of-the-envelope hand calculations can be useful in assessing stability where extensive liquefaction occurs in the saturated materials within or below CCPs retained by a stable perimeter dike. Detailed slope stability calculations, which accurately represent the planned closure geometry, will be used in the Phase B facility assessments.

(a) Slope Stability if Extensive Liquefaction

If extensive liquefaction is indicated, stability will be evaluated for the static conditions immediately following the cessation of the earthquake motions. Residual or steady state strengths will be assigned in zones of liquefied soil, with reduced strengths that account for cyclic softening and pore pressure build up assumed in non-liquefied soil. In both Phase A and B, complete failure (large, unacceptable displacements) will be assumed if the safety factor (FS_{slope}) computed in this step is less than one (Figures 3 and 4).

For slopes where the post-earthquake $FS_{slope} \geq 1$, deformations will be estimated in the Phase B assessment (Step 6 and Figure 4). Slope deformations will not be estimated in the Phase A portfolio assessment, where ground motion time histories will not be available. In Phase A, slopes exhibiting $FS_{slope} \geq 1$ with liquefaction will be assumed stable with tolerable deformations; this condition may exist, for example, where liquefied ash at the base of a closed storage facility is contained within a stable perimeter dike.

Note that pseudostatic stability analyses are not useful for evaluating a factor of safety where extensive liquefaction is expected, because appropriate pseudostatic coefficients can not be defined.

(b) Slope Stability if No Significant Liquefaction

If no significant liquefaction is expected, seismic stability will be analyzed in Phase A using approximate, pseudostatic stability methods (Figure 3). The added inertial loads from the earthquake will be represented with a simple, horizontal pseudostatic coefficient (k_h), which provides an approximate representation of the dynamic loads imposed by an earthquake. The horizontal pseudostatic coefficient will be set to one-tenth of the peak ground acceleration in rock ($k_h = 0.1 \cdot PGA_{rock}$). In Phase A, tolerable deformations (less than about 5 meters) will be assumed if the pseudostatic $FS_{slope} \geq 1$, and failure will be assumed if the pseudostatic $FS_{slope} < 1$.

This approach and criteria are based on the work of Hynes-Griffin and Franklin (1984). They performed Newmark deformation analyses, integrated over 350 ground motion time histories, used an amplification factor of three to represent peak accelerations at the base of an earth embankment, and assumed a displacement of 1 meter would be tolerable for an embankment dam. For a typical CCP facility, assuming no pool is retained following closure, “failure” would imply displacements significantly greater than 1 meter. A tolerable displacement of about 5 meters will be assumed here, for the Phase A risk assessments. From the upper bound curve plotted by Hynes-Griffin and Franklin (1984), a displacement of 5 meters would correspond to a yield acceleration of about 0.03 times the peak acceleration along the slip surface. Then, assuming an amplification factor of 3 for the ground motions at the base of the embankment, this suggests $k_h = 0.1 \cdot PGA_{rock}$ can be used conservatively in the pseudostatic analysis to judge failure, as described above.

Pseudostatic factors of safety will not be computed in the Phase B assessment. Instead, where a liquefaction failure is not predicted, potential slope displacements will be computed as described in Step 6.

Step 6 – Predict Deformations

In the Phase A Portfolio Assessment, closed facilities that are expected to remain stable (pseudostatic $FS_{slope} \geq 1$ with no liquefaction, or post-earthquake $FS_{slope} \geq 1$ with liquefaction) will be assumed to have tolerable displacements. Dynamic slope deformations are difficult to estimate without detailed analysis; the available empirical or approximate methods do not represent the conditions of interest, or the level of effort is not consistent with the goals of the first phase of risk assessments. In addition, earthquake ground motion time histories will not be available for the Phase A analyses.

In the Phase B Facility Assessments, the potential deformation of stable slopes will be evaluated as indicated in Figure 4. Conventional methods of analysis will be implemented to estimate potential slope displacements that accumulate during earthquake shaking; movements are assumed to stop when the earthquake ends, consistent with a post-earthquake safety factor greater than one. The acceleration time histories obtained from the ground response analyses in Step 3a will be used as inputs for computing deformations with one of the following simplified methods:

- Newmark's (1965) method involves double integration of accelerations greater than the yield acceleration (k_y), which will be determined from a succession of pseudostatic slope stability analyses in which k_h is varied. The value of k_h where the pseudostatic FS_{slope} = 1.0 corresponds to the yield acceleration.
- The Makdisi-Seed (1978, 1979) procedure, which better accounts for the dynamic response of embankments. This procedure was developed based on parametric numerical simulations for earthen dams. The procedure is iterative, considers the fundamental periods of the embankment response, and can be completed in steps using published charts. Results from QUAKE can also be used as input in this procedure.

The slope deformations predicted in Phase B will be conservative, because the yield acceleration will be computed based on reduced, post-earthquake soil strengths. In reality, the yield acceleration declines in successive cycles of seismic loading, as pore pressures accumulate and saturated soils become weaker. The analysis outlined in Figure 4 assumes reduced strengths and, where liquefaction is predicted, residual strengths at the start of the earthquake. Detailed numerical simulations can be used to track the progressive softening and liquefaction of soil within an embankment during an earthquake; such analyses are expensive and time consuming. Rigorous analyses of this type will not be justified except in a "Phase C" analysis, or where performance in a given seismic design event must be demonstrated. Note that the logic in Figure 4 might appear to assume a slope will be stable if there is no significant liquefaction; however, the deformation analysis will indicate unlimited deformations and certain failure if $FS_{slope} < 1$ for static, post-earthquake conditions.

Step 7 – Consider Other Potential Failure Modes

For most of the closed facilities, soil liquefaction, slope instability, and slope deformations will be the most likely seismic failure modes. However, depending on the unique configuration of each CCP facility, other potential failure modes may contribute significantly to the seismic risks. For example, the loss of critical drainage structures or retaining walls could lead to a failure condition. Other potential failure modes will be identified and evaluated quantitatively in this step.

As a secondary objective of the Phase A effort, the assessment team will consider the potential for failure of the active storage facilities, due to an earthquake occurring prior to closure. Many of the wet CCP storage facilities retain large pools of water, so this assessment will need to consider additional failure modes such as seepage and embankment cracking. The objective here will be to identify operating facilities that may have unusually high seismic risks, and might deserve more study or higher priority in the closure program.

Step 8 – Estimate Annual Probability of Seismic Failure

As indicated in the flowcharts in Figures 3 and 4, the assessments of seismic performance (in both the Phase A and Phase B efforts) will consider a range of potential earthquakes with differing return periods. The analyses will be repeated until the limiting (lowest) earthquake return period (from the candidate events defined in Step 1) that predicts failure of a particular CCP storage facility is obtained. Interpolation may be used, as appropriate, to narrow the definition of the limiting earthquake.

The return period for each earthquake scenario (Table 4) represents the annual probability of exceedance for the associated ground motion parameter. Hence, for each earthquake scenario, the event with the smallest return period that causes failure represents a limiting case, where all events having longer return periods would also cause failure. The inverse of the limiting return period thus represents the annual probability of seismic failure due to that earthquake scenario.

Step 9 – Estimate Potential Consequences of Failure

The potential consequences of a failure at each closed facility will be estimated in this step. The potential consequences will be unique to each site, but may include any of the following:

- restoration of the site and storage facility,
- clean-up to address environmental impacts,
- off-site disposal of released materials,
- damages and loss of use for transportation routes, including buried or overhead utilities,
- damages to buildings and other infrastructure,
- economic losses from the possible shutdown of power generation, and
- loss of human life (expected to be unlikely at most sites following closure).

Except for the potential loss of life, the failure consequences will be expressed in terms of present day costs. Detailed cost estimates of the potential consequences of failure will not be attempted in the Phase A assessments; instead, the potential magnitude of total consequence costs will be estimated using broad categories (< $100K, < $500K, < $1M, < $5M, < $10M, < $50M, < $100M). Cost estimates that better reflect the local site conditions will be produced by the closure design teams during the Phase B assessments.

Step 10 – Estimate Possible Mitigation Costs

The final step in the process will involve estimating the costs to mitigate seismic risks, perhaps by altering the closure design to withstand stronger earthquakes. Examples of possible mitigation measures include:

- ground improvements to reduce liquefaction potential (stone columns, deep soil mixing, jet grouting, or other appropriate technology),
- altering the geometry of outslopes (setbacks, benches, or flatter slopes) to improve

stability,

- adding buttresses or other supporting structures at the toe of slopes,
- enhanced drainage features, and
- relocation of infrastructure or people away from potential impact zones.

These mitigation approaches generally involve higher construction costs, which can be quantified in terms of present dollars. As with the consequence costs, detailed estimates of mitigation costs will not be attempted in the Phase A assessments. The potential magnitude of mitigation will be estimated in categories (< $100K, < $500K, < $1M, < $5M, < $10M, < $50M, < $100M). Mitigation cost estimates that better reflect the local conditions and facility layout will be developed by the closure design teams during the Phase B assessments.

Table 1. Expected Results from the Phase A and B Seismic Risk Assessments

TVA Facility	Prob. Failure	Econ. Costs	Loss of Life	Mitigat. Costs	Data Quality
ALF East Ash Disposal					
ALF East Stilling Pond					
BRF Dry Fly Ash Disposal					
BRF Fly Ash Pond And Stilling Basin Area 2					
BRF Bottom Ash Disposal Area 1					
BRF Gypsum Disposal Area 2a					
COF Disposal Area 5					
COF Ash Pond 4					
CUF Dry Ash Stack					
CUF Ash Pond					
CUF Gypsum Storage Area					
GAF Fly Ash Pond E					
GAF Bottom Ash Pond A					
GAF Stilling Pond B, C & D					
JSF Dry Fly Ash Stack					
JSF Bottom Ash Disposal Area 2					
JOF Ash Disposal Area 2					
KIF Dike C					
PAF Scrubber Sludge Complex					
PAF Peabody Ash Pond					
PAF Slag Areas 2a & 2b					
SHF Consolidated Waste Dry Stack					
SHF Ash Pond					
WCF Ash Pond Complex					
WCF Gypsum Stack					

Prob Failure = Annual probability of failure due to earthquakes
Econ. Costs = Economic costs resulting from a failure
Loss of Life = Potential loss of life resulting from a failure
Mitigat. Costs = Costs to mitigate seismic risks in closure design
Data Quality = Qualitative indication of how well conditions in the facility are characterized

Table 2. Risk Severity Scoring (Draft) used by TVA

as of 4/22/2009

TVA Risk Event Consequence Rating Scale (Work-In-Progress)

Strategic Objective	Success Factor	5 Worst Case	4 Severe	3 Major	2 Moderate	1 Minor
Customer	Public Image	International media attention; nearly unanimous public criticism	National media attention; federal, state officials, and customers publicly critical	Regional / local media attention; customers voice concern	Minimal media attention; letters / emails to executive leadership voicing concern	No media attention; sparse criticism
	Rate Impact	Average total retail rate increases by 15%, relative to peers	Average total retail rate increases by 10%-15%, relative to peers	Average total retail rate increases by 5%-10%, relative to peers	Average total retail rate increases by 2%-5%, relative to peers	Average total retail rate increases by 0-2%, relative to peers
People	Safety	Fatalities	Wide spread injuries	Major injuries	Significant injuries	Minor injuries
	Employee Confidence	Widespread departures of key staff with scarce skills or knowledge	Sharp, sustained drop in CHI results; departures of key staff with scarce skills or knowledge	Sharp decline in CHI results	Modest decline in CHI results	No effect on CHI results
Financial	Cash Flow Impact	>$500M	$100M - $500M	$25M - $100M	$5M - $25M	<$5M
	Credit Worthiness	Credit rating downgrade to below investment grade	Credit Rating Downgrade	TVA put on credit watch	TVA put on negative outlook	Credit rating agencies and bondholders express concern
Assets and Operations	LNS (Load not served)*	10% of System Daily Sales (48,000 MWhrs)	1% of System Daily Sales (4,800 MWhrs)	0.1% of System Daily Sales (480 MWhrs)	0.05% of System Daily Sales (240 MWhrs)	140 MWhrs
	CPI (Connection Point Interruptions)	10% of CPs are down simultaneously	5% of CPs are down simultaneously	CPI totaling 10% of current CP count (124 for FY09)	CPI totaling 7.5% of current CP count (93 for FY09)	CPI totaling 5% of current CP count (62 for FY09)
	Duration (in Hours) of Service Interruption	3,000 cumulative hours for CPs	1,000 cumulative hours for CPs	500 cumulative hours for CPs	150 cumulative hours for CPs	50 cumulative hours for CPs
	Delivered Cost of Power	Sustained increase in delivered cost of power >1 year	Increase in delivered cost of power <1 year	Increase in delivered cost of power <1 month	Increase in delivered cost of power <1 week	Delivered cost of power not effected
	Damage to environment; type and magnitude of contamination / discharge	Major coal, nuclear plant accident or dam failure	Significant hazardous waste discharged; nuclear plant accident; dam integrity failure resulting in drawdown of pool elevation	Hazardous materials / waste discharge; clean up / remediation time takes approximately two weeks	Localized environmental damage, no impact to wildlife, clean up / remediation time less than two weeks	Minimal environmental damage, no hazardous discharge; clean up time takes a few days

Table 3. Risk Likelihood Scoring used by TVA

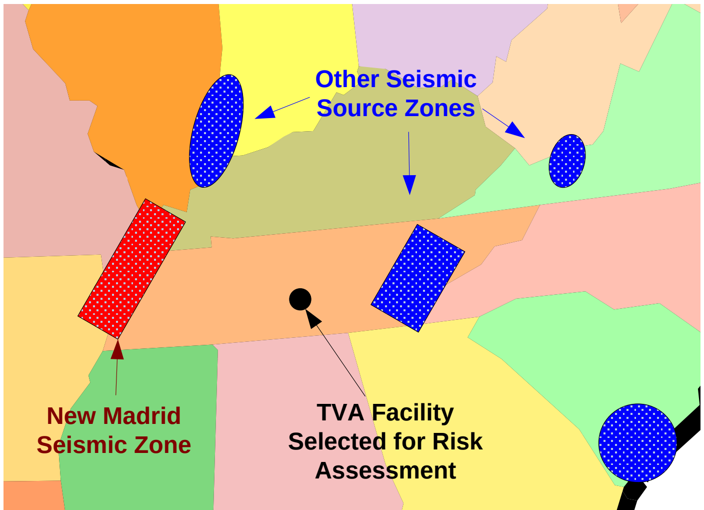
TVA Risk Event Probability Rating Scale		
Score	Rating	Description
5	Virtually Certain	95% probability that the event will occur in the next 3 years /10 years
4	Very Likely	75% probability that the event will occur in the next 3 years/10 years
3	Even Odds	50% probability that the event will occur in the next 3 years/10 years
2	Unlikely	25% probability that the event will occur in the next 3 years/10 years
1	Remote	5% probability that the event will occur in the next 3 years/10 years

- *The 3-year timeframe will be the primary focus for the business unit risk maps*
- *The 10-year risks will be collected by the ERM organization and charted separately for the enterprise*

Table 4. Seismic Hazard Input Data for Probabilistic Assessment of TVA Facilities

Seismic Sources	Return Period (years)	Annual Probability of Exceedance	Peak Ground Acceleration (g)	Earthquake Magnitude
New Madrid Seismic Zone	2,500 1,000 500 250 100	0.0004 0.001 0.002 0.004 0.01	*Values to be determined from the seismic hazard curves*	*Values to be determined from the hazard de-aggregation data**
All Other Seismic Sources	2,500 1,000 500 250 100	0.0004 0.001 0.002 0.004 0.01		

** Representative magnitude corresponding to the maximum contribution to the seismic hazard for liquefaction, as determined from the de-aggregation data weighted by the magnitude scaling factor (maximum PGA / MSF)*



Note: Schematic representation only, locations not accurately depicted, some sources omitted.

Figure 1. Schematic Representation of Seismic Source Model for TVA Facilities

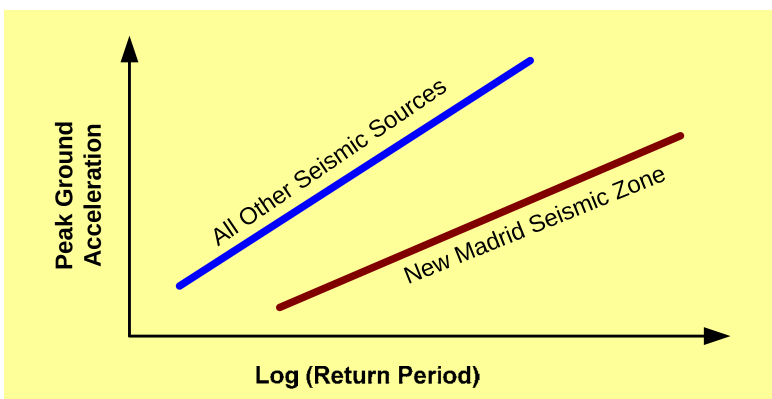


Figure 2. Typical Seismic Hazard Curves for Proposed Probabilistic Assessment of TVA Facilities

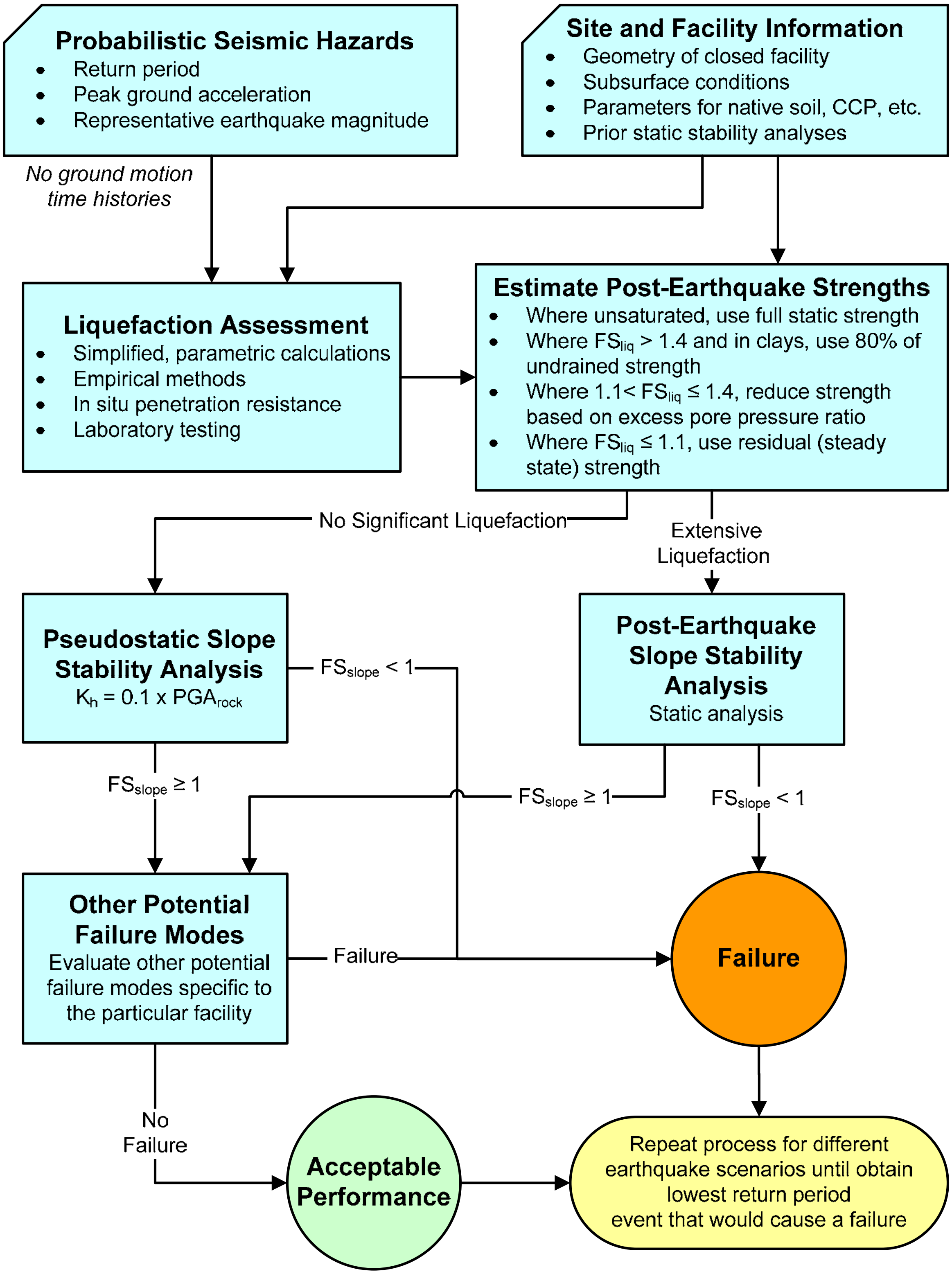


Figure 3. Simplified Flowchart for Assessing Facility Performance During a Probabilistic Seismic Event in Phase A

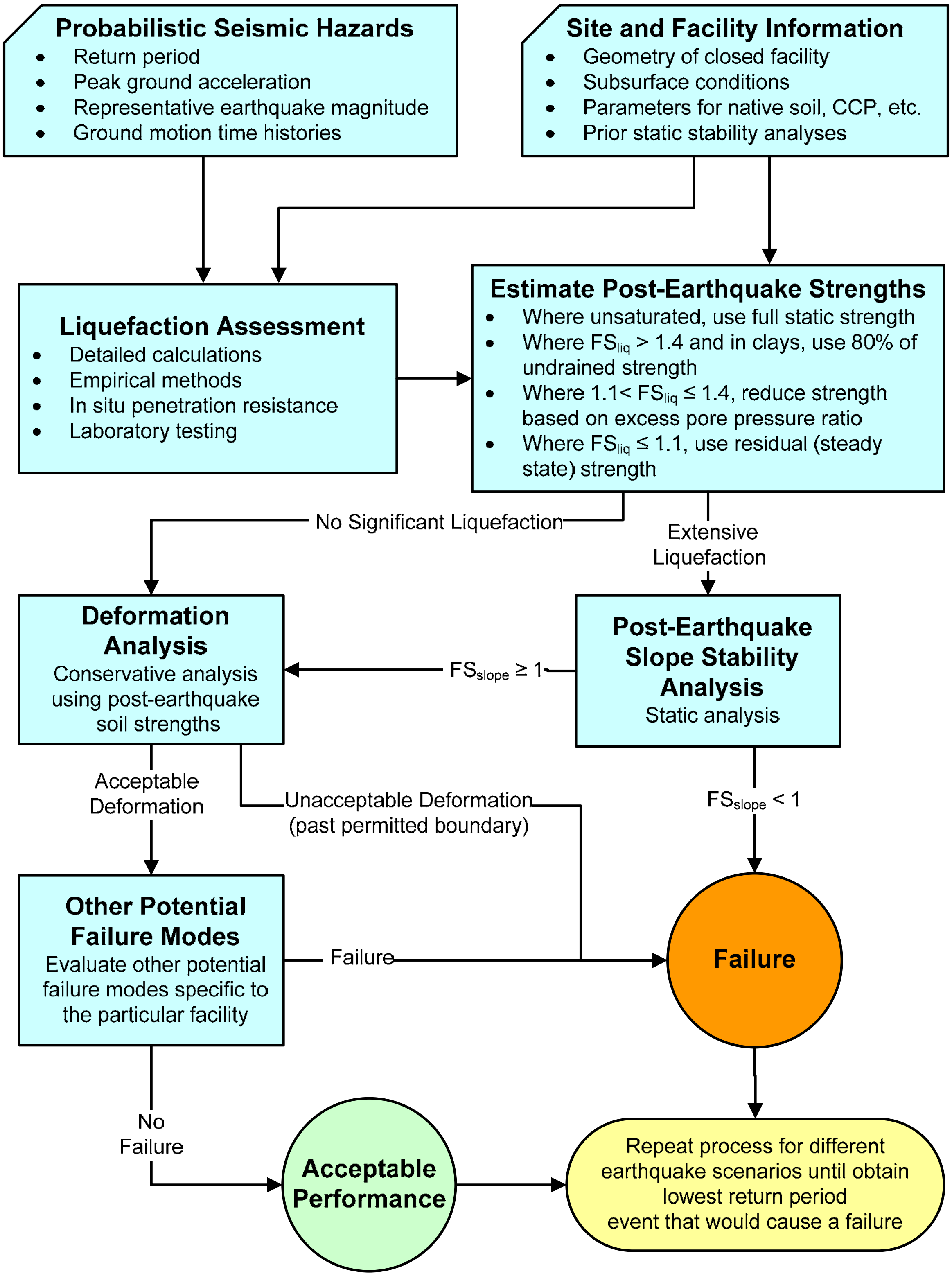


Figure 4. Simplified Flowchart for Assessing Facility Performance During a Probabilistic Seismic Event in Phase B

APPENDIX A

Document 5

CDM Smith Report – Revision No. 1. Stability Analysis, April 30, 2012

CDM Smith
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April 30, 2012

Mr. James D. Mullins, P.E.
Senior Program Manager
Tennessee Valley Authority
CCP Engineering
1101 Market Street, LP 5E-C
Chattanooga, TN 37402

Subject: Report – Revision No. 1
Existing Conditions Stability Analyses
Ash Pond Area at Watts Bar Fossil Plant

Dear Mr. Mullins:

The purpose of this letter report is to present the results of the existing conditions stability analyses performed by CDM Smith for the Ash/Stilling Pond area at the Watts Bar Fossil (WBF) plant near Spring City, Tennessee. These analyses were performed to support the U.S. Environmental Protection Agency's assessment of the Tennessee Valley Authority's (TVA) Coal Combustion Products (CCP) disposal facilities.

Project Background

The WBF plant is directly downstream of the Watts Bar Dam and Lock and abuts the west bank of the Tennessee River. Currently, the WBF plant is not operational and decommissioning is underway. The WBF plant was a coal-fired power plant built by TVA between 1940 and 1945. The plant was operated in two stages, from end of construction to 1957 and from 1970 to 1982. During the plant operation, ash and boiler slag generated by the plant were stockpiled and stored on-site.

In 2010, TVA contracted with CDM Smith to perform Phase I preliminary design services to support final closure of the WBF plant as part of the WBF Plant Coal Combustion Products Closure Project. The final closure encompasses multiple areas which include disposal facilities, impoundments, and stormwater ponds permitted in accordance with multiple regulations. The project includes closure of five (5) main areas: (i) the Borrow Source Area, (ii), Slag Processing Area (iii), Chemical Pond Area, (iv) Ash/Stilling Pond Area, and (v) Riverbank Area, as shown on **Figure 1**.

As part of this work, TVA requested that CDM Smith provide an existing condition evaluation for the stability of the Ash/Stilling Pond Area. This evaluation considered stability of the Ash/Stilling

Pond Area under static and seismic loading conditions based upon available data, as described herein.

Available Information

During the preliminary design phase for the closure, CDM Smith reviewed the following available information provided by TVA:

- MACTEC geotechnical report at Borrow Area
- QA/QC reports for closure construction at Chemical Pond and Slag Disposal Area during 2006 to 2009.
- TVA Disposal Facility Assessment, Phase I Plant Summary, Watts Bar Fossil Plant (WBF), by Stantec, 2009.
- Final Report – Development of Hazard Deaggregation Inputs for Use in Risk Analysis of Fossil Plants, by AMEC GeoMatrix, March 2010.
- Watts Bar Fossil Plant, Annual Inspection Report for Ash/Waste Disposal Areas, from 1967 through 2008.
- Watts Bar Fossil Plant, Slag Disposal Area Closure Plan, Project Planning Document, approved by TVA in January 2007.
- Fly Ash, Bottom Ash, and Scrubber Gypsum Study, by Law Engineering, November 1995.
- TVA Coal Combustion Products Management Program, Master Programmatic Document (Revision 1.0), December 7, 2009.

In addition, CDM Smith performed the following site-specific investigations to supplement the available data:

- Site walk and surficial soil sampling in the Borrow Area and Slag Disposal Area in August 2011.
- Bulk sampling and laboratory testing of underwater ash samples from the Ash/Stilling Pond Area in August 2011.

CDM Smith

- Site survey of the Slag Disposal Area and Ash/Stilling Pond Area in December of 2011. Survey was performed by TVA at the request of CDM Smith.

- Subsurface exploration program along west bank of Tennessee River consisting of three geotechnical borings and installation of two groundwater observation wells.

The subsurface exploration program was completed in January 2012 and boring logs and water level readings from the wells are currently available and included as **Attachment A**. Laboratory testing results for disturbed and undisturbed samples collected in these borings are also contained in Attachment A.

Existing Conditions Evaluation

The existing conditions stability analyses for the Ash/Stilling Pond Area were performed at two critical cross-sections, as shown on **Figure 2**. The two critical cross-sections were selected at locations exhibiting the steepest exterior embankment slopes and riverbank slopes. The locations of the geotechnical borings completed in January 2012 are also shown on this figure. Cross-section A-A' extends through the Wet Ash Pond Area and Cross-Section B-B' extends through the Dry Ash Area, as shown on **Figures 3A** and **3B**, respectively. The cross-sections were developed based upon available topographic survey, design plans for the ponds, and the subsurface conditions encountered in the test borings. Currently there are no bathymetric survey data available for the river bank slope below normal water level. For this evaluation, the river bank slope was assumed to follow the same natural slope above water level and extend to the top of the bedrock at the river bed.

Selection of Design Parameters

The engineering design properties of the ash and soil layers for the seepage and stability analysis of the cross-sections are summarized in **Tables 1a and 1b**. The basis for selection of the design properties is also listed in the tables. In general, ash properties were estimated based upon available data from similar TVA facilities and soil properties were estimated based upon the subsurface investigation data, empirical correlations, and experience in similar geologic conditions.

Table 1a: Parameters used in SEEP/W Seepage Analyses

Layer	**Material**	k_h ft/day	k_h cm/sec	k_h / k_v	**Basis of Parameter Selection**
	Ash Material	0.45	1.6E-04	20	Based on comparison between laboratory testing data for existing ash material and TVA's CCP material database
1	Fill	0.0028	1.0E-06	10	From Peck[1]; typical value for mixture of sand, clay, and silt.
2A	Medium Stiff to Stiff Clay	0.0014	5.0E-07	15	From Peck; typical value for low-permeability soil.
2B	Soft Clay and Silt	0.0014	5.0E-07	15	From Peck; typical value for low-permeability soil.
3	Sand	2.83	1.0E-03	4	From Peck; typical value for sand.
4	Weathered Rock and Gravel	28.35	1.0E-02	4	From Peck; typical value for sand and gravel mixtures.
5	Interbedded Shale and Limestone Bedrock	0.0006	2.0E-07	1	From Domenico[2]; page 39; high-end value for Shale bedrock.

Reference:

1. Ralph B. Peck, 'Foundation Engineering', 2nd edition, 1974; page 43.

2. Patrick A. Domenico, 'Physical and Chemical Hydrogeology', 2nd edition, 1997.

Table 1b: Strength Parameters used in SLOPE/W Stability Analyses

Layer	Material	Unit Weight, pcf	Friction Angle, degrees	Undrained Shear Strength, psf	Basis of Parameter Selection(1)
	Ash Material (wet)	70	20	-	Comparison between laboratory testing data for existing ash material and TVA's CCP material database
.	Ash Material (dry)	85	25	-	
1	Fill	120/115(2)	32	-	Selected based on lower 1/3 N-values(3) from B-2 and B-3
2A	Medium Stiff to Stiff Clay	110/105(2)	29	1300	Selected based upon lower 1/3 pocket penetrometer readings(4) from B-2 and B-3 and laboratory shear strength testing on U-1 from B-2.
2B	Soft Clay and Silt	110	28	500	Selected based upon N-values and pocket penetrometer readings(4) from B-2 and B-3
3	Sand	120	30	-	Selected based on Lower 1/3 N-values(3) from B-2 and B-3
4	Weathered Rock and Gravel	125	40	-	Based upon experience in similar geologic conditions
5	Bedrock	Impenetrable			Assumed

Notes:

1. Correlation of N-value and friction angle from Ralph B. Peck, 'Foundation Engineering', 2nd edition, 1974; page 310.
2. Values listed are saturated/moist unit weights.
3. Lower 1/3 value is defined as the value where at least 2/3 of all the readings are greater or equal. N-value is defined as the sum of the blows to drive the 2nd and 3rd 6-inch-increments of each split spoon sample.
4. Pocket penetrometer readings were performed on split spoon samples and Shelby tube sample during drilling.

Laboratory strength data was not available at the time of the issue of the original report. However, laboratory results for an undisturbed Shelby Tube sample (U-1) taken in the medium to stiff clay layer (Layer 2A) in boring B-2 are now available. Based upon the laboratory results and the insitu data from borings B-2 and B-3 (pocket penetrometer), the undrained shear strength for this layer has been revised to 1300 psf. The undrained shear strength data for Layer 2A and Layer 2B are shown on **Figure 4**.

For stability analyses under seismic conditions, peak ground acceleration for the WBF plant site was selected based upon a review of the *"Final Report - Development of Hazard Deaggregation Inputs for Use in Risk Analysis of Fossil Plants"*, by AMEC GeoMatrix, dated March 28, 2010 and the USGS 2008 Hazards Map available at http://earthquake.usgs.gov. **Table 2** summarizes the data from the three TVA plants closest to WBF and the USGS Hazard Map values for the WBF. Based upon these data, the peak ground acceleration for WBF was interpolated to be 0.042g for a 500-year return period and a 0.116g for a 2500-year return period. Ground motion corresponding to a 2500-year return period is consistent with seismic stability guidance provided by the TVA Master Programmatic Document (Revision 1.0). A peak ground acceleration of 0.116g was used in the stability analyses under seismic conditions.

Table 2: Summary of Available Seismic Hazards Results (AMEC Report and USGS)

Plant	Latitude	Longitude	Return Period (years)	Probability of Exceedance	PGA, (g)	
					AMEC Report	USGS
Bull Run	36.00	-84.15	2500	2% in 50 years	0.131	0.155
			500	10% in 50 years	0.043	0.044
Kingston	35.90	-84.51	2500	2% in 50 years	0.115	0.134
			500	10% in 50 years	0.041	0.041
Watts Bar	35.61	-84.78	2500	2% in 50 years	***0.116***	0.135
			500	10% in 50 years	***0.042***	0.042
Widows Creek	34.90	-85.75	2500	2% in 50 years	0.1	0.115
			500	10% in 50 years	0.038	0.038

Bolded values were interpolated from tabulated data.

Seepage Analyses and Results
The phreatic surface for each stability analysis was developed from seepage analyses performed with the SEEP/W 2007 software package by GEO-SLOPE International, Ltd. This computer program uses the inputted geometry, soil, rock, and ash properties, and boundary conditions (surface water and groundwater conditions) to develop a steady-state seepage profile. For our analyses, the model was calibrated using the field data gathered during the recent geotechnical investigation by CDM Smith including:

- Water levels observed in the Ash/Stilling Pond
- Groundwater levels measured in the observation wells
- River level elevation data available at "http://www.tva.gov/lakes/wbh_o.htm" for the dates/times.

Once the model was calibrated, the steady-state phreatic surface was developed for normal pool conditions in the Ash/Stilling Pond (EL. 705).

Static Slope Stability Analyses and Results
Analyses for overall (global) stability under static conditions were performed using the SLOPE/W 2007 modeling software package from GEO-SLOPE. This computer program uses the inputted slope geometry, soil, rock, and ash properties, and phreatic surface and calculates the factors of safety against deep-seated circular failures. Phreatic surfaces generated by SEEP/W were imported to SLOPE/W for the static and seismic slope stability analyses. The Spencer method was selected for the slope stability analyses. The minimum acceptable static factor of safety against overall slope failure is 1.5 for normal pool conditions.

Effective stress strength parameters were used for all materials in static analyses. The stability analyses are included in **Attachment B** and the minimum factors of safety for deep-seated circular failure surfaces are presented in **Table 3**. Failure surfaces less than 5 feet deep are considered to be sloughing/surficial failures. The stability analyses did exhibit some lower factors of safety for sloughing/surficial failures along the river bank, but these failure surfaces did not extend into the pond berm such that the global stability of the ash pond would be impacted. Results presented herein considered the deep-seated failures that extend into the ash pond areas only. All factors of safety for static conditions equal or exceed the minimum required.

Table 3: Results of Slope Stability Analyses – Static Conditions

Run #	Modeling Scenario	Calculated Factor of Safety	
		Inboard Slope	Outboard Slope
A-1	Static Slope Stability at Wet Pond	1.9	1.8
B-1	Static Slope Stability at Dry Ash Area	2.4	1.5

Seismic Slope Stability Analyses and Results

The stability analyses under seismic loading conditions were performed using a pseudostatic method, where the added inertial load from an earthquake is represented by a horizontal pseudostatic coefficient. Based upon the Standard Penetration Test N-values and fines content of the subsurface soils, the soils at the site are not considered to be susceptible to liquefaction. The analyses assumed no liquefaction of the subsurface soils and undrained shear strength parameters were used for the natural clay soils (Layer 2A and Layer 2B). The peak ground acceleration was estimated as 0.116g. Tolerable deformations were assumed for cases where the pseudostatic factor of safety is greater than 1.0. Normal pool conditions were assumed (El. 705).

The stability analyses are included in Attachment B and the minimum factors of safety for deep-seated circular surfaces are presented in **Table 4**. All factors of safety for seismic (pseudostatic) conditions equal or exceed the minimum required.

Table 4: Results of Slope Stability Analyses – Seismic Conditions

Run #	Modeling Scenario	Calculated Factor of Safety	
		Inboard Slope	Outboard Slope
A-2	Seismic Conditions at Wet Pond	1.4	1.1
B-2	Seismic Conditions at Dry Ash Pond	1.3	1.0

Conclusions

The slope stability analyses indicate acceptable factors of safety under static and seismic loading conditions for all sections. The seismic slope stability analyses presented in this letter use a pseudostatic approach to represent existing conditions. For seismic assessment of the closure design, TVA will employ a comprehensive risk-based approach, with design and mitigation decisions based upon the probability and consequences of failure. This approach is outlined in the document *"Seismic Risk Assessment, Closed CCP Storage Facilities, Tennessee Valley Authority"* dated March, 2010 and included as **Attachment C**.

Limitations

This letter report has been prepared for specific application to the subject project in accordance with generally accepted geotechnical engineering practices. No other warranty, express or implied, is made. In the event that any changes occur, the conclusions and recommendations presented in this memorandum should not be considered valid, unless changes are reviewed and conclusions of this memorandum are modified or verified in writing.

STEPHEN L. WHITESIDE
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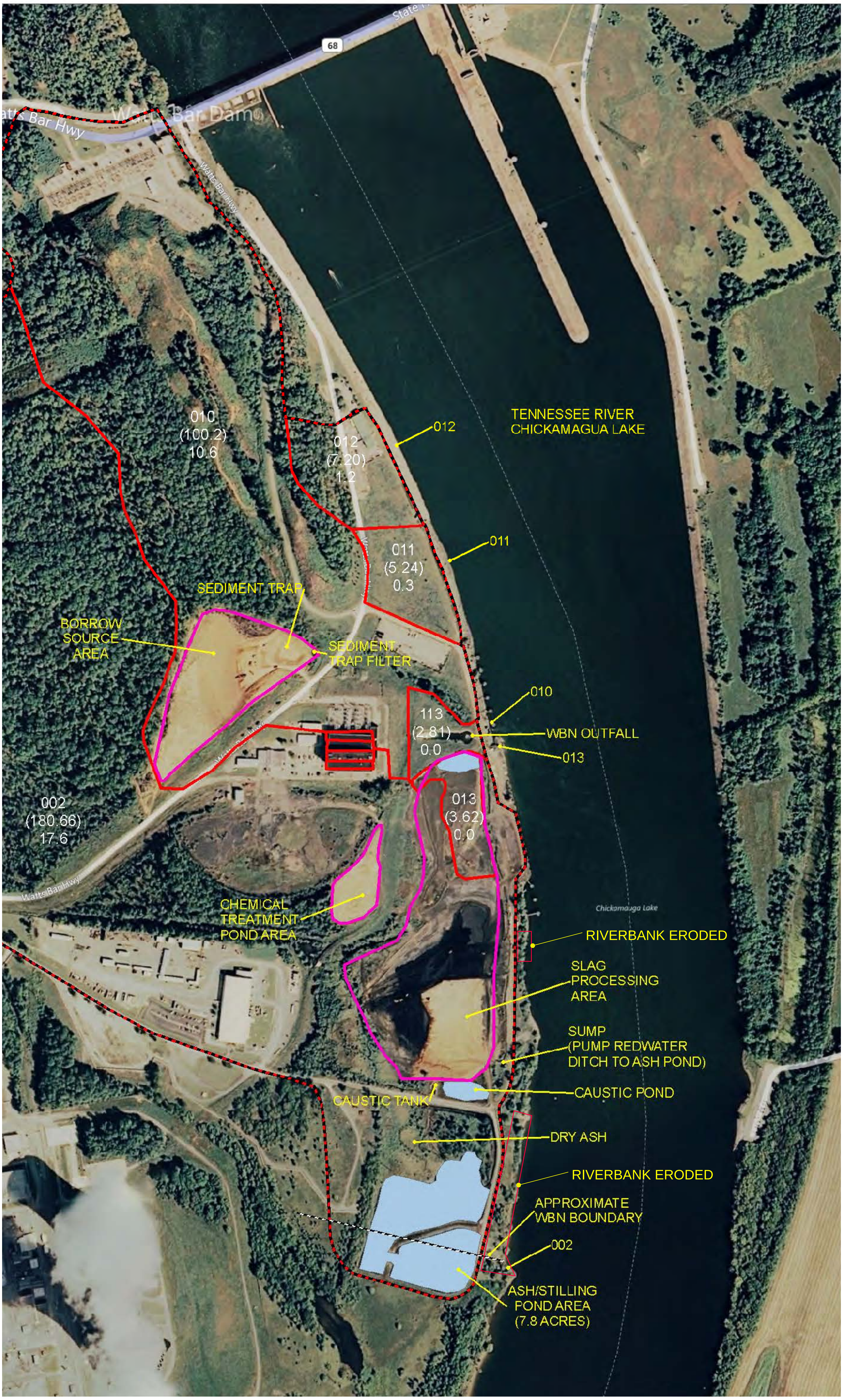
4/30/12

Very truly yours,

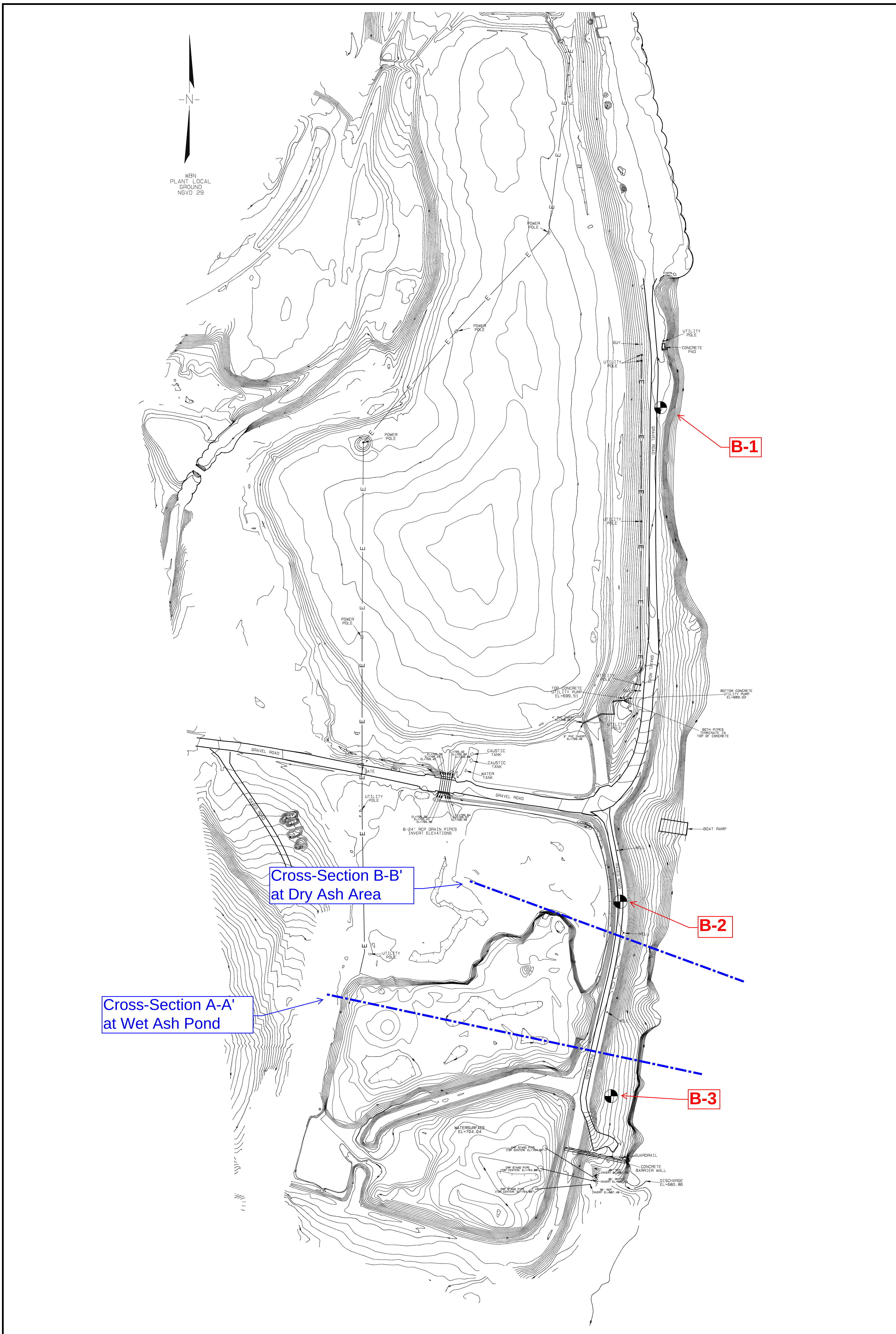
Stephen L. Whiteside, P.E.
Vice President
CDM Smith Inc.

cc: Michael Bachand, CDM Smith
Jintao Wen, CDM Smith
Danielle Neamtu, CDM Smith

FIGURES



CDM Smith	**Watts Bar Fossil Plant** **Tennessee Valley Authority** **Coal Combustion Products Closure Project**	**Locations of Closure Areas** **Figure 1**

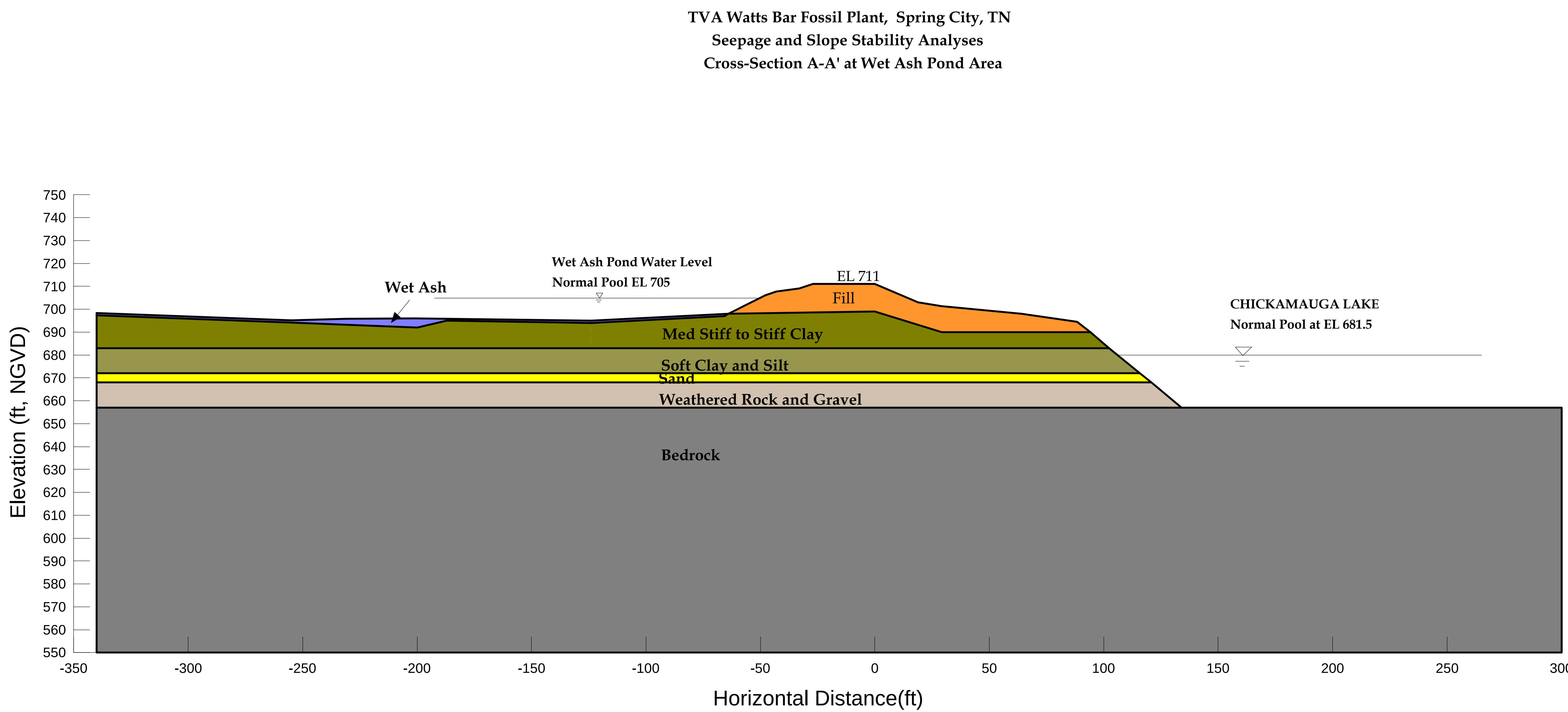


CDM Smith

Watts Bar Fossil Plant
Tennessee Valley Authority
Coal Combustion Products Closure Project

Borings and Cross-Section Locations

Figure 2

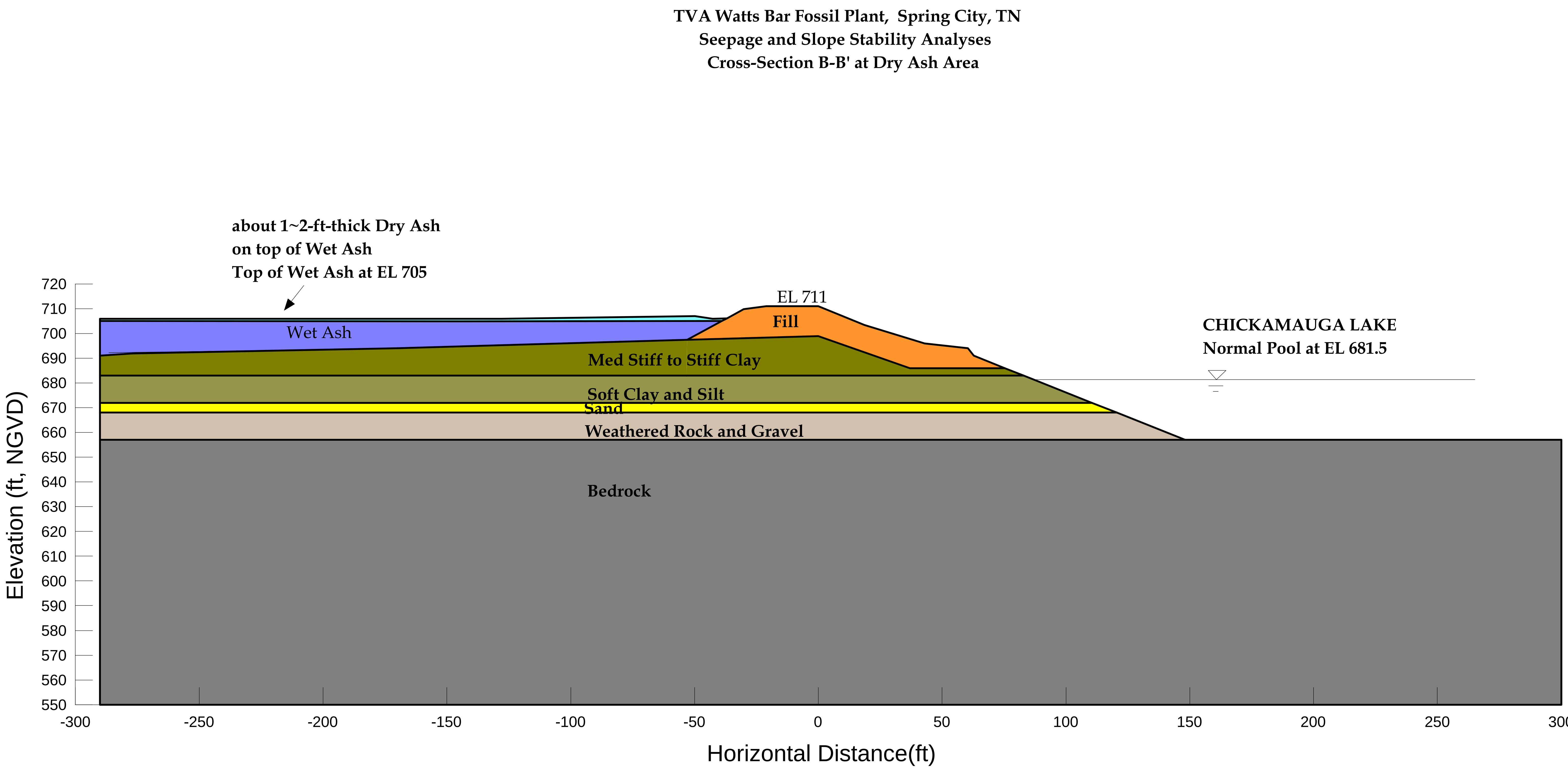


CDM Smith

Watts Bar Fossil Plant
Tennessee Valley Authority
Coal Combustion Products Closure Project

Cross-Section A-A' at Wet Ash Pond Area

Figure 3A



CDM Smith

Watts Bar Fossil Plant
Tennessee Valley Authority
Coal Combustion Products Closure Project

Cross-Section B-B' at Dry Ash Area

Figure 3B

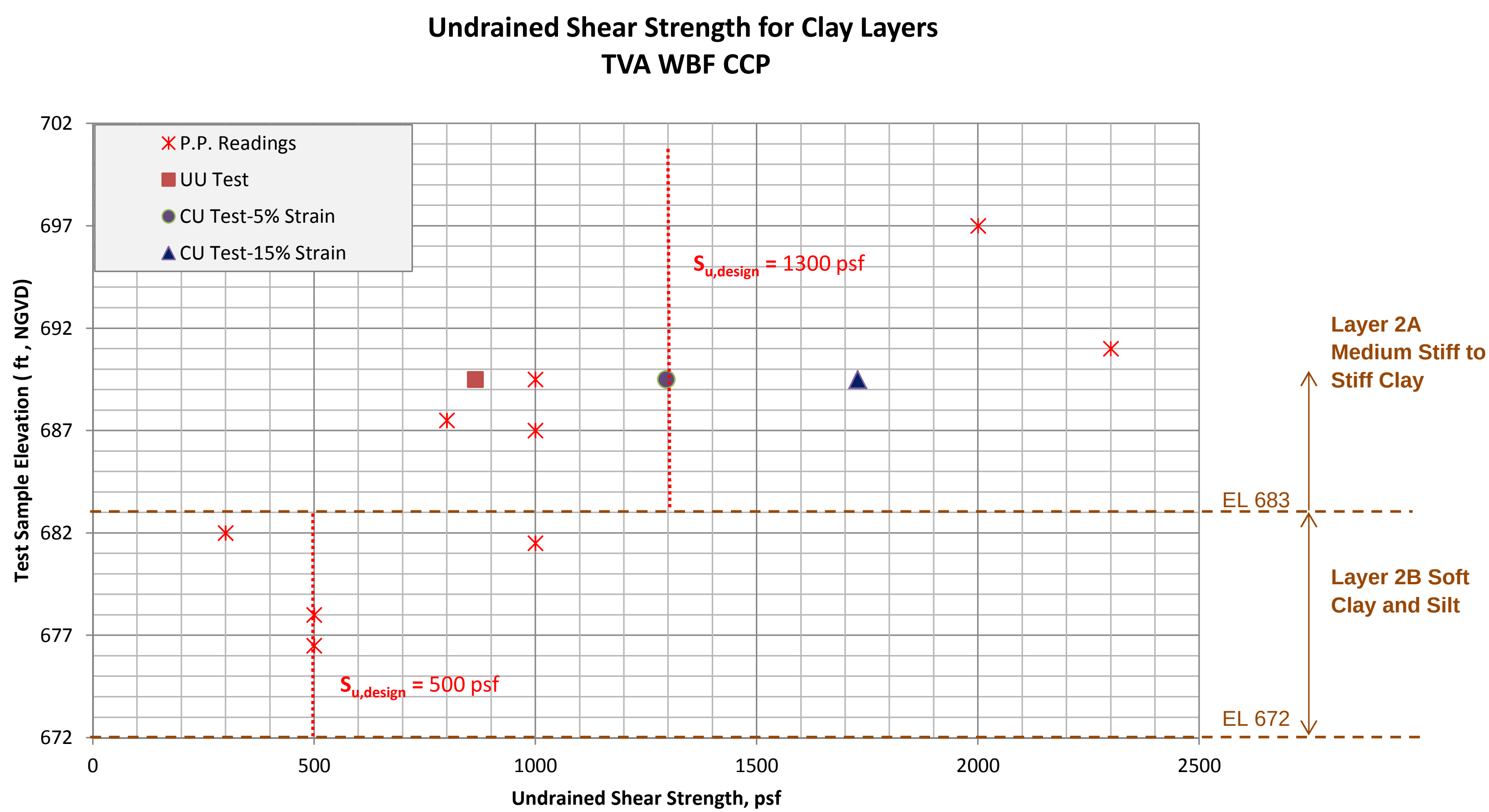


Figure 4. Undrained Shear Strength for Natural Clays

ATTACHMENT A

CDM Smith

BOREHOLE LOG
B-1

Client: TVA
Project Name: TVA Watts Bar Fossil Plant
Project Location: Spring City, TN
Project Number: 83529

Drilling Contractor: Total Depth Drilling
Surface Elevation (ft.): 699
Drilling Method/Rig: 3.25" HSA/CME-55
Total Depth (ft.): 44.6
Drillers: Tim Hall
Depth to Initial Water Level (ft-bgs): 9.3
Drilling Date: Start: 11-16-11 **End:** 11-17-11
Abandonment Method: Converted to observation well
Borehole Coordinates:
Field Screening Instrument:
N 466,232.90 E 2,331,561.10
Logged By: M. Howe

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			699.0 / 0				GW	2-inches GRAVEL.
SS	S-1	12/11			10, 40		FILL	Moist to wet, dense to very dense, tan-brown and gray, GRAVEL and SILT. **-FILL-**
SS	S-2	24/22			12, 17, 27, 22			Moist, dense, dark brown and yellow-brown, fine to coarse SAND, little silt, gravel, trace clay.
SS	S-3	24/18		0.25	13, 19, 21, 11			Moist, hard, orange-brown to blue-gray and tan, SILT, some sand.
SS	S-4	24/20	694.0 / 5	1.0	3, 8, 5, 6			Moist, stiff, tan to blue-gray mottling, CLAY, trace silt, sand, and wood fragments.
SS	S-5	24/16		0.75	2, 3, 2, 2			Moist, medium stiff, tan to blue-gray, CLAY, trace silt, sand, and gravel.
SS	S-6	24/18	689.0 / 10	0.5	3, 3, 4, 6			Moist, medium stiff, medium brown to tan-brown, SILT, some sand, trace gravel.
SS	S-7	24/18	684.0		2, 2, 2, 2		SC/SM	Wet, very loose to loose, fine SAND, some clay, little silt. **- ALLUVIAL SOIL -**

EXPLANATION OF ABBREVIATIONS

DRILLING METHODS:
HSA - Hollow Stem Auger
SSA - Solid Stem Auger
HA - Hand Auger
AR - Air Rotary
DTR - Dual Tube Rotary
FR - Foam Rotary
MR - Mud Rotary
RC - Reverse Circulation
CT - Cable Tool
JET - Jetting
D - Driving
DTC - Drill Through Casing

SAMPLING TYPES:
AS - Auger/Grab Sample
CS - California Sampler
BX - 1.5" Rock Core
NX - 2.1" Rock Core
GP - Geoprobe
HP - Hydro Punch
SS - Split Spoon
ST - Shelby Tube
WS - Wash Sample
OTHER:
AGS - Above Ground Surface

REMARKS

Hammer weight = 140 pounds, drop height = 30 inches
Split spoon = 2 inches OD, 24 inches long

Borehole coordinates are approximate based upon handheld GPS and elevations are estimated by overlaying coordinates with the survey.

Reviewed by: Danielle Neamtu **Date:** 4-25-12

CDM Smith

BOREHOLE LOG
B-1

Client: TVA
Project Name: TVA Watts Bar Fossil Plant
Project Location: Spring City, TN
Project Number: 83529

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. / Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			684.0 / 15				SC/SM	
SS	S-8	24/24	679.0 / 20	0.75	2 3 4 5		CL	Moist to wet, medium stiff, red-brown to tan-brown, CLAY, little to some sand.
SS	S-9	24/24	674.0 / 25	0.5	2 4 4 5			Moist to wet, medium stiff to stiff, orange-brown to gray-tan, CLAY, some silt, trace to little sand.
SS	S-10	24/24	669.0 / 30		1 2 3 7		SM/SC	Wet, loose, gray to tan, fine SAND, little silt, clay.
SS	S-11	15/10			11 67 100/3"		SC/SM	Moist to wet, very dense, gray, fine to coarse SAND, little clay, silt, trace gravel. **-WEATHERED ROCK-** Auger refusal at 33.0 feet below ground surface. Split-spoon refusal at 34.3 feet below ground surface.
NQ	C-1	63/6	664.0 / 35		2:45 1:45 2:15 8:30 3:15		GW	**RUN 1: 34.3 to 39.6 feet-bgs** REC = 9.5%, RQD = 0% Moderately hard, highly weathered, green and brown to gray, aphanitic, INTERBEDDED SHALE, LIMESTONE, and RIVER ROCK; extremely thin bedding, low angle jointing, very close spacing, rough, discolored, open, quartz vugs.

CDM Smith

BOREHOLE LOG
B-1

Client: TVA
Project Name: TVA Watts Bar Fossil Plant
Project Location: Spring City, TN
Project Number: 83529

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
					3:00		GW	
NQ	C-2	60/7.5	659.0 / 40		8:15, 5:00, 4:15, 6:45, 3:30		SHALE/LS	**RUN 2: 39.6 to 44.6 feet-bgs** REC = 12.5%, RQD = 0% Moderately hard to hard, highly weathered, gray, aphanitic, INTERBEDDED SHALE and LIMESTONE; very thin to extremely thin bedding, low angle jointing, very close to close spacing, rough, discolored, open, calcite veins.
			654.0 / 45					Boring terminated at 44.6 feet below ground surface.
			649.0 / 50					
			644.0 / 55					
			639.0 / 60					

BOREHOLE-PP READINGS/NO ROCK TVA WATTS BAR FOSSIL PLANT.GPJ CDM_CORP.GDT 4/25/12

CDM Smith

BOREHOLE LOG
B-2

Client: TVA
Project Name: TVA Watts Bar Fossil Plant
Project Location: Spring City, TN
Project Number: 83529

Drilling Contractor: Total Depth Drilling
Surface Elevation (ft.): 711
Drilling Method/Rig: 3.25" HSA/CME-55
Total Depth (ft.): 46.1
Drillers: Allan Fowler
Depth to Initial Water Level (ft-bgs): 27.4
Drilling Date: Start: 1-10-12 **End:** 1-10-12
Abandonment Method: Grouted to ground surface
Borehole Coordinates:
Field Screening Instrument:
N 465,036.40 E 2,331,471.00
Logged By: M. Howe

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			711.0 0				ASPHALT	3-inches ASPHALT PAVEMENT.
							GW	8-inches GRAVEL BASE.
SS	1	24/23		3.0	5 6 8 9		FILL	Moist, stiff, orange brown, CLAY, **-FILL-**
SS	2	24/24		>4.5	3 6 10 14			Moist, very stiff, orange brown, CLAY, some silt, trace gravel. Moist, very stiff, dark brown, CLAY, some silt, trace gravel.
SS	3	24/24	706.0 5	>4.5	5 8 11 12			Moist, very stiff, dark brown with gray mottling, CLAY, some silt.
SS	4	24/24		4.0	5 8 10 11			Moist, very stiff, dark brown with light brown and gray mottling, CLAY, some silt.
SS	5	24/24	701.0 10	4.5	4 6 7 9			Moist, stiff, dark brown with gray and light brown mottling, CLAY, some silt.
SS	6	24/14	696.0	2.0	3 6 7		CH	Moist, stiff, orange to yellow brown, CLAY, little sand (in lenses). **- ALLUVIAL SOIL -**

EXPLANATION OF ABBREVIATIONS

DRILLING METHODS:
HSA - Hollow Stem Auger
SSA - Solid Stem Auger
HA - Hand Auger
AR - Air Rotary
DTR - Dual Tube Rotary
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JET - Jetting
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DTC - Drill Through Casing

SAMPLING TYPES:
AS - Auger/Grab Sample
CS - California Sampler
BX - 1.5" Rock Core
NX - 2.1" Rock Core
GP - Geoprobe
HP - Hydro Punch
SS - Split Spoon
ST - Shelby Tube
WS - Wash Sample
OTHER:
AGS - Above Ground Surface

REMARKS

Hammer weight = 140 pounds, drop height = 30 inches
Split spoon = 2 inches OD, 24 inches long

Borehole coordinates are approximate based upon handheld GPS and elevations are estimated by overlaying coordinates with the survey.

Reviewed by: Danielle Neamtu **Date:** 4-25-12

CDM Smith

BOREHOLE LOG
B-2

Client: TVA
Project Location: Spring City, TN
Project Name: TVA Watts Bar Fossil Plant
Project Number: 83529

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			696.0 15		8		CH	
SS	7	24/24	691.0 20	2.3	3 3 5 5		CL	Moist, medium stiff to stiff, medium brown to tan, CLAY, trace to little sand.
ST	1	24/24		1.0				Shelby tube sample collected from 20.5 to 22.5 feet below ground surface. Moist to wet, medium brown, CLAY, little silt, trace sand.
SS	8	24/19		0.8	2 3 3 3			Moist, medium stiff, medium brown, CLAY, trace to little silt.
			686.0 25					
SS	9	24/24	681.0 30	1.0	1 2 3 3			Moist to wet, medium stiff, medium brown, CLAY, little silt, trace sand.
SS	10	24/24	676.0 35	0.5	1 2 2 2			Wet, soft to medium stiff, medium brown, CLAY, some silt, little sand.
					1			Wet, loose, medium brown, fine to medium SAND, trace silt.

CDM Smith

BOREHOLE LOG
B-2

Client: TVA
Project Name: TVA Watts Bar Fossil Plant
Project Location: Spring City, TN
Project Number: 83529

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
SS	11	24/24	671.0 / 40		2 4 6		SP-SM	
SS	12	23/23			1 11 13			Wet, medium dense, medium brown, fine to medium SAND, trace silt.
			666.0 / 45		100/5"		SW/GW	Wet, very dense, gray, fine to coarse SAND and GRAVEL, trace silt. **-WEATHERED ROCK-**
SS	13	1/1			100/1"			Auger refusal at 46.0 feet below ground surface. Split-spoon refusal at 46.1 feet below ground surface.
			661.0 / 50					
			656.0 / 55					
			651.0 / 60					

CDM Smith

BOREHOLE LOG
B-3

Client: TVA
Project Location: Spring City, TN
Project Name: TVA Watts Bar Fossil Plant
Project Number: 83529

Drilling Contractor: Total Depth Drilling
Drilling Method/Rig: 3.25" HSA/CME-55
Drillers: Tim Hall
Drilling Date: Start: 11-15-11 **End:** 11-16-11
Borehole Coordinates:
N 464,593.80 E 2,331,431.10

Surface Elevation (ft.): 701
Total Depth (ft.): 54.8
Depth to Initial Water Level (ft-bgs): 18.1
Abandonment Method: Converted to observation well
Field Screening Instrument:
Logged By: M. Howe

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			701.0 / 0				TOPSOIL	2-inches TOPSOIL.
SS	S-1	24/18		3.5	2, 4, 5, 6		FILL	Moist, stiff, medium brown to dark brown, CLAY, trace sand, **-FILL-**
SS	S-2	24/24		1.0	4, 7, 12, 9			Moist, very stiff, medium brown to dark brown with orange, CLAY, trace sand.
SS	S-3	24/20	696.0 / 5	2.0	4, 6, 6, 5			Moist, stiff, medium brown with orange, SILT, some sand.
SS	S-4	24/22		1.0	6, 5, 7, 5			Moist, medium dense, medium brown to orange-brown, fine SAND, little silt.
SS	S-5	24/19		1.0	4, 4, 7, 5			Moist, stiff, medium brown to orange-brown, CLAY, little sand. Moist, medium dense, medium brown to orange-brown, fine SAND, some silt, clay.
			691.0 / 10					
SS	S-6	24/22		1.0	3, 4, 5, 5		CL	Moist to wet, stiff, medium brown, CLAY, little silt. **- ALLUVIAL SOIL -**
			686.0					

EXPLANATION OF ABBREVIATIONS

DRILLING METHODS:
HSA - Hollow Stem Auger
SSA - Solid Stem Auger
HA - Hand Auger
AR - Air Rotary
DTR - Dual Tube Rotary
FR - Foam Rotary
MR - Mud Rotary
RC - Reverse Circulation
CT - Cable Tool
JET - Jetting
D - Driving
DTC - Drill Through Casing

SAMPLING TYPES:
AS - Auger/Grab Sample
CS - California Sampler
BX - 1.5" Rock Core
NX - 2.1" Rock Core
GP - Geoprobe
HP - Hydro Punch
SS - Split Spoon
ST - Shelby Tube
WS - Wash Sample
OTHER:
AGS - Above Ground Surface

REMARKS

Hammer weight = 140 pounds, drop height = 30 inches
Split spoon = 2 inches OD, 24 inches long

Borehole coordinates are approximate based upon handheld GPS and elevations are estimated by overlaying coordinates with the survey.

Reviewed by: Danielle Neamtu **Date:** 4-25-12

CDM Smith

BOREHOLE LOG
B-3

Client: TVA
Project Name: TVA Watts Bar Fossil Plant
Project Location: Spring City, TN
Project Number: 83529

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. / Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			686.0 / 15				CL	
SS	S-7	24/10	681.0 / 20	0.3	1 1 1 2		CL-ML	Wet, very soft to soft, medium brown to tan-brown, SILT and CLAY, little sand.
SS	S-8	24/24	676.0 / 25	0.5	2 1 2 2		CL	Wet, soft, medium brown to tan-brown, CLAY, some silt, trace sand.
SS	S-9	24/24	671.0 / 30		2 1 2 3		SP-SM	Wet, very loose, medium brown to gray-brown, fine SAND, little silt.
SS	S-10	24/15	666.0 / 35		2 8 11 12			Wet, medium dense, tan to gray, fine to coarse SAND, some gravel, trace silt.
SS	S-11	8/8			48 100/2"		SM/SC	Wet, very dense, gray, fine to coarse SAND, little silt, clay, trace gravel. **-WEATHERED ROCK-**

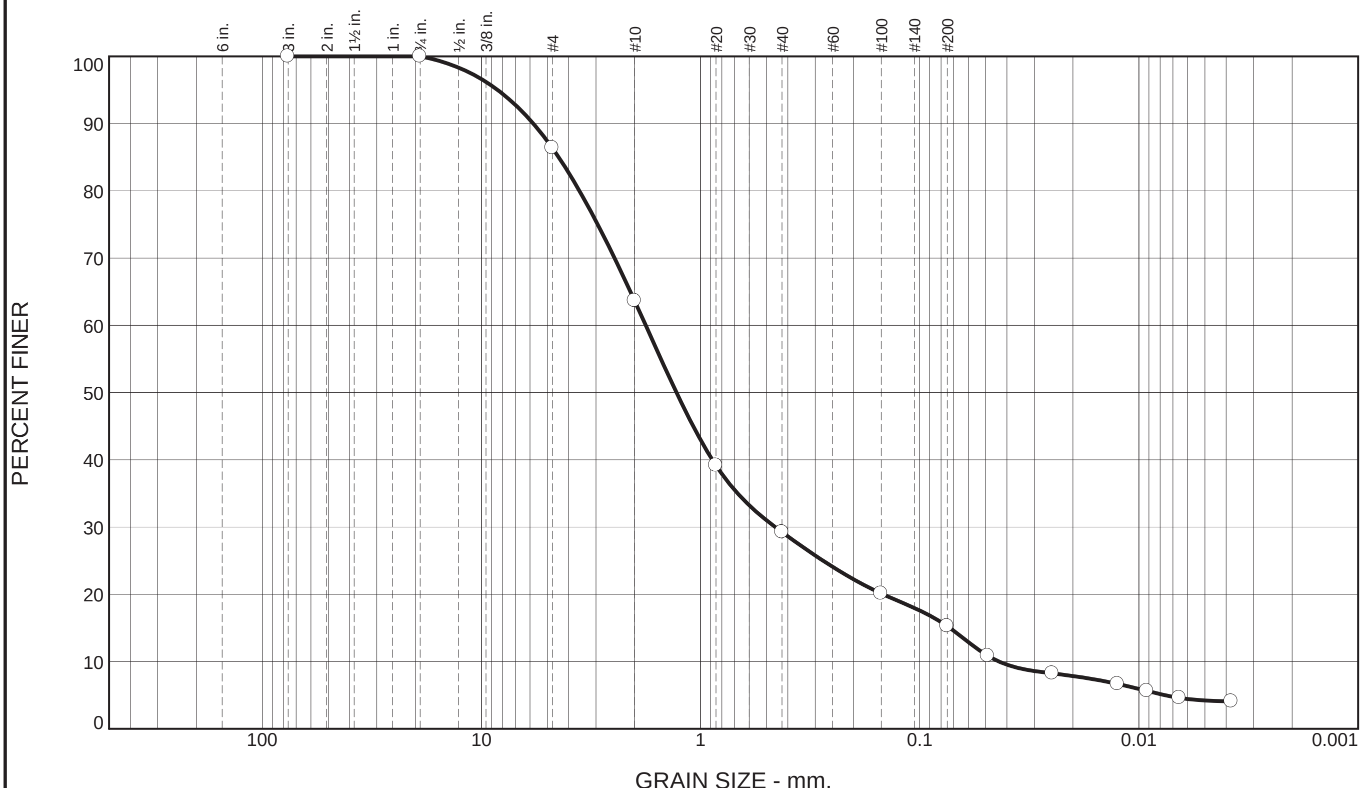
CDM Smith

BOREHOLE LOG
B-3

Client: TVA | **Project Name:** TVA Watts Bar Fossil Plant
Project Location: Spring City, TN | **Project Number:** 83529

Sample Type	Sample Number	Sample Adv/Rec (inches)	Elev. Depth (ft.)	Pocket Penetrometer Reading (tsf)	Blows per 6-in	Graphic Log	USCS Designation	Material Description
			661.0 / 40				SM/SC	Split-spoon refusal at 38.7 feet below ground surface. Auger refusal at 40.4 feet below ground surface.
NQ	C-1	52.8/6			7:30, 6:00, 6:00, 5:15, 2:00		GW	**RUN 1: 40.4 to 44.8 feet-bgs** REC = 9%, RQD = 0% Moderately hard to hard, highly weathered, brown and orange to gray, aphanitic, interbedded SHALE, LIMESTONE, and RIVER ROCK; extremely thin bedding, low angle jointing, very close spacing, rough, discolored, open, calcite veins.
NQ	C-2	60/14	656.0 / 45		4:30, 7:00, 6:00, 7:15, 8:15		SHALE/LS	**RUN 2: 44.8 to 49.8 feet-bgs** REC = 23%, RQD = 0% Moderately hard to hard, highly weathered,gray, aphanitic, interbedded LIMESTONE and SHALE; very thin bedding, low angle jointing, very close spacing, rough, discolored, open, calcite veins.
NQ	C-3	60/9.5	651.0 / 50		9:45, 16:15, 7:30, 8:15, 6:45		SHALE/LS	**RUN 3: 49.8 to 54.8 feet-bgs** REC = 16%, RQD = 0% Moderately hard, highly weathered,gray, aphanitic, interbedded LIMESTONE and SHALE; extremely thin to very thin bedding, low angle jointing, very close spacing, rough, discolored, open.
			646.0 / 55					Boring terminated at 54.8 feet below ground surface.
			641.0 / 60					

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	13.6	22.8	34.4	13.9	11.1	4.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3	100.0		
3/4	100.0		
#4	86.4		
#10	63.6		
#20	39.1		
#40	29.2		
#100	20.1		
#200	15.3		

* (no specification provided)

Material Description

Silty Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 5.8002 D_{85}= 4.4393 D_{60}= 1.7795
D_{50}= 1.2880 D_{30}= 0.4565 D_{15}= 0.0730
D_{10}= 0.0435 C_u= 40.87 C_c= 2.69

Classification

USCS= SM AASHTO=

Remarks

As received moisture content=6.9%
Soil classification and description based on
Visual Manual Procedure ASTM D2488

Source of Sample: B-1 **Depth:** 1-3
Sample Number: S-2

Date: 11/16/2011

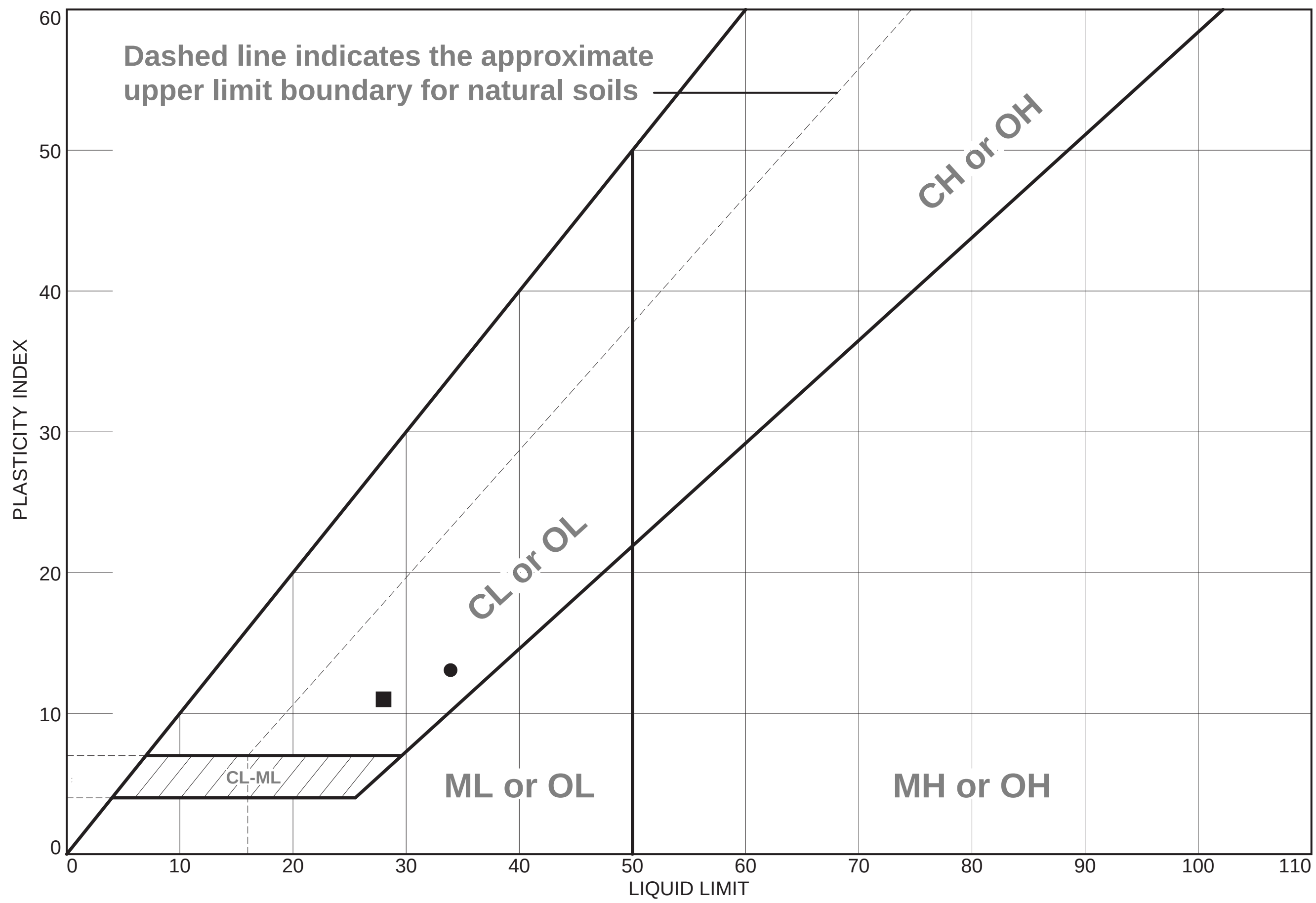
CDM Smith

Cambridge, Massachusetts

Client: TVA
Project: Watts Bar Fossil Plant CCP Closure
Project No: 95618-83529 **Figure**

Tested By: NE **Checked By:** MR

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean clay	34	21	13			CL
■	Lean clay	28	17	11			CL

Project No. 95618-83529 **Client:** TVA

Project: Watts Bar Fossil Plant CCP Closure

●**Source of Sample:** B-1 **Depth:** 5-7 **Sample Number:** S-4

■**Source of Sample:** B-1 **Depth:** 23-25 **Sample Number:** S-9

Remarks:

●As received moisture content=16.2%

■As received moisture content=23.0%

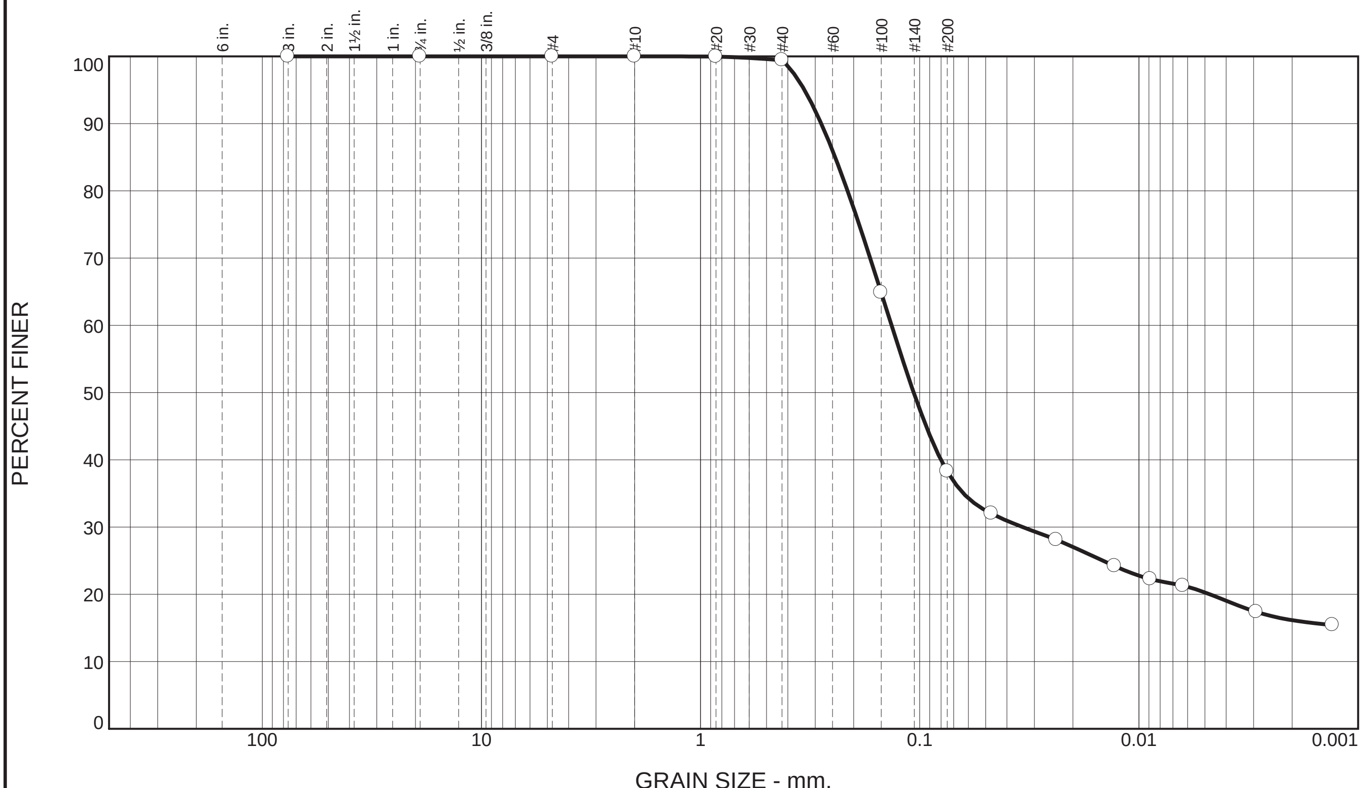
CDM Smith

Cambridge, Massachusetts

Figure

Tested By: NE **Checked By:** MR

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.6	61.1	18.0	20.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3	100.0		
3/4	100.0		
#4	100.0		
#10	100.0		
#20	100.0		
#40	99.4		
#100	64.9		
#200	38.3		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2815 D_{85}= 0.2433 D_{60}= 0.1345
D_{50}= 0.1066 D_{30}= 0.0339 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

As received moisture content=20.2%
Soil classification and description based on Visual Manual Procedure ASTM D2488

Source of Sample: B-1 **Depth:** 13-15
Sample Number: S-7

Date: 11/16/2011

CDM Smith

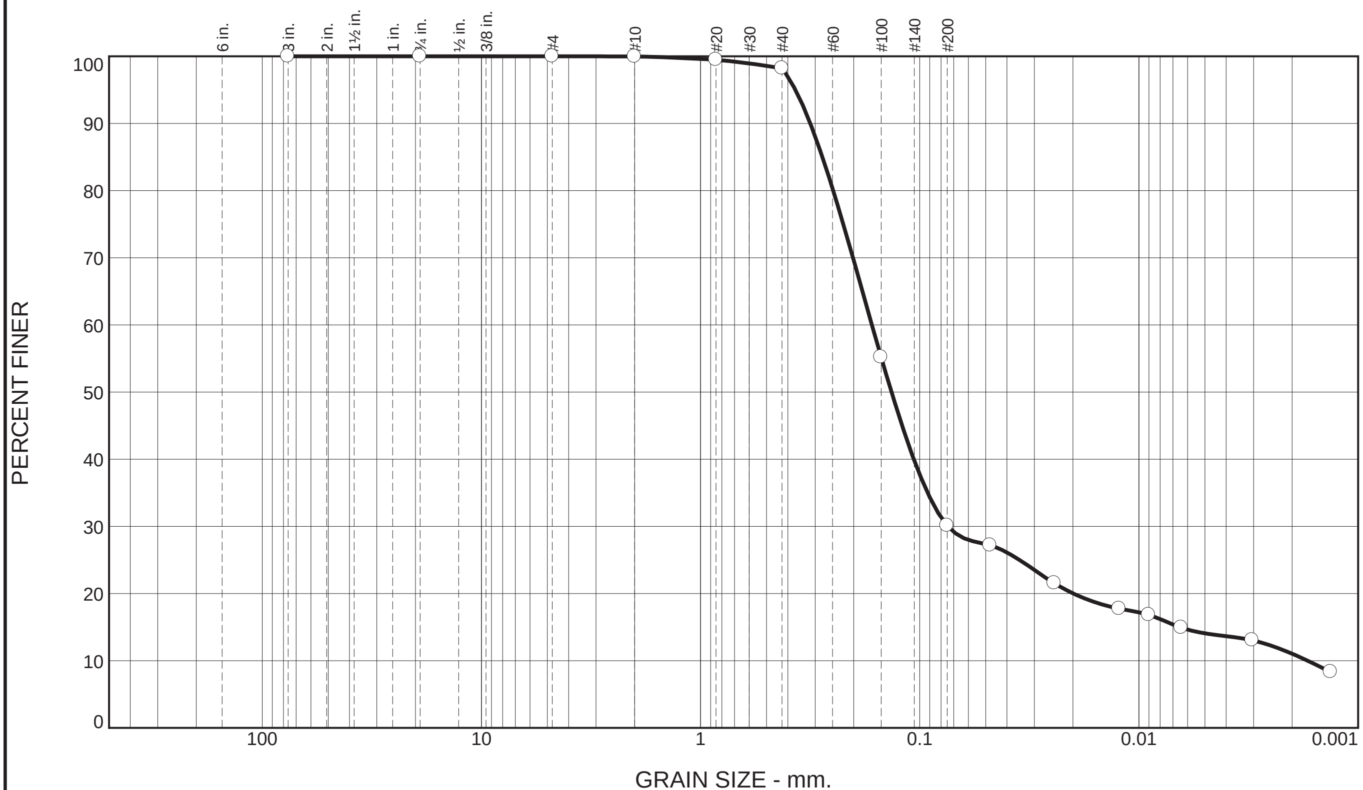
Cambridge, Massachusetts

Client: TVA
Project: Watts Bar Fossil Plant CCP Closure

Project No: 95618-83529 **Figure**

Tested By: NE **Checked By:** MR

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	1.8	68.1	16.0	14.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3	100.0		
3/4	100.0		
#4	100.0		
#10	100.0		
#20	99.5		
#40	98.2		
#100	55.2		
#200	30.1		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3160 D_{85}= 0.2782 D_{60}= 0.1653
D_{50}= 0.1346 D_{30}= 0.0744 D_{15}= 0.0065
D_{10}= 0.0017 C_u= 97.09 C_c= 19.66

Classification

USCS= SM AASHTO=

Remarks

As received moisture content=34.5%
Soil classification and description based on Visual Manual Procedure ASTM D2488

Source of Sample: B-1 **Depth:** 28-30
Sample Number: S-10

Date: 11/16/2012

CDM Smith

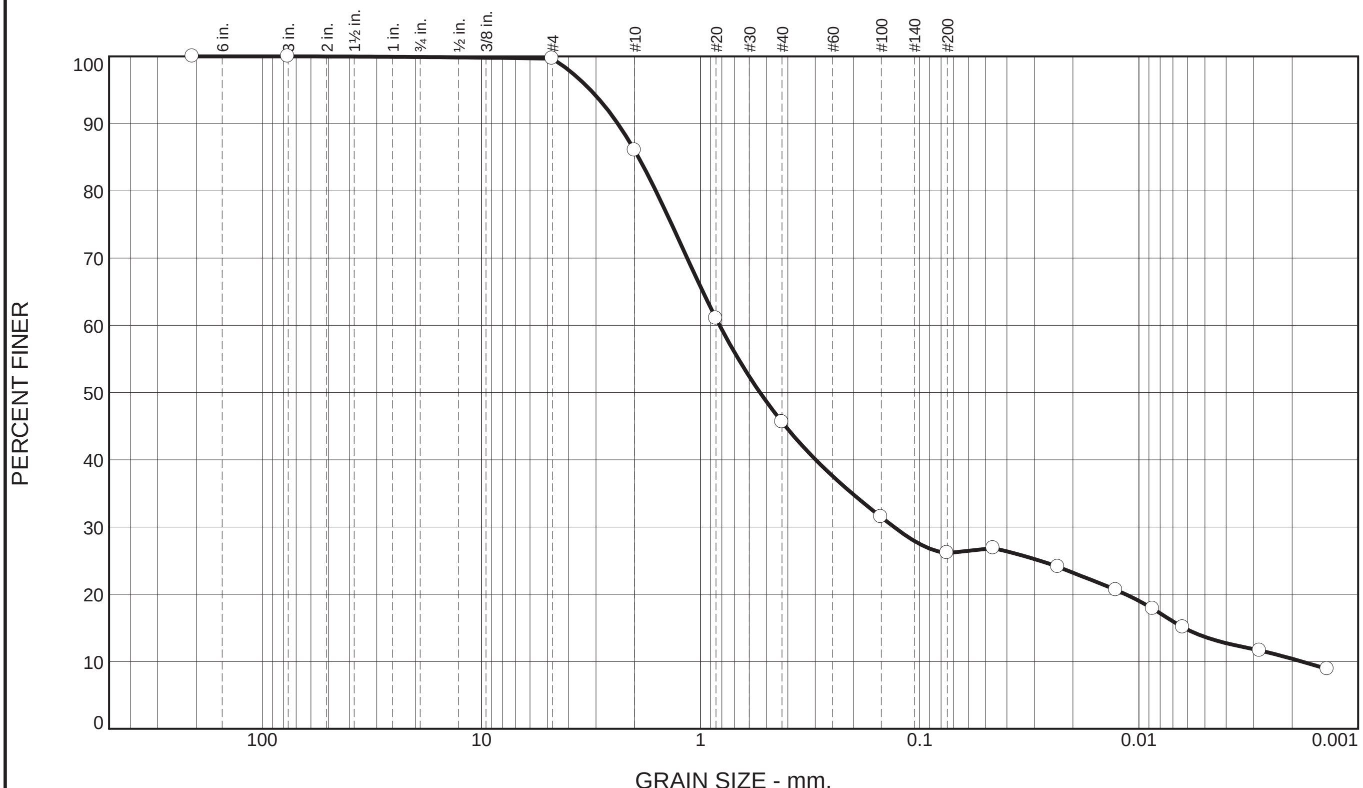
Cambridge, Massachusetts

Client: TVA
Project: Watts Bar Fossil Plant CCP Closure

Project No: 95618-83529 **Figure**

Tested By: NE **Checked By:** MR

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.1	0.2	13.7	40.4	19.5	12.4	13.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
8.19	100.0		
3	100.0		
#4	99.7		
#10	86.0		
#20	61.0		
#40	45.6		
#100	31.5		
#200	26.1		

* (no specification provided)

Material Description

Silty sand
Note: Portion of sample soft, weathered rock easily broken into smaller fractions during sample preparation.

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.3878 D_{85}= 1.9212 D_{60}= 0.8189
D_{50}= 0.5357 D_{30}= 0.1310 D_{15}= 0.0062
D_{10}= 0.0018 C_u= 453.07 C_c= 11.59

Classification

USCS= SM AASHTO=

Remarks

As received moisture content=7.4%
Soil classification and description based on Visual Manual Procedure ASTM D2488

Source of Sample: B-1 **Depth:** 33-34.5
Sample Number: S-11

Date: 11/16/2011

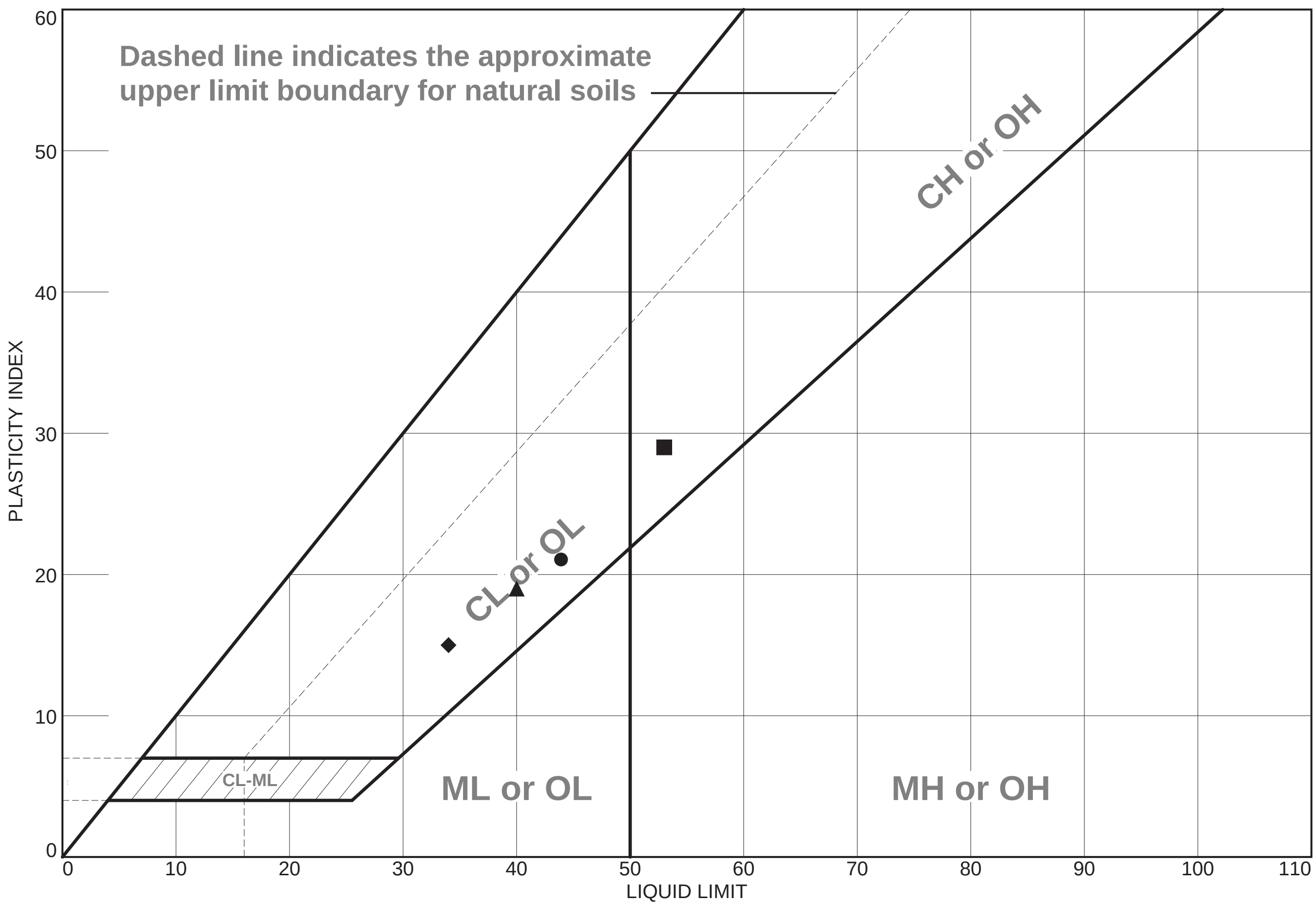
CDM Smith

Cambridge, Massachusetts

Client: TVA
Project: Watts Bar Fossil Plant CCP Closure
Project No: 95618-83529 **Figure**

Tested By: NE **Checked By:** MR

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean clay	44	23	21			CL
■	Fat clay	53	24	29			CH
▲	Lean clay	40	21	19			CL
◆	Lean clay	34	19	15			CL

Project No. 95618-83529 **Client:** TVA

Project: Watts Bar Fossil Plant CCP Closure

●**Source of Sample:** B-2 **Depth:** 5-7 **Sample Number:** S-3
■**Source of Sample:** B-2 **Depth:** 13.5-15.5 **Sample Number:** S-6
▲**Source of Sample:** B-2 **Depth:** 28.5-30.5 **Sample Number:** S-9
◆**Source of Sample:** B-2 **Depth:** 20.5-22.5 **Sample Number:** U-1

Remarks:
●As received moisture content=21.7%
■As received moisture content=20.6%
▲As received moisture content=27.8%
◆As received moisture content=14.6%

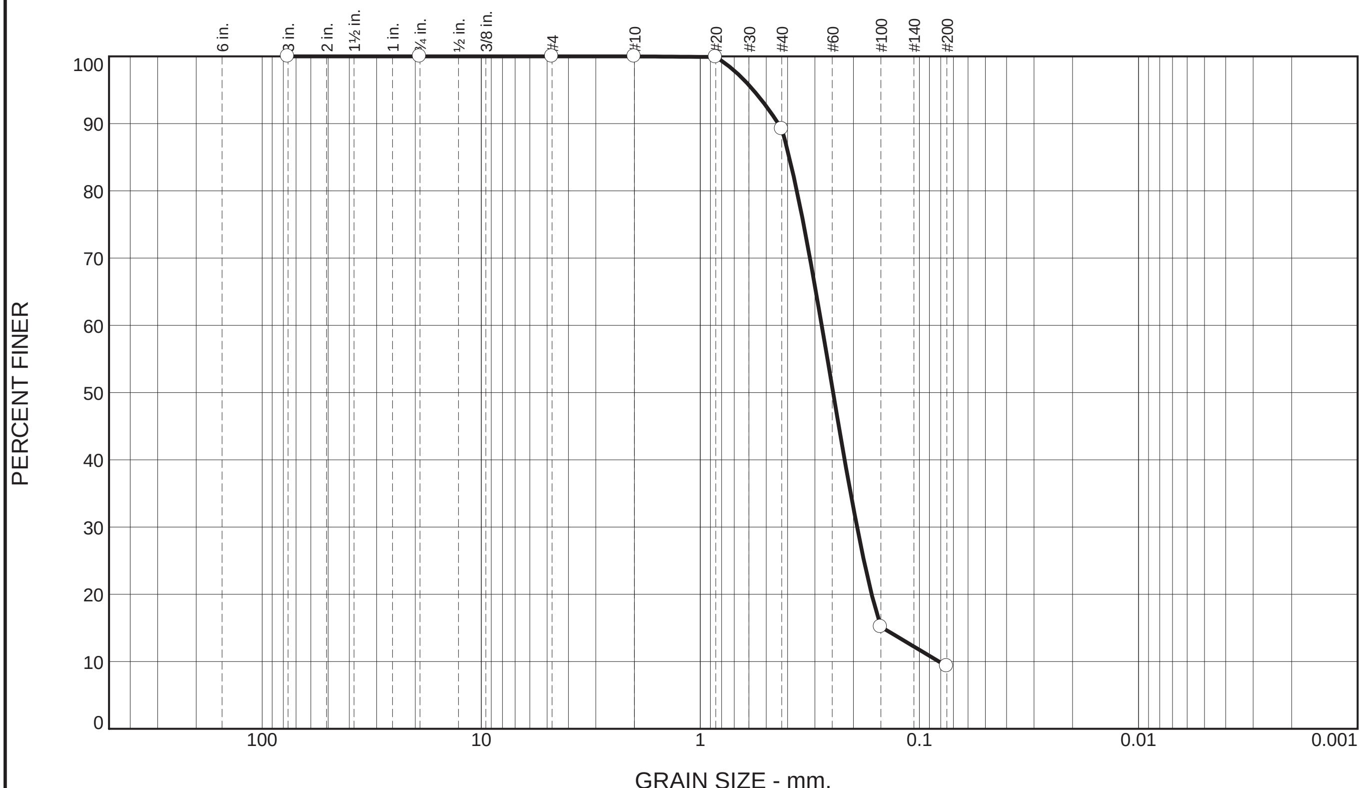
CDM Smith

Cambridge, Massachusetts

Figure

Tested By: NE **Checked By:** MR

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	10.8	79.9	9.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3	100.0		
3/4	100.0		
#4	100.0		
#10	100.0		
#20	99.9		
#40	89.2		
#100	15.2		
#200	9.3		

* (no specification provided)

Material Description

Poorly graded sand with silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.4406 D_{85}= 0.3927 D_{60}= 0.2792
D_{50}= 0.2477 D_{30}= 0.1926 D_{15}= 0.1470
D_{10}= 0.0814 C_u= 3.43 C_c= 1.63

Classification

USCS= SP-SM AASHTO=

Remarks

As received moisture content=27.6%
Soil classification and description based on Visual Manual Procedure ASTM D 2488

Source of Sample: B-2 **Depth:** 38.5-40.5
Sample Number: S-11

Date: 1/10/2012

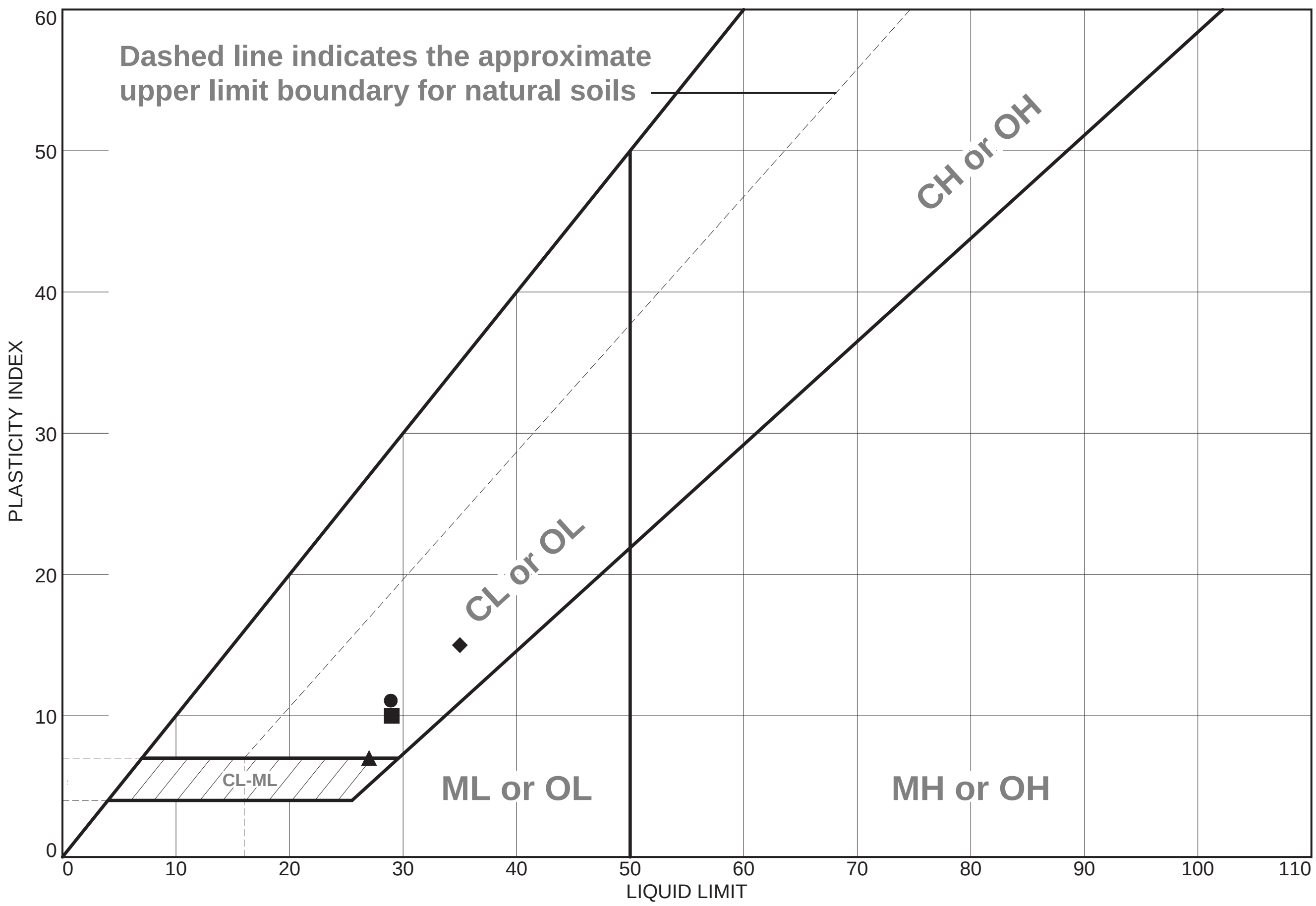
CDM Smith

Cambridge, Massachusetts

Client: TVA
Project: Watts Bar Fossil Plant CCP Closure
Project No: 95618-83529 **Figure**

Tested By: NE **Checked By:** MR

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean clay	29	18	11			CL
■	Lean clay	29	19	10			CL
▲	Lean clay-low plasticity silt	27	20	7			CL-ML
◆	Lean clay	35	20	15			CL

Project No. 95618-83529 **Client:** TVA

Project: Watts Bar Fossil Plant CCP Closure

●**Source of Sample:** B-3 **Depth:** 2-4 **Sample Number:** S-2
■**Source of Sample:** B-3 **Depth:** 13-15 **Sample Number:** S-6
▲**Source of Sample:** B-3 **Depth:** 18-20 **Sample Number:** S-7
◆**Source of Sample:** B-3 **Depth:** 23-25 **Sample Number:** S-8

Remarks:
●As received moisture content=14.5%
■As received moisture content=21.3%
▲As received moisture content=28.1%
◆As received moisture content=31.9%

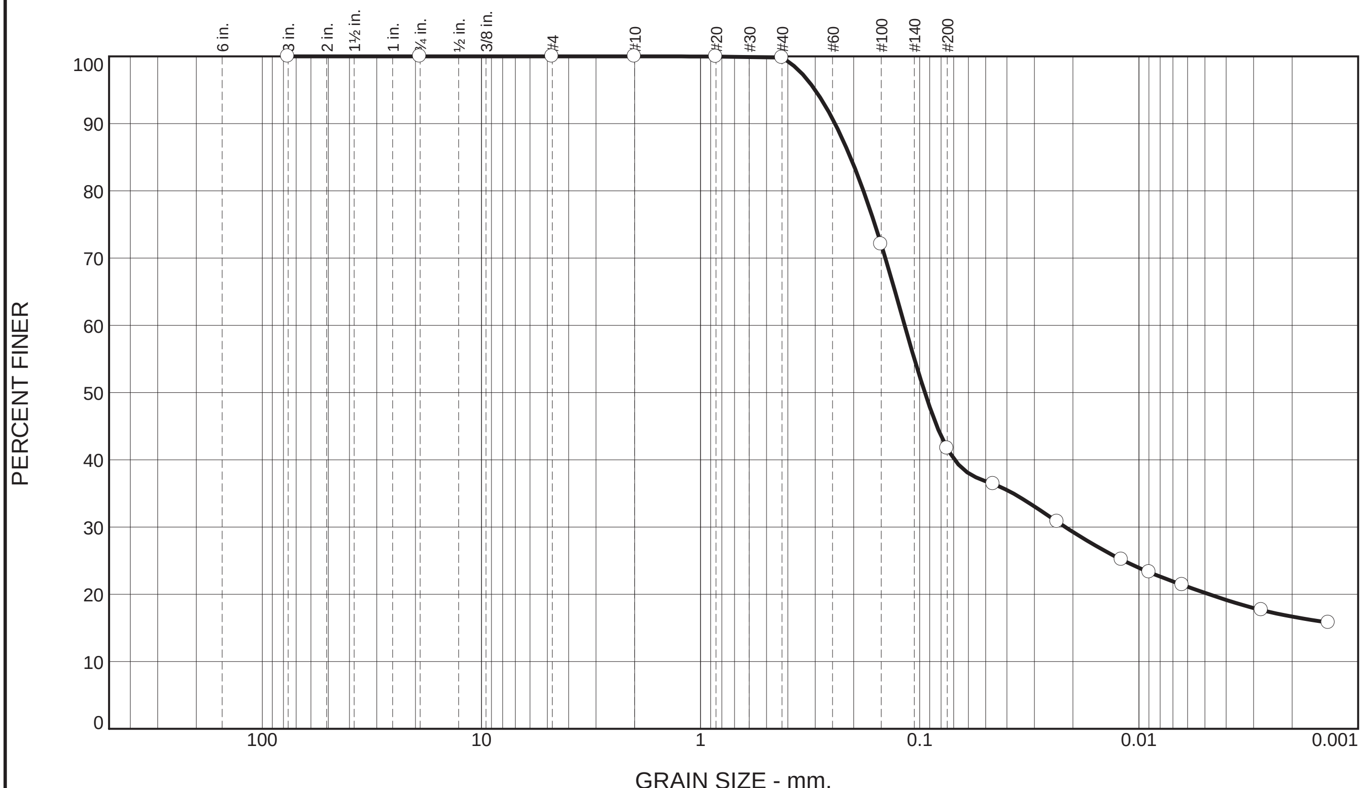
CDM Smith

Cambridge, Massachusetts

Figure

Tested By: NE **Checked By:** MR

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.2	58.1	21.5	20.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3	100.0		
3/4	100.0		
#4	100.0		
#10	100.0		
#20	100.0		
#40	99.8		
#100	72.1		
#200	41.7		

* (no specification provided)

Material Description

Clayey sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2437 D_{85}= 0.2072 D_{60}= 0.1172
D_{50}= 0.0948 D_{30}= 0.0217 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

As received moisture content=14.5%
Soil classification and description based on Visual Manual Procedure ASTM D 2488

Source of Sample: B-3 **Depth:** 8-10
Sample Number: S-5

Date: 11/15/2011

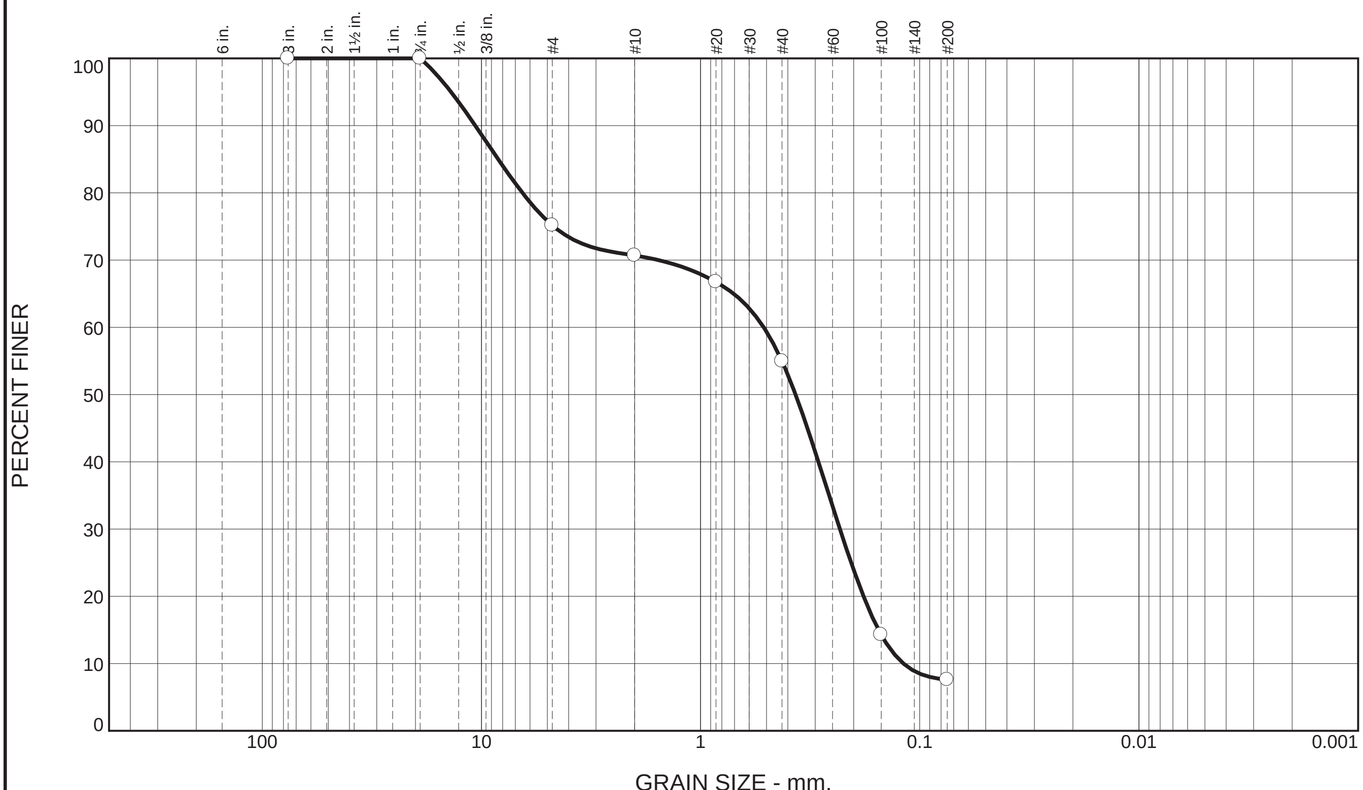
CDM Smith

Cambridge, Massachusetts

Client: TVA
Project: Watts Bar Fossil Plant CCP Closure
Project No: 95618-83529 **Figure**

Tested By: NE **Checked By:** MR

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	24.8	4.5	15.8	47.3	7.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3	100.0		
3/4	100.0		
#4	75.2		
#10	70.7		
#20	66.7		
#40	54.9		
#100	14.3		
#200	7.6		

* (no specification provided)

Material Description

Poorly graded sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 10.6723 D_{85}= 8.3878 D_{60}= 0.5154
D_{50}= 0.3688 D_{30}= 0.2314 D_{15}= 0.1542
D_{10}= 0.1187 C_u= 4.34 C_c= 0.88

Classification

USCS= SP-SM AASHTO=

Remarks

As received moisture content=22.0%
Soil classification and description based on Visual Manual Procedure ASTM D 2488

Source of Sample: B-3 **Depth:** 33-35
Sample Number: S-10

Date: 11/15/2011

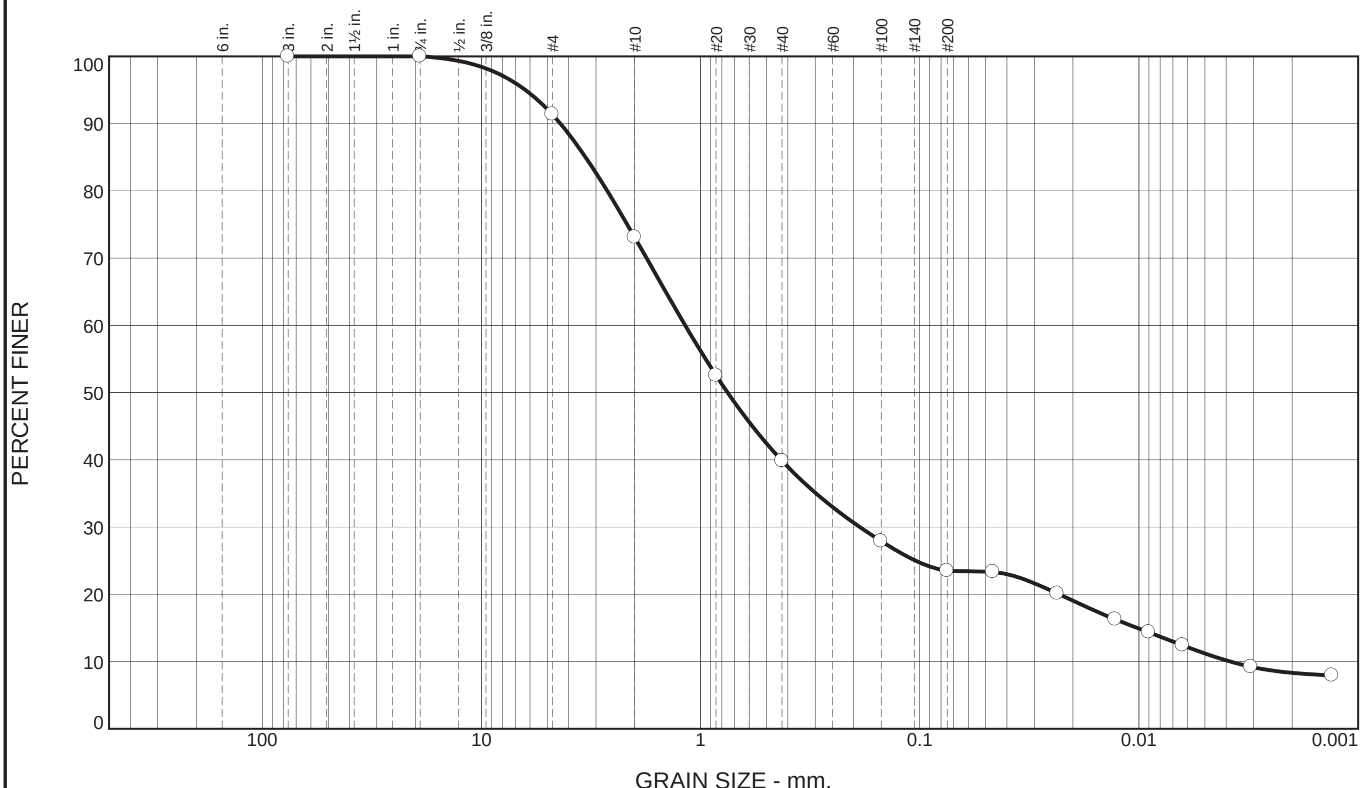
CDM Smith

Cambridge, Massachusetts

Client: TVA
Project: Watts Bar Fossil Plant CCP Closure
Project No: 95618-83529 **Figure**

Tested By: NE **Checked By:** MR

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	8.6	18.3	33.3	16.3	12.3	11.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3	100.0		
3/4	100.0		
#4	91.4		
#10	73.1		
#20	52.5		
#40	39.8		
#100	27.9		
#200	23.5		

* (no specification provided)

Material Description

Silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 4.3626 D_{85}= 3.3491 D_{60}= 1.1759
D_{50}= 0.7530 D_{30}= 0.1876 D_{15}= 0.0102
D_{10}= 0.0038 C_u= 306.42 C_c= 7.80

Classification

USCS= SM AASHTO=

Remarks

As received moisture content=11.6%
Soil classification and description based on Visual Manual Procedure ASTM D2488

Source of Sample: B-3 **Depth:** 38-38.7
Sample Number: S-11

Date: 11/15/2011

CDM Smith

Cambridge, Massachusetts

Client: TVA
Project: Watts Bar Fossil Plant CCP Closure
Project No: 95618-83529 **Figure**

Tested By: NE **Checked By:** MR

Geotechnical Engineering Laboratory

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TEST SUMMARY - ASTM D4767

Client:	TVA	**Test Date:**	3/14/2012	**LL :**	34
Project:	Watts Bar	**Exploration No:**	B-2	**PL :**	19
Location:	Spring City, TN	**Sample No:**	U-1 Specimen 1	**PI :**	15
Project No:	95618-83529	**Depth (ft):**	21	**USCS:**	CL

Initial

Moisture Content (%):	20.7%
Dry Unit Weight (pcf):	105.9
Diameter (in):	1.407
Height (in):	3.125
Void Ratio (-):	0.59
Saturation (%):	94.7%
Moisture Content (Trim.%):	19.9%
Cross Sectional Area (in^2):	1.555

Final

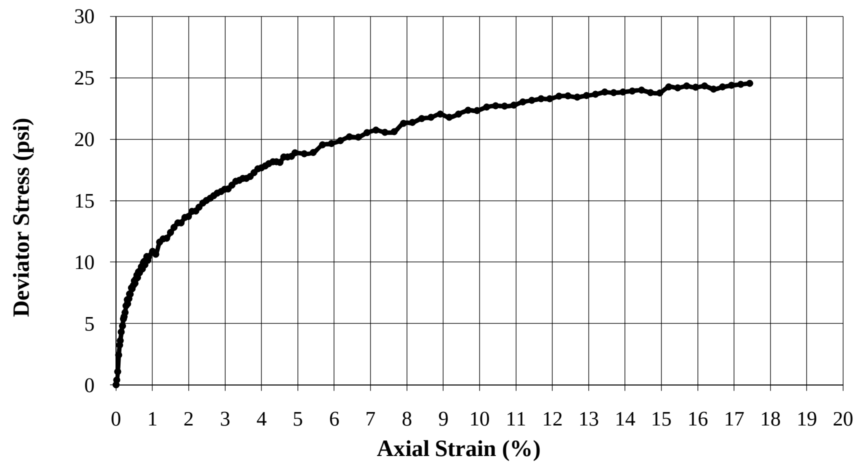
Moisture Content (%):	23.2%
Dry Unit Weight (pcf):	103.3
Height (in):	2.564
Void Ratio (-):	0.63
Saturation (%):	99.4%
Cross Sectional Area (in^2):	1.926

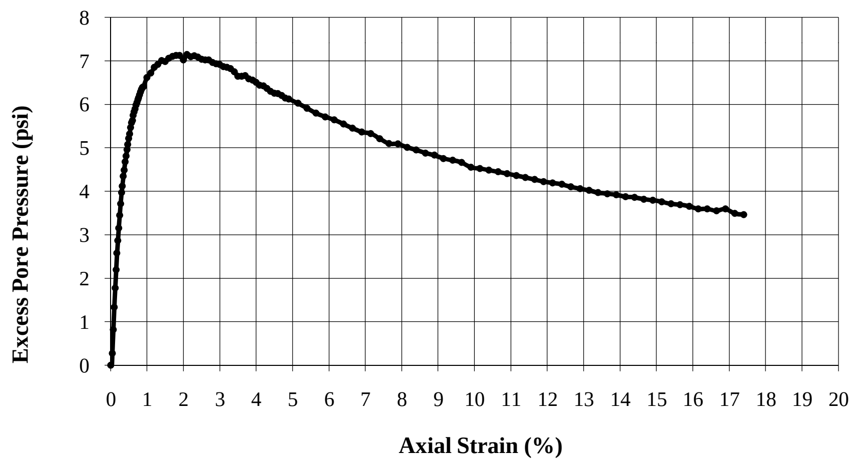
End of Consolidation Data

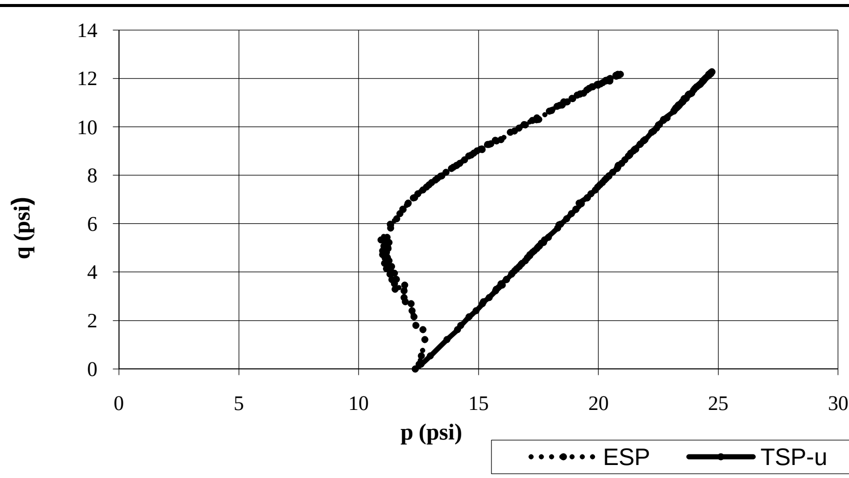
A_c Evaluated using Method	B
Sample Saturated using Method	B
Moisture Content (%):	23.2%
Dry Unit Weight (pcf):	103.3
Height (in):	3.125
Void Ratio (-):	0.63
Saturation (%):	99.4%
Cross Sectional Area (in^2):	1.590
Pore Pressure Parameter B (-):	0.97
Final Back Pressure (psi):	80
Consolidation Pressure (psi):	12.21

Shear Data

Shear Strain Rate (%/hr):	1%
Max. Deviator Stress (psi):	24.56
Strain at Failure (%):	15.00
Minor Eff. Pr. Stress (psi):	8.88
Major Eff. Pr. Stress (psi):	33.44
Undrained Strength Ratio (-):	1.01







Notes:

1. Value of Specific Gravity Gs is assumed
2. Failure criterion: max. deviator stress at strain ≤ 15%

Remarks:

Geotechnical Engineering Laboratory

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TEST SUMMARY - ASTM D4767

Client:	TVA	**Test Date:**	3/14/2012	**LL :**	34
Project:	Watts Bar	**Exploration No:**	B-2	**PL :**	19
Location:	Spring City, TN	**Sample No:**	U-1 Specimen 2	**PI :**	15
Project No:	95618-83529	**Depth (ft):**	21	**USCS:**	CL

Initial

Moisture Content (%):	19.3%
Dry Unit Weight (pcf):	104.4
Diameter (in):	1.385
Height (in):	3.220
Void Ratio (-):	0.61
Saturation (%):	84.8%
Moisture Content (Trim.%):	20.6%
Cross Sectional Area (in^2):	1.507

Final

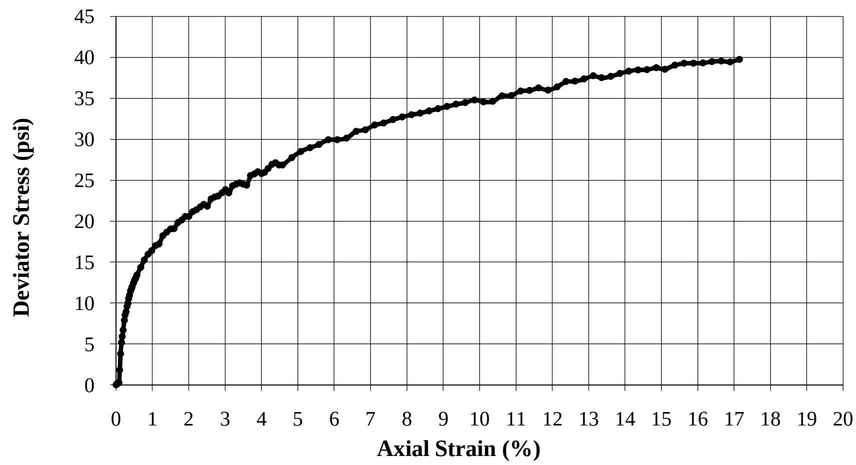
Moisture Content (%):	22.8%
Dry Unit Weight (pcf):	104.0
Height (in):	2.651
Void Ratio (-):	0.62
Saturation (%):	99.4%
Cross Sectional Area (in^2):	1.820

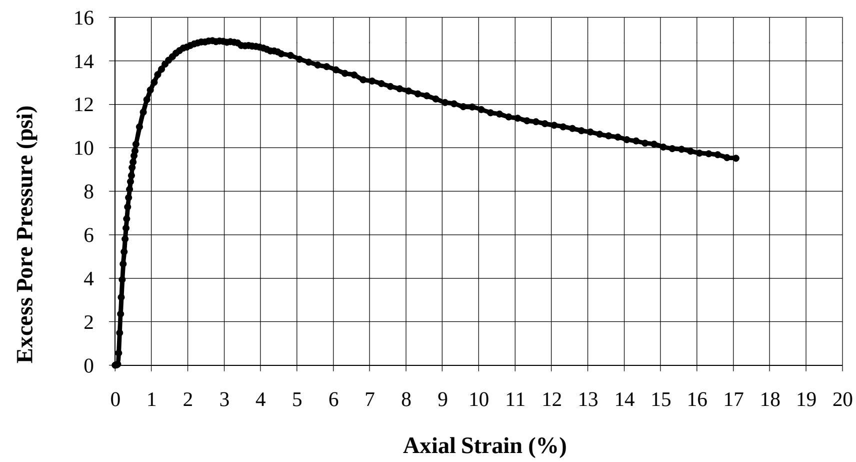
End of Consolidation Data

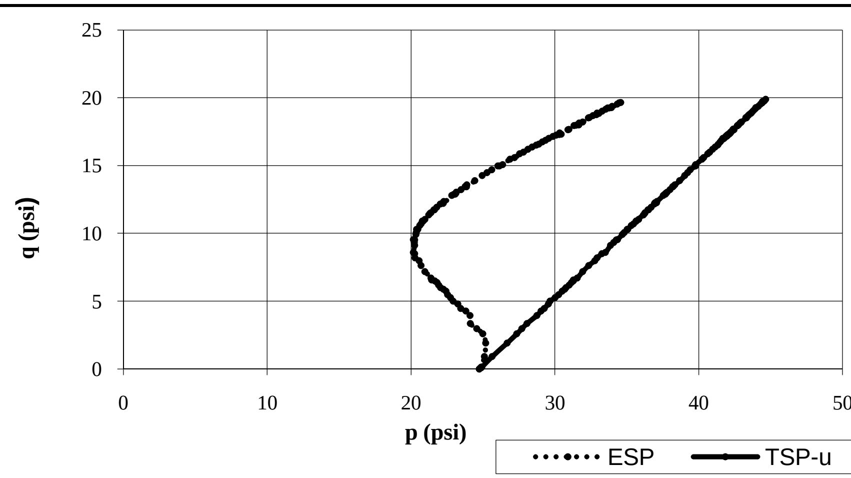
A_c Evaluated using Method	B
Sample Saturated using Method	B
Moisture Content (%):	22.8%
Dry Unit Weight (pcf):	104.0
Height (in):	3.219
Void Ratio (-):	0.62
Saturation (%):	99.4%
Cross Sectional Area (in^2):	1.508
Pore Pressure Parameter B (-):	0.97
Final Back Pressure (psi):	85
Consolidation Pressure (psi):	24.34

Shear Data

Shear Strain Rate (%/hr):	1%
Max. Deviator Stress (psi):	39.77
Strain at Failure (%):	15.00
Minor Eff. Pr. Stress (psi):	15.25
Major Eff. Pr. Stress (psi):	55.02
Undrained Strength Ratio (-):	0.82







Notes:

1. Value of Specific Gravity Gs is assumed
2. Failure criterion: max. deviator stress at strain ≤ 15%

Remarks:

CDM Smith

Geotechnical Engineering Laboratory

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TEST SUMMARY - ASTM D4767

Client:	TVA	**Test Date:**	3/14/2012	**LL :**	34
Project:	Watts Bar	**Exploration No:**	B2	**PL :**	19
Location:	Spring City, TN	**Sample No:**	U-1 Specimen 3	**PI :**	15
Project No:	95618-83529	**Depth (ft):**	21	**USCS:**	CL

Initial

Moisture Content (%):	20.8%
Dry Unit Weight (pcf):	104.5
Diameter (in):	1.411
Height (in):	3.085
Void Ratio (-):	0.61
Saturation (%):	91.7%
Moisture Content (Trim.%):	20.2%
Cross Sectional Area (in^2):	1.564

Final

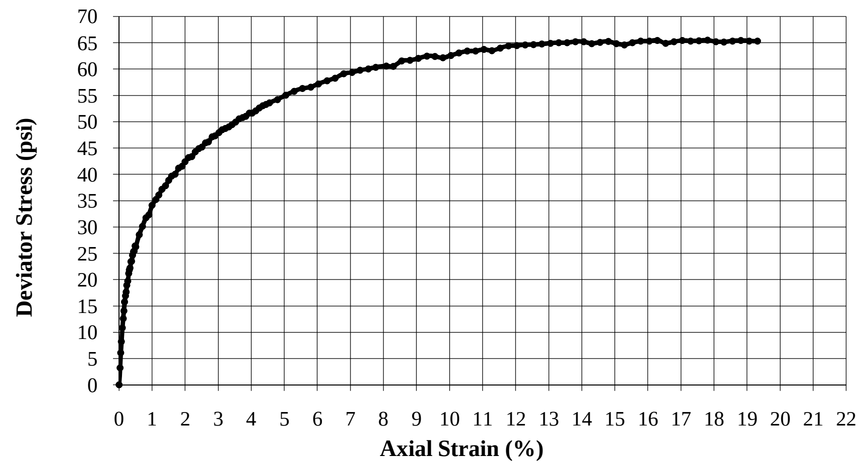
Moisture Content (%):	21.1%
Dry Unit Weight (pcf):	107.1
Height (in):	2.480
Void Ratio (-):	0.57
Saturation (%):	99.4%
Cross Sectional Area (in^2):	1.853

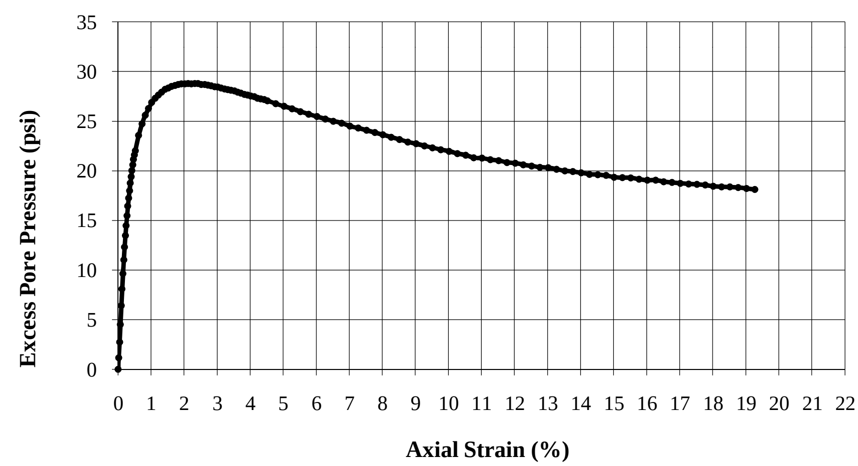
End of Consolidation Data

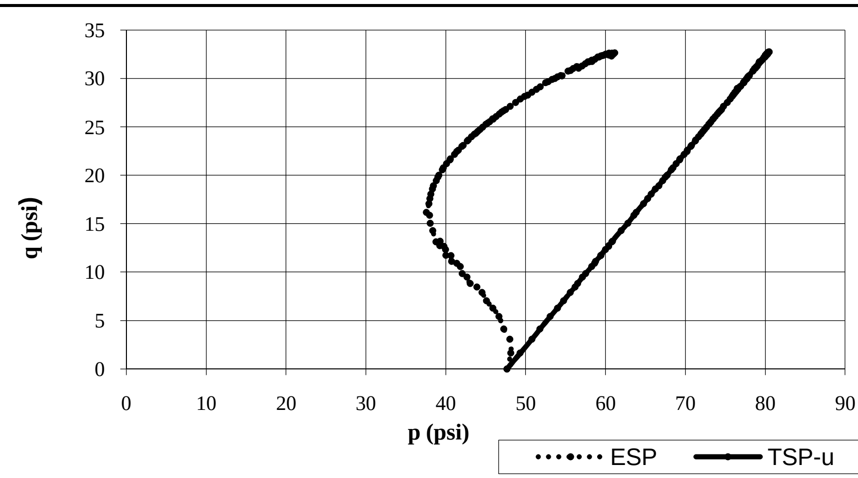
A_c Evaluated using Method	B
Sample Saturated using Method	B
Moisture Content (%):	21.1%
Dry Unit Weight (pcf):	107.1
Height (in):	3.084
Void Ratio (-):	0.57
Saturation (%):	99.4%
Cross Sectional Area (in^2):	1.523
Pore Pressure Parameter B (-):	0.97
Final Back Pressure (psi):	107
Consolidation Pressure (psi):	48.22

Shear Data

Shear Strain Rate (%/hr):	1%
Max. Deviator Stress (psi):	65.49
Strain at Failure (%):	15.00
Minor Eff. Pr. Stress (psi):	29.14
Major Eff. Pr. Stress (psi):	94.62
Undrained Strength Ratio (-):	0.68







Notes:

1. Value of Specific Gravity Gs is assumed
2. Failure criterion: max. deviator stress at strain ≤ 15%

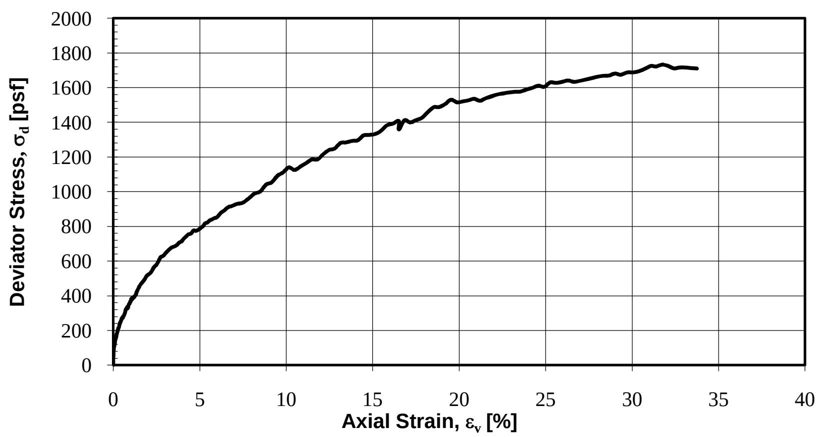
Remarks:

Geotechnical Engineering Laboratory

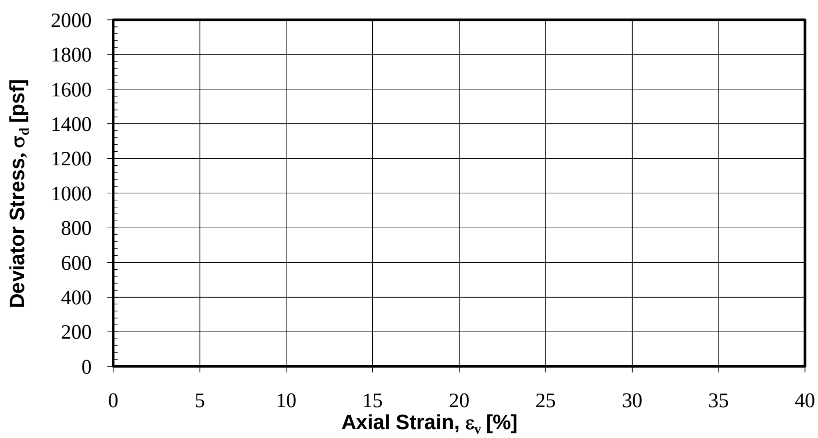
UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST SUMMARY - ASTM D2850

Client:	TVA	**Test Date:**	3/14/2012	**LL :**	34
Project:	Watts Bar	**Exploration No:**	B-2	**PL :**	19
Location:		**Sample No:**	U-1	**PI :**	15
Project No:	95618-83529	**Depth (ft):**	21.5	**USCS:**	CL

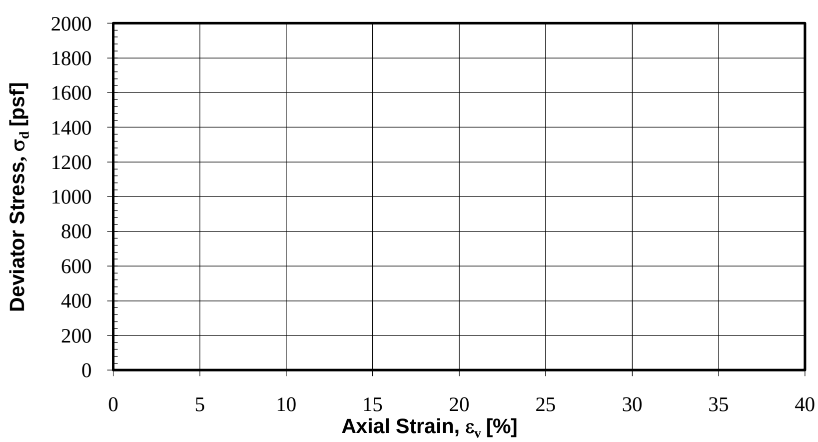
Specimen 1	**Initial**	**Final**
Moisture Content (%):	21.1%	22.0%
Dry Unit Weight (pcf):	126.5	-
Diameter (in):	1.390	-
Height (in):	2.750	-
Void Ratio (-):	0.61	0.61
Saturation (%):	93.3%	97.4%
Specific Gravity (-)(1):	2.70	
Moisture Content (Trim.%):	20.2%	
Strain Rate (%/min):	0.7	
Confining Pressure (psi):	7	
Strain at Failure (%):	15.00	
Compressive Strength (psf)(2)	12.0	



Specimen	**Initial**	**Final**
Moisture Content (%):		
Dry Unit Weight (pcf):		
Diameter (in):		
Height (in):		
Void Ratio (-):		
Saturation (%):		
Specific Gravity (-)(1):		
Moisture Content (Trim.%):		
Strain Rate (%/min):		
Confining Pressure (psi):		
Strain at Failure (%):		
Compressive Strength (psi)(2)		



Specimen	**Initial**	**Final**
Moisture Content (%):		
Dry Unit Weight (pcf):		
Diameter (in):		
Height (in):		
Void Ratio (-):		
Saturation (%):		
Specific Gravity (-)(1):		
Moisture Content (Trim.%):		
Strain Rate (%/min):		
Confining Pressure (psi):		
Strain at Failure (%):		
Compressive Strength (psi)(2)		



Notes:

1. Value of specific gravity is assumed
2. Failure criterion: maximum deviator stress at strain less than or equal to 15%

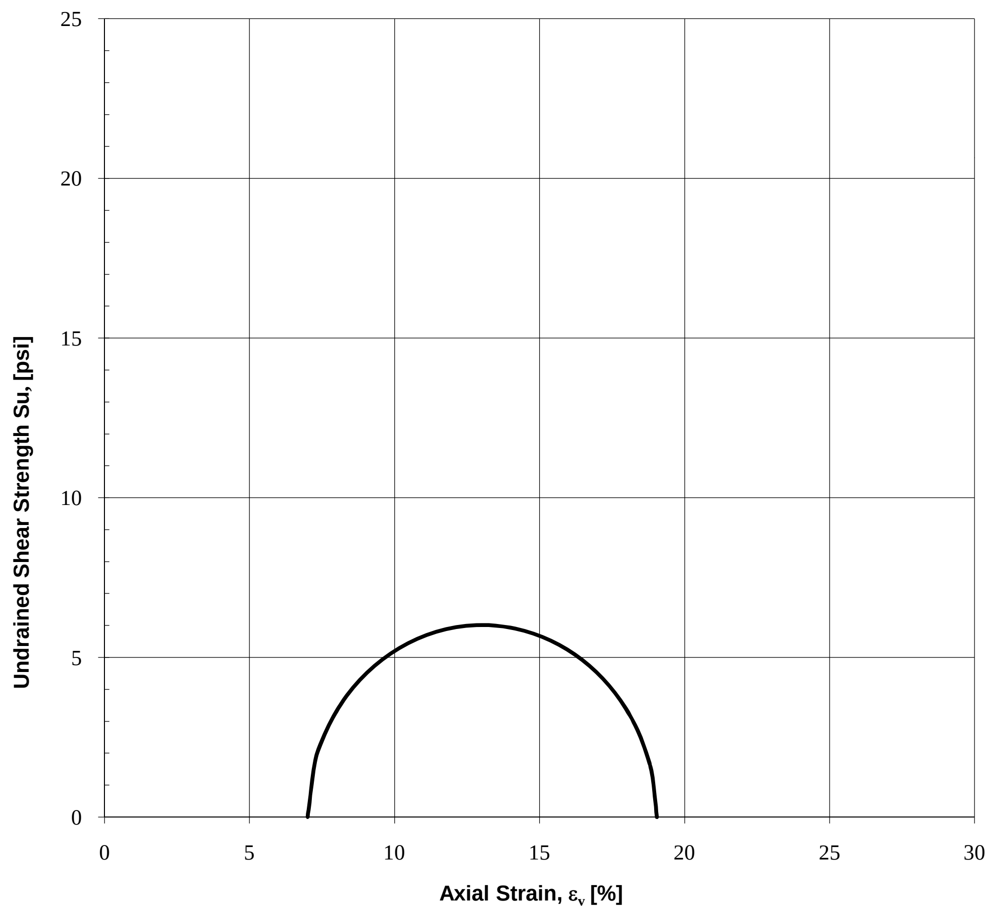
Test Remarks:

Geotechnical Engineering Laboratory

UNCONSOLIDATED-UNDRAINED TEST - MOHR CIRCLES

Client:	TVA	**Test Date:**	3/14/2012	**LL :**	34
Project:	Watts Bar	**Exploration No:**	B-2	**PL :**	19
Location:	0	**Sample No:**	U-1	**PI :**	15
Project No:	95618-83529	**Depth (ft):**	21.5	**USCS:**	CL

	Specimen 1	Specimen 2	Specimen 3
Confining Pressure (psi)	7	0	0
Undrained Shear Strength Su (psi)	6.02	0.00	0.00
Strain at Failure (%)	15.00	0.00	0.00
Initial Moisture Content (%)	21.1%	0.0%	0.0%
Initial Saturation (%)	93.3%	0.0%	0.0%
Average Su (psi)			



Notes:

Test Remarks:

CDM Smith

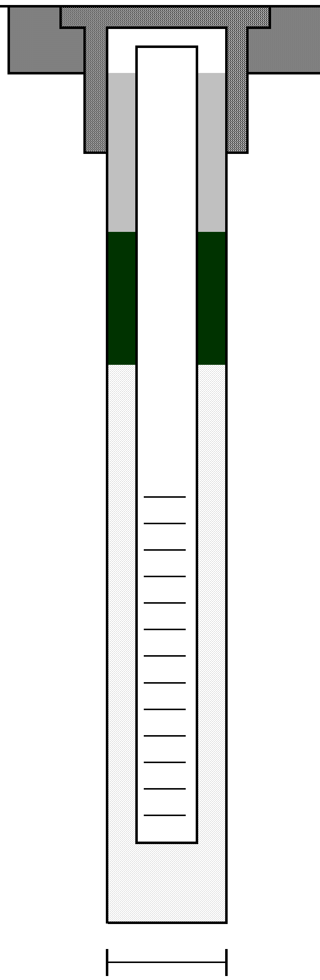
5400 Glenwood Ave
Suite 300
Raleigh, NC 27612
(919) 787-5620

Monitoring Well Installation Log

Client:	TVA	Contractor:	Total Depth Drilling	Boring/Well No.:	B-1/MW-1
Project Name:	Watts Bar Fossil Plant	Driller:	Tim Hall	Date Installed:	11/17/11 - 01/11/12
Project Location:	Watts Bar (Rhea Co.), TN	Ground EL:	699.0 ft	Logged By:	MRH
Project Number:	83529	Riser EL:		Page:	1 of 1

GROUND SURFACE

ROADWAY BOX



SURFACE SEAL: (Thickness & Type)	1 ft - Portland Cement
BACKFILL MATERIAL: (Type)	Soil sloughed into hole
TOP OF SEAL:	16 ft
SEAL CONSTRUCTION: (Thickness & Type)	7 ft - Bentonite
TOP OF SANDPACK:	23 ft
RISER CONSTRUCTION: (Type, Diameter Material)	Schedule 40 PVC, 2 - Inch
TOP OF SCREEN:	25 ft
SANDPACK TYPE:	Filter Sand - DSI Well Gravel Pack
SCREEN MATERIAL: (Type, Slot, Diameter Material)	Schedule 40 PVC, 0.10, 2-Inch
BOTTOM OF SCREEN:	35 ft
BOTTOM OF BOREHOLE:	44.6 ft
BOREHOLE DIAMETER:	0.75 ft - soil/0.24 ft - rock

NOTE: All depths are in feet below ground surface, unless noted otherwise.

Remarks:

Updated On: 04/09/01

CDM Smith

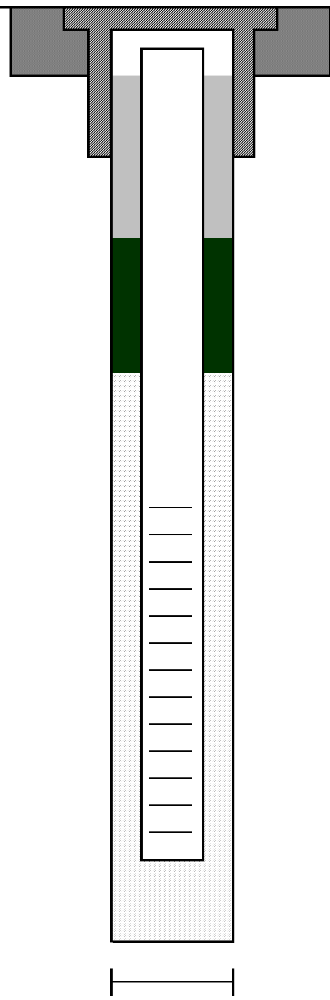
5400 Glenwood Ave
Suite 300
Raleigh, NC 27612
(919) 787-5620

Monitoring Well Installation Log

Client:	TVA	Contractor:	Total Depth Drilling	Boring/Well No.:	B-3/MW-3
Project Name:	Watts Bar Fossil Plant	Driller:	Tim Hall	Date Installed:	11/16/2011
Project Location:	Watts Bar (Rhea Co.), TN	Ground EL:	701.0 ft	Logged By:	MRH
Project Number:	83529	Riser EL:		Page:	1 of 1

GROUND SURFACE

ROADWAY BOX



SURFACE SEAL: (Thickness & Type) 3 ft - Portland Cement

BACKFILL MATERIAL: (Type) Filter Sand (DSI gravel pack)

TOP OF SEAL: 24 ft

SEAL CONSTRUCTION: (Thickness & Type) 4 ft - Bentonite

TOP OF SANDPACK: 28 ft

RISER CONSTRUCTION: (Type, Diameter Material) Schedule 40 PVC, 2-Inch

TOP OF SCREEN: 30 ft

SANDPACK TYPE: :Filter Sand - DSI Well Gravel Pack

SCREEN MATERIAL: (Type, Slot, Diameter Material) Schedule 40 PVC, 0.10, 2-Inch

BOTTOM OF SCREEN: 40 ft

BOTTOM OF BOREHOLE: 54.8 ft

BOREHOLE DIAMETER: 0.75 ft - soil/0.24 ft - rock

NOTE: All depths are in feet below ground surface, unless noted otherwise.

Remarks:

Updated On: 04/09/01

Summary of Groundwater Level Readings

TVA WBF CCP Closure

Spring City, TN

Location	Ground Surface Elevation	Groundwater Level Readings		Date	Time (24 hr)
		in feet below ground surface	Elevation, ft		
B-1	699	12.1	686.9	11/16/2011	17:15
		13.1	685.9	11/16/2011	17:40
		9.32	689.7	1/11/2012	10:40
B-2	711	37.1	673.9	1/10/2012	13:05
		27.4	683.6	1/10/2012	14:50
B-3	701	31.15	669.9	11/15/2011	10:20
		15.70	685.3	11/16/2011	11:00
		19.00	682.0	1/10/2012	15:10
		18.11	682.9	1/11/2012	11:10

Note: Elevations & locations based on estimated distance to existing features.

ATTACHMENT B

Client: TVA
Project: WBF CCP CLOSURE

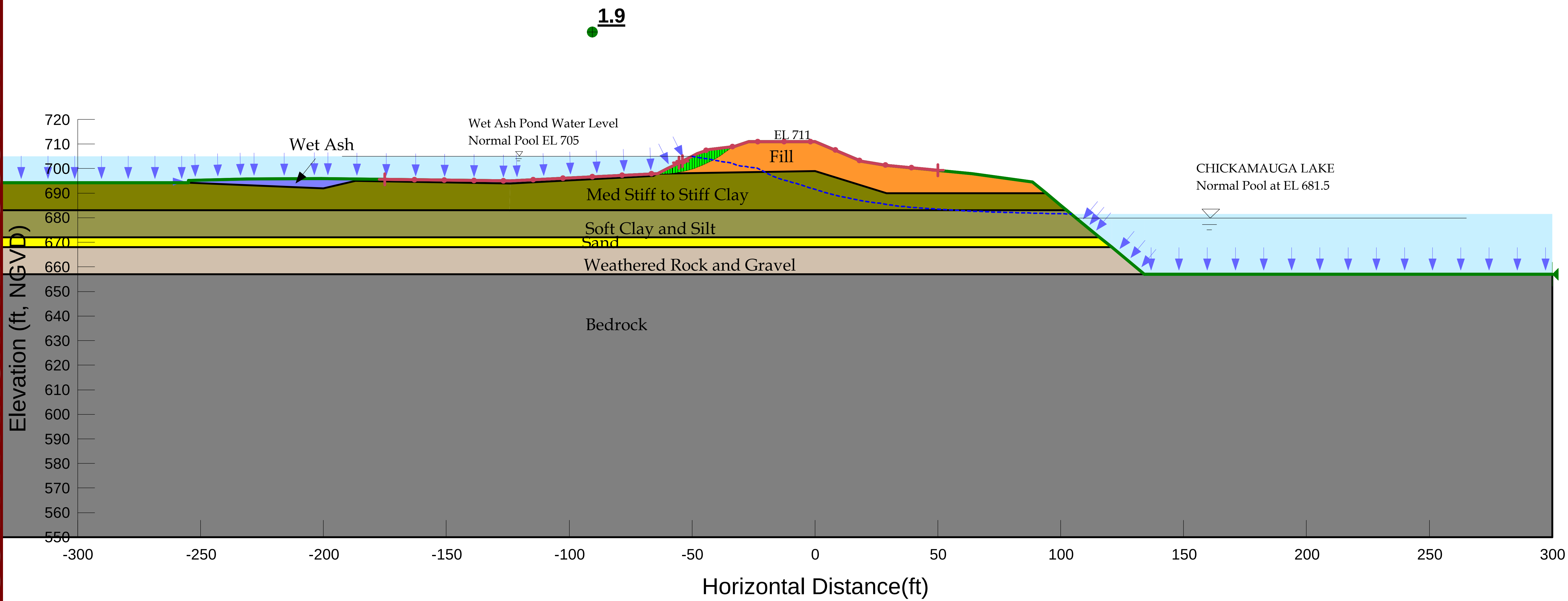
Computed By: Wen, Jintao
Date & Time: 1/20/2012 10:03:52 AM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section A-A' at Wet Ash Pond Area

Case Number: A-1
Location: Section A-A'

Model Scenario:
Existing Conditon at Wet Ash Pond Area
Static Analysis

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 0 psf 29 °
Layer 2B: Soft Clay and Silt 110 pcf 0 psf 28 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

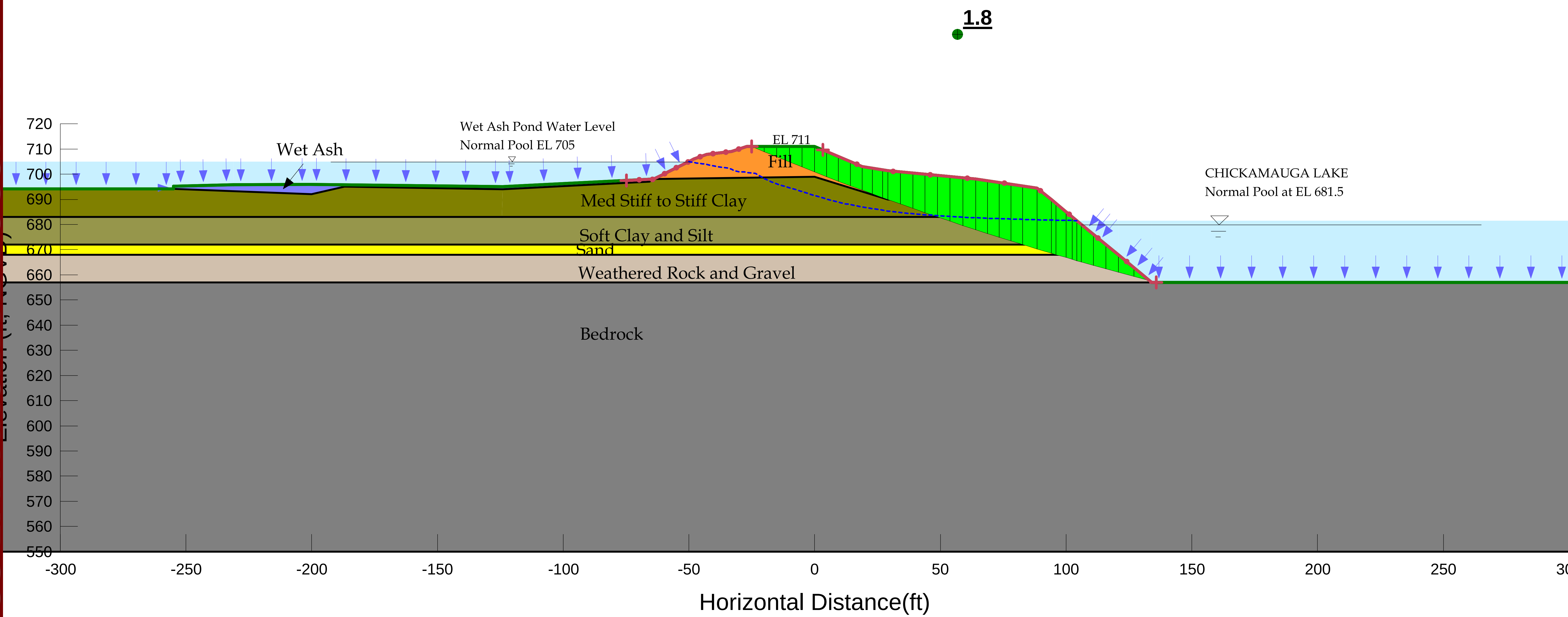
Computed By: Wen, Jintao
Date & Time: 1/20/2012 10:03:52 AM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section A-A' at Wet Ash Pond Area

Case Number: A-1
Location: Section A-A'

Model Scenario:
Existing Conditon at Wet Ash Pond Area
Static Analysis

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 0 psf 29 °
Layer 2B: Soft Clay and Silt 110 pcf 0 psf 28 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

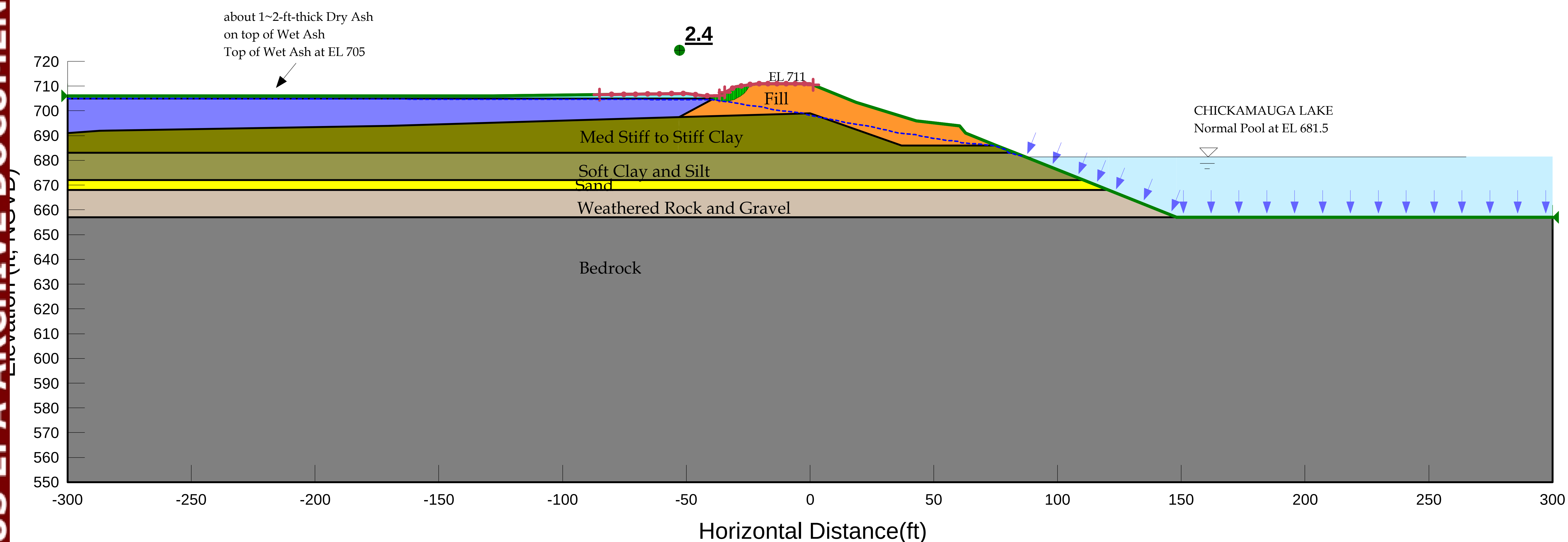
Computed By: Wen, Jintao
Date & Time: 1/20/2012 9:58:09 AM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section B-B' at Dry Ash Area

Case Number: B-1
Location: Section B-B'

Model Scenario:
Existing Conditon at Dry Ash Area.
Static Analsyes

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Dry Ash 85 pcf 0 psf 25 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 0 psf 29 °
Layer 2B: Soft Clay and Silt 110 pcf 0 psf 28 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

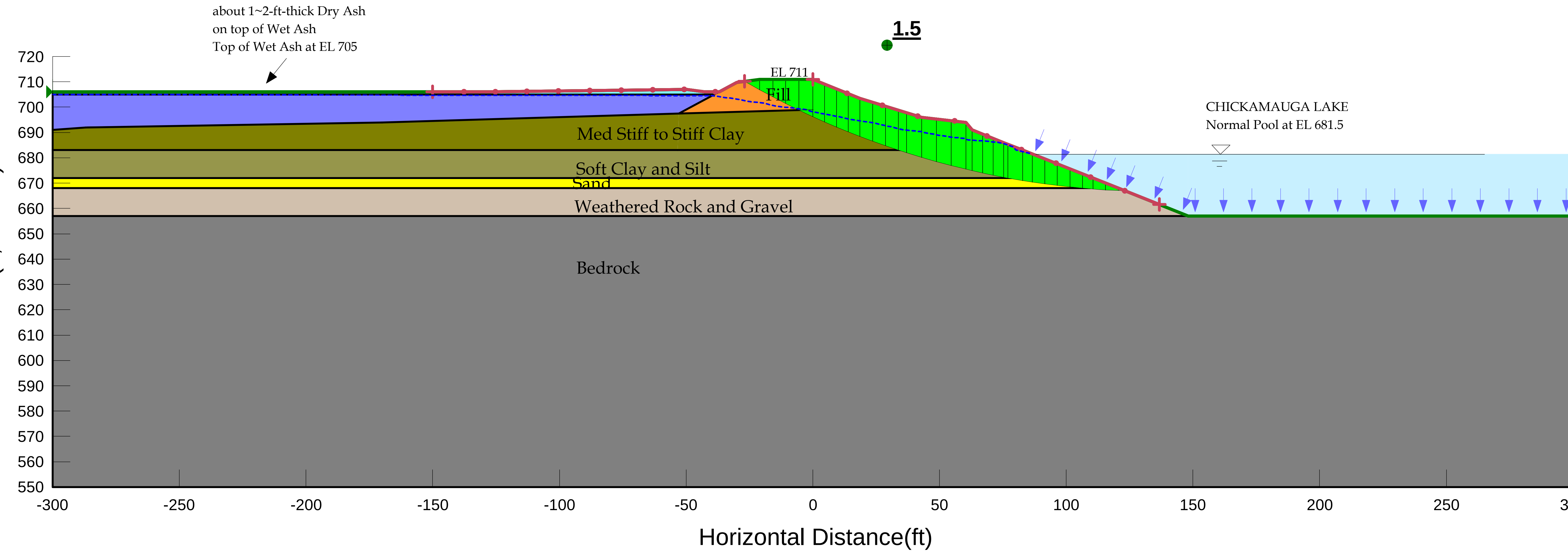
Computed By: Wen, Jintao
Date & Time: 1/20/2012 10:02:17 AM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section B-B' at Dry Ash Area

Case Number: B-1
Location: Section B-B'

Model Scenario:
Existing Conditon at Dry Ash Area.
Static Analsyes

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Dry Ash 85 pcf 0 psf 25 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 0 psf 29 °
Layer 2B: Soft Clay and Silt 110 pcf 0 psf 28 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

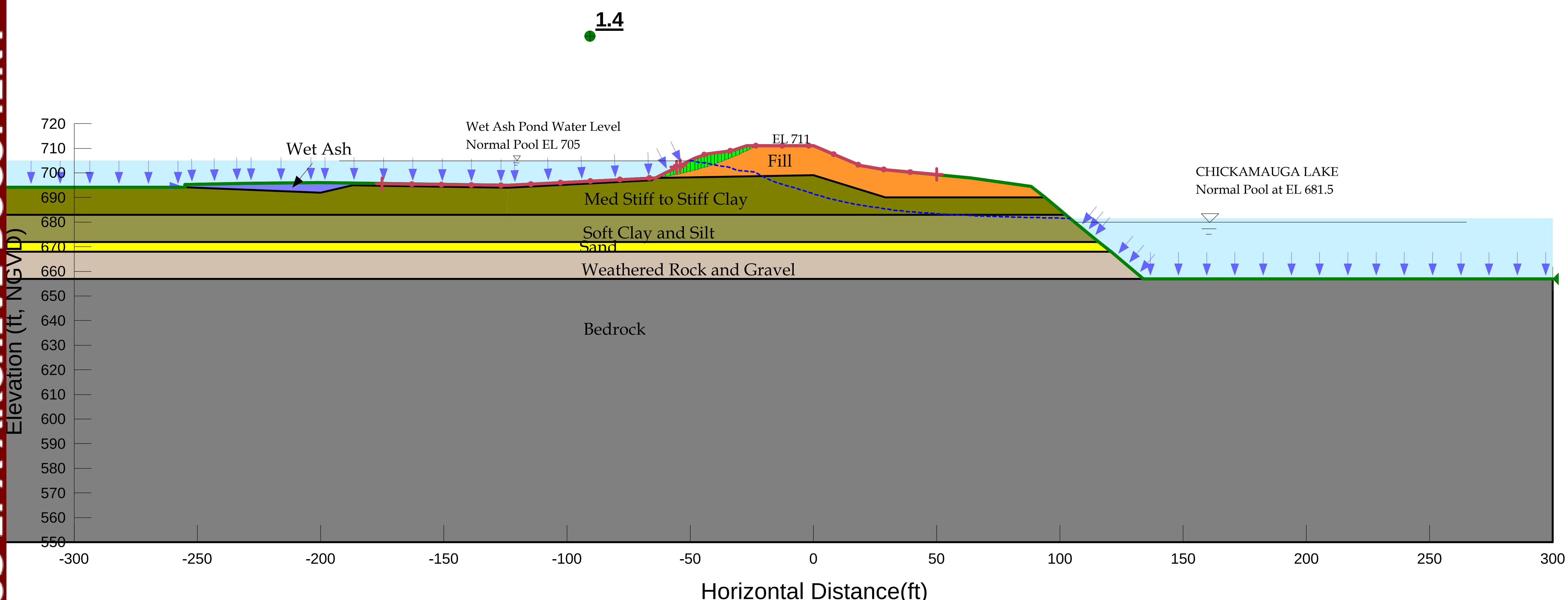
Computed By: Wen, Jintao
Date & Time: 4/25/2012 3:08:18 PM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section A-A' at Wet Ash Pond Area

Case Number: A-2
Location: Section A-A'

Model Scenario:
Existing Conditon at Wet Ash Pond
Seismic Analysis PHA=0.116g

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 1300 psf 0 °
Layer 2B: Soft Clay and Silt 110 pcf 500 psf 0 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

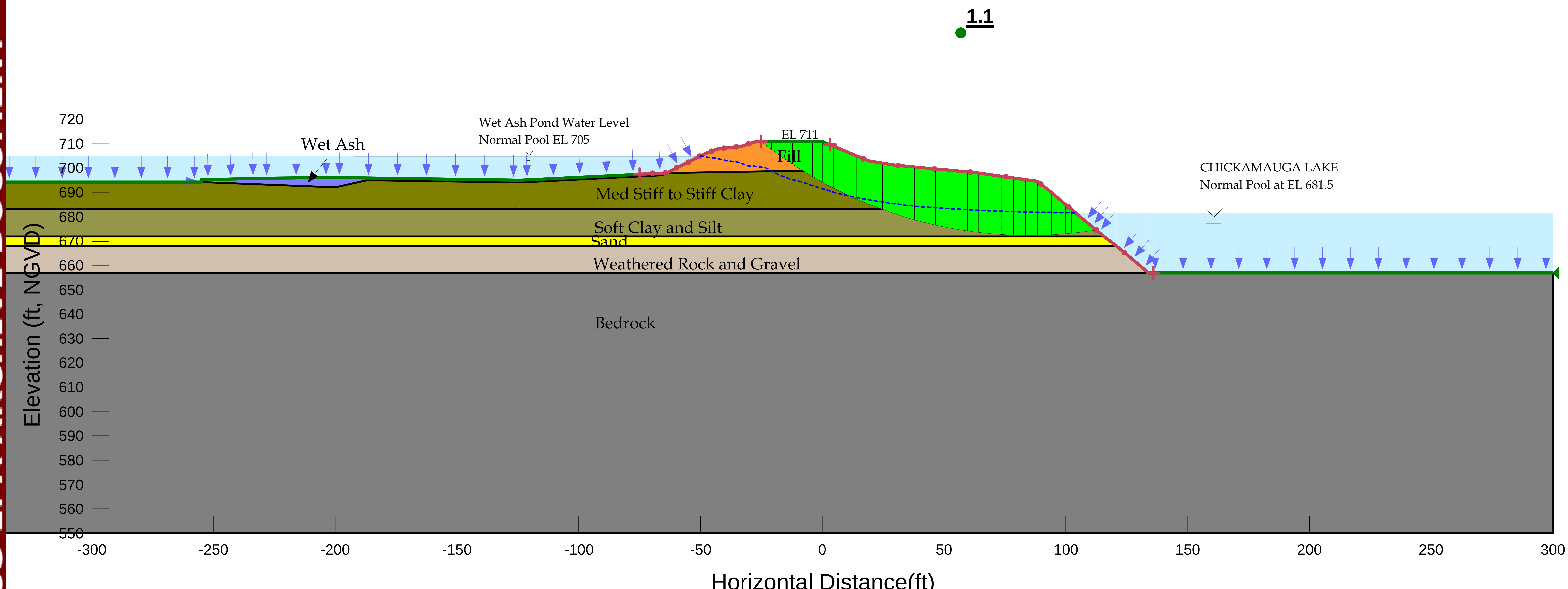
Computed By: Wen, Jintao
Date & Time: 4/25/2012 3:08:18 PM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section A-A' at Wet Ash Pond Area

Case Number: A-2
Location: Section A-A'

Model Scenario:
Existing Conditon at Wet Ash Pond
Seismic Analysis PHA=0.116g

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 1300 psf 0 °
Layer 2B: Soft Clay and Silt 110 pcf 500 psf 0 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

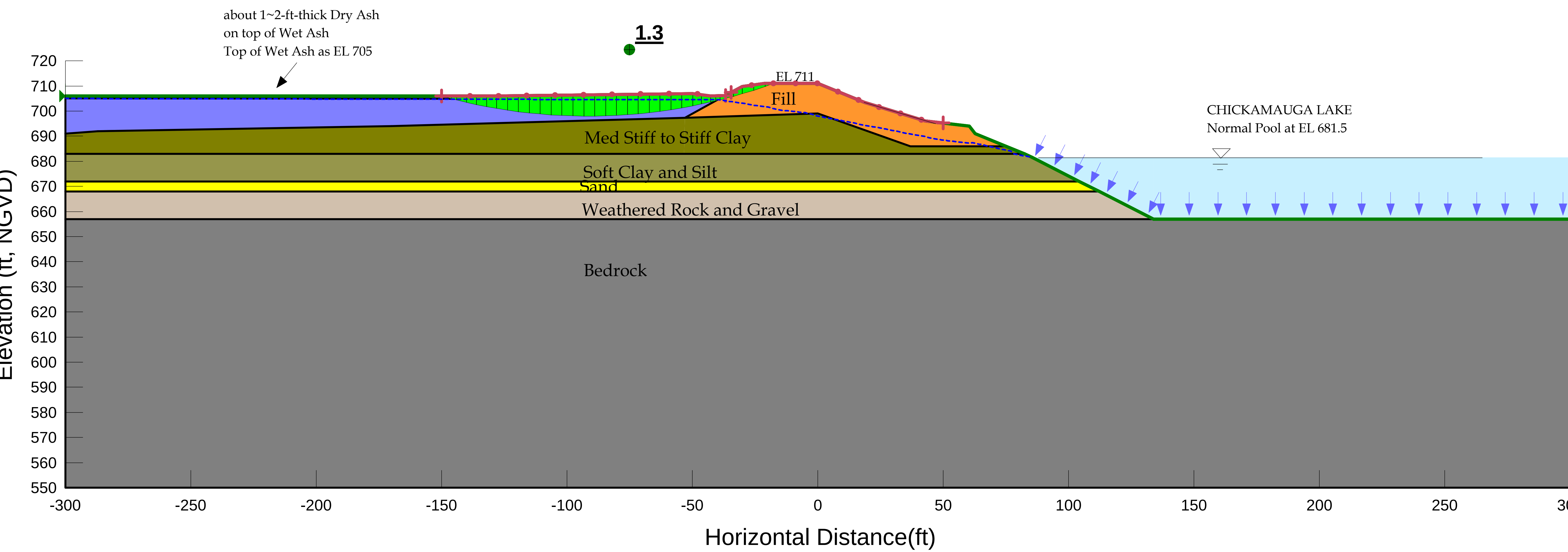
Computed By: Wen, Jintao
Date & Time: 4/25/2012 3:05:46 PM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section B-B' at Dry Ash Area

Case Number: B-2
Location: Section B-B'

Model Scenario:
Existing Conditon at Dry Ash Area.
Seismic Analsyes, PHA=0.116g

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Dry Ash 85 pcf 0 psf 25 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 1300 psf 0 °
Layer 2B: Soft Clay and Silt 110 pcf 500 psf 0 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

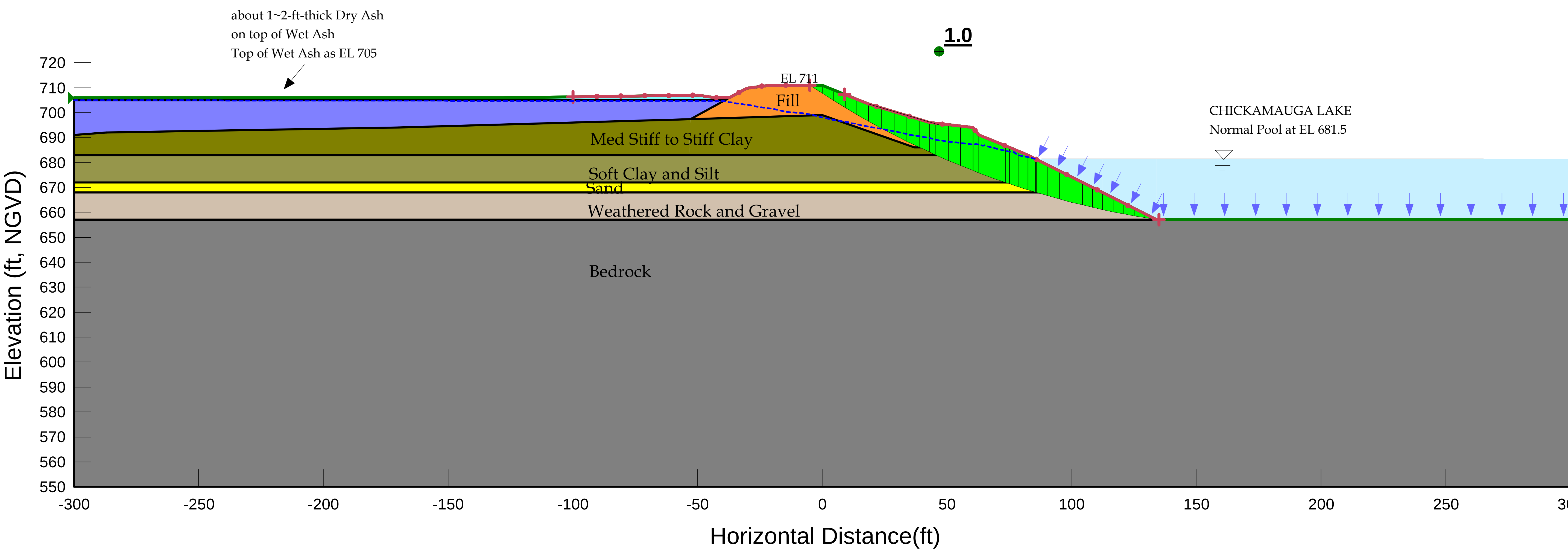
Computed By: Wen, Jintao
Date & Time: 4/25/2012 3:05:46 PM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section B-B' at Dry Ash Area

Case Number: B-2
Location: Section B-B'

Model Scenario:
Existing Conditon at Dry Ash Area.
Seismic Analsyes, PHA=0.116g

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Dry Ash 85 pcf 0 psf 25 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 1300 psf 0 °
Layer 2B: Soft Clay and Silt 110 pcf 500 psf 0 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



Client: TVA
Project: WBF CCP CLOSURE

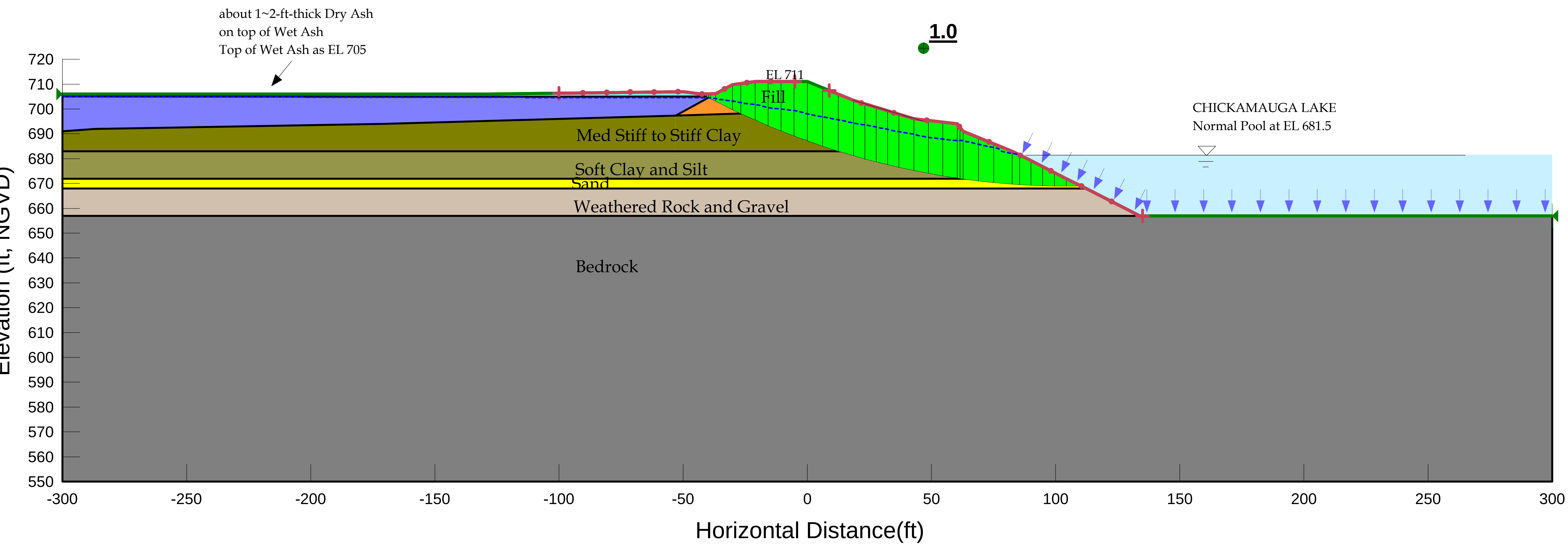
Computed By: Wen, Jintao
Date & Time: 4/20/2012 10:49:51 AM

TVA Watts Bar Fossil Plant, Spring City, TN
Seepage and Slope Stability Analyses
Cross-Section B-B' at Dry Ash Area

Case Number: B-2
Location: Section B-B'

Model Scenario:
Existing Conditon at Dry Ash Area.
Seismic Analsyes, PHA=0.09g

Layer 1: Fill 120 pcf 115 pcf 0 psf 32 °
Dry Ash 85 pcf 0 psf 25 °
Layer 2A: Med Stiff to Stiff Clay 110 pcf 105 pcf 1000 psf 0 °
Layer 2B: Soft Clay and Silt 110 pcf 500 psf 0 °
Layer 3: Sand 120 pcf 0 psf 30 °
Layer 4: Weathered Rock and Gravel 125 pcf 0 psf 40 °
Wet Ash 70 pcf 0 psf 20 °
Layer 5: Bedrock



ATTACHMENT C

Seismic Risk Assessment
Closed CCP Storage Facilities
Tennessee Valley Authority Fossil Plants

Prepared by:

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Seismic Risk Assessment
Closed CCP Storage Facilities
Tennessee Valley Authority Fossil Plants

This document outlines proposed engineering analyses to estimate seismic failure risks at wet storage facilities for coal combustion products, following closure, at various TVA fossil power plants. The specific details outlined in this document are subject to future discussion and modification by the project team.

OVERVIEW

Tennessee Valley Authority (TVA) operates storage facilities for coal combustion products (CCPs) at eleven fossil power generating stations. As TVA transitions to dry systems for handling these materials, 18 to 25 wet storage facilities (CCP ponds, impoundments, dredge cells, etc.) will be closed (drained and capped). The CCP storage facilities are currently operated in accordance with state and federal regulations, but previously issued permits have not required evaluations for seismic performance. Moreover, the existing permits do not require seismic qualification for the storage facilities in their closed configurations.

TVA recognizes there is a potential for strong earthquakes to occur within the region, and there is a tangible risk for seismic failure at each closed CCP facility. These risks, including both the likelihood of failure and the consequences, must be understood to effectively manage TVA's portfolio of byproduct storage sites. This white paper summarizes the methodology that will be used to estimate these risks at the CCP storage facilities following closure.

Seismicity in the TVA service area is attributed to the New Madrid fault and smaller, less concentrated crustal faults. These two earthquake scenarios generate significantly different seismic hazards at each locality and will be considered independently within the risk assessment. At each closed byproduct facility, potential seismic failure modes will be evaluated in sequence. Instability due to soil liquefaction, slope instability due to inertial loading, and other potential failure mechanisms will be addressed. Seismic performance will be evaluated for differing earthquake return periods until a limiting (lowest return period) event that would cause failure is obtained. The probability of seismic failure will then correspond to the probability of this limiting earthquake event. The assessment of risk will also include estimates of potential consequences, as well as costs to mitigate the risks, that reflects the unique setting of the individual storage facilities after closure.

Following the same general methodology, seismic risks will be estimated in two phases. The near-term "Portfolio Seismic Assessment" will provide a rough estimate of seismic risks. The likely performance of each facility will be evaluated using simplified analyses, empirical methods, and the judgment of experienced engineers. The results will establish a ranking of the relative risks across the closure portfolio and also provide a preliminary picture of overall seismic risk. For the subsequent "Facility Seismic Assessments", seismic performance will be judged on the basis of site-specific data and detailed engineering analyses, which will be completed during the closure design process for individual facilities.

SEISMIC RISKS

This white paper provides an overview of the engineering methods proposed by Stantec for estimating seismic risks at TVA's closed byproduct storage sites. For each facility, four specific questions must be answered quantitatively:

(1) What is the approximate probability that a strong earthquake will occur?

Several seismic source zones could produce earthquakes large enough to impact these TVA sites. Very large magnitude earthquakes have occurred within the New Madrid seismic zone, which is located along the western boundaries of Tennessee and Kentucky. Because of their observed large magnitude and frequency of occurrence, New Madrid events contribute substantially to the seismic risks at all TVA sites. Ground motions from a New Madrid earthquake would attenuate with distance toward the east, such that local area sources also contribute significantly to site-specific seismic hazards.

Seismicity across the Tennessee Valley was previously characterized by AMEC/Geomatrix (2004), in a probabilistic study that focused on TVA dam sites. The same seismogenic model can be applied in evaluating earthquakes that would impact other TVA sites. Accordingly, probabilistic seismic hazards obtained from the 2004 AMEC/Geomatrix model will be used in the seismic risk assessment of the closed CCP storage facilities.

(2) Will a given earthquake cause failure in the closed facility?

Many of the TVA byproduct storage facilities are underlain by a substantial thickness of loose, saturated, alluvial soils (silts and sands). Some facilities will have layers of ash or other uncemented CCPs that remain saturated following closure. These materials, especially sluiced fly ash, are prone to liquefaction in a strong earthquake, as cyclic motions cause a build up of pore water pressure and a consequent loss of effective stress and shearing resistance. Extensive liquefaction in a foundation or CCP deposit under a storage facility would be expected, in most cases, to result in lateral spreading and massive slope movements (failure). Even without liquefaction, large slope deformations or failures may be triggered by lateral inertial loads during an earthquake. Liquefaction and dynamic loading of slopes are the most likely failure mechanisms, but other seismic failure modes, which may be unique to a particular closed storage facility, must also be evaluated.

(3) What are the potential consequences of a failure?

In addition to understanding the probability of failure, a risk assessment should consider the potential consequences. A failure is likely to have economic costs associated with clean-up and restoration of the site. Depending on the local site conditions, failure of a closed CCP facility may or may not cause significant impacts on the environment, waterways, transportation routes, buried or overhead utilities, or other infrastructure. Substantial economic costs would result if power generation is interrupted. Failure consequences may also include the potential loss of human life at some sites.

In this proposed seismic risk assessment, the definition of "failure" will be constrained to mean the displacement of stored materials to a distance beyond the permitted boundary of the facility. While smaller deformations in a closed storage facility could cause economic damages, the resulting consequences for TVA should be manageable. Hence, this risk assessment will focus on potential "failures" where stored materials could move past the permitted boundary.

(4) What are the approximate costs to mitigate the risks of a seismic failure?

With an understanding of the probability and consequences of failure, the potential risks can be quantified and understood, possibly leading to decisions to mitigate seismic risks in the closure of certain facilities. Mitigation measures might include ground improvement to reduce liquefaction potential (stone columns, deep soil mixing, jet grouting, or other appropriate technology), stabilization of slopes by flattening or buttressing, enhanced drainage features, or some other engineered solution. The potential cost of these risk mitigation strategies are needed to make appropriate management decisions.

PORTFOLIO AND FACILITY ASSESSMENTS

Seismic evaluations will be completed for each of the CCP storage facilities that TVA has slated for closure; a tentative list is given in Table 1. The assessment of seismic risks will be accomplished in two phases:

A. Portfolio Seismic Assessment

In this first phase, the seismic risk assessment will be carried out using general site information, simplified analyses, empirical methods, and the judgment of experienced engineers. A team of four to five engineers will complete this evaluation for the entire portfolio, with assistance from the engineering teams currently working on each facility. After the probabilistic seismic hazards are defined, this phase of the work can be completed in a relatively short timeframe.

Given the level of effort and the simplified engineering analyses to be employed, the seismic risk estimates from the Phase A assessment will be approximate. Rather than attempting to compute precise risk numbers, Phase A will focus on capturing the relative risks between the different closed facilities. The key to successfully meeting this objective will be the consistent application of the assessment process across the portfolio.

This effort will result in a ranked list of sites that can be used to illustrate where seismic risks are greatest within the portfolio. The results will also provide some insight for understanding and communicating the magnitude of potential risks associated with seismic loading of the closed CCP facilities.

As a secondary objective, the Phase A assessment team will also consider the potential for failure of the active storage facilities, due to an earthquake occurring prior to closure. The seismic risks associated with the operating facility will not be estimated, but the Phase A assessment process provides an opportunity to identify potential failure mechanisms that should be addressed in the short term. This information may suggest the need to re-prioritize the closure schedule. Prior to closure, many of the wet CCP storage facilities retain large pools of water and are thus more susceptible to uncontrolled releases in an earthquake. TVA has already made the decision to close these wet storage facilities to manage these risks, so the effort in Phase A will focus on identifying sites that may have unusually high seismic risks and deserve more study or higher priority in the closure program.

B. Facility Seismic Assessment

In this subsequent phase of work, more detailed engineering analyses will be carried out using site-specific geometry, subsurface conditions, material parameters, and results from static slope stability analyses. Simplified, state-of-the-practice methods of engineering analysis will be used; more complex analytical methods will be generally impractical for this risk assessment.

This phase of the work will be accomplished for individual facilities as part of the closure design, after the completion of other engineering analyses. The risks will be quantified by the design team, with assistance from the portfolio seismic assessment team. Significant, detailed effort will be required to assess each closed facility.

Compared to Phase A, the risk estimates obtained at this stage will be more reliable and better represent the actual risks for seismic failure. While it will be impossible to know how accurately the risks have been characterized at the completion of Phase B, the objective is to obtain results that are within perhaps ± 30% of the "actual" risk numbers. TVA expects to use the Phase B results to decide if the risks are acceptable, or if the closure design should be modified to mitigate risks for a seismic failure.

The engineering methodology (described below) to be followed in the Phase A and B evaluations will not characterize all of the uncertainties with respect to seismic performance. The uncertainties in the soil parameters and in the liquefaction, stability, and deformation analyses will not be quantified and carried through the risk assessment. Consequently, the estimated risk numbers will be approximate, but the results will be sufficiently accurate to support TVA decisions regarding prioritization for closure or the need for seismic mitigation. At most sites, the risks are expected to be high enough or low enough that further refinement in the risk numbers would not change these decisions. More detailed analysis beyond Phase B would be unjustified in these cases.

This assessment plan does not preclude the possibility that more detailed risk evaluations could be undertaken in subsequent phases of work. The Phase B results might reveal a subset of closed facilities with marginal risks, where a more rigorous and complete calculation of the risks would be needed to support a management decision. Hence, at the conclusion of the Phase B assessments, a "Phase C" evaluation may be needed for select sites and facilities, wherein uncertainties in the soil parameters and performance analyses would be quantified and carried through the risk assessment.

RESULTS AND APPLICATION

The results from the Phase A Portfolio Assessment will be presented in a table, like Table 1. For each facility evaluated, the estimated annual probability of failure due to a seismic event, the expected consequences (economic costs and potential loss of life), and the mitigation costs (design features to reduce risks) will be tabulated. The same parameters, but more accurate numbers, will be reported from the more in-depth Phase B assessments. A qualitative description of the data quality (based on the number of borings, test data on key soil properties, etc.) will also be included, to indicate how well the site conditions were characterized at the time of the Phase A or B assessment.

In both Phase A and B, the evaluation teams will prepare a discussion of significant issues driving the seismic risks at each site. This summary will include knowledge gaps, likely failure mechanisms, unique consequences, suggested approaches for risk mitigation, and other key information. The Phase A evaluation of a facility may point out the need for additional data to support later seismic analyses in Phase B; needed field or laboratory testing could then be accomplished and documented as part of the facility closure design effort.

In the short term, TVA will utilize the Phase A results to better plan budgets and schedules for managing the closure process over the next several years. The Phase A assessment will also be used as an opportunity to identify operating facilities with especially high seismic risks. While these risks will not be quantified for conditions prior to closure, the consideration of potential seismic failure modes may prompt additional study and reconsideration of priorities. Where justified, the priorities for closure may be changed to more quickly address sites with higher seismic risks.

More accurate risk estimates will be obtained from the Phase B assessments, which will be completed as part of the closure design process. Those results will be used, within TVA's existing decision making framework, to judge if seismic mitigation is needed. For context, the criteria in Tables 2 and 3 represent the risk-based framework TVA uses to guide enterprise-level decisions. This framework relies upon broad, qualitative scoring of consequences and risks for the organization. For managing the seismic risks at the closed CCP facilities, complete probabilistic calculations of risk are not needed; approximate estimates of seismic risk will be sufficient to support TVA decisions.

The risks computed in Phase A and B will not be compared to a prescribed threshold or design risk level. Criteria for tolerable seismic risk in these closed CCP storage facilities has not been defined in the existing permits, in TVA policy, or in TVA design guidance.

METHODOLOGY

The same general methodology, outlined in ten steps below and in Figures 1 through 4, will be used to evaluate seismic risk in both the Phase A Portfolio Assessments and the Phase B Facility Assessments. While advanced engineering analyses may be required to demonstrate acceptable seismic performance in a design situation, simplified analyses will be used here, consistent with the goal of estimating the probability of failure.

In Step 1, seismic hazard parameters will be defined for each site; the results will be used as inputs for both the Phase A and Phase B assessments. Then, the evaluation of a particular facility will begin with a review of existing site information (Step 2), followed by engineering analyses for seismic performance. As described in Steps 3 through 7 below, the engineering analyses in Phase B will be more detailed than the simplified estimates in Phase A. The analyses will commence with an initial selection of an earthquake return period and evaluation for seismic performance. Steps 3 through 7 will be repeated until the limiting (lowest) earthquake return period expected to cause failure is obtained. Flowcharts summarizing Steps 1 through 7 in the Phase A and B seismic performance assessments are given in Figures 3 and 4, respectively. The earthquake event with the lowest return period that causes failure will then be used to compute the probability of failure in Step 8. The potential consequences and mitigation costs will be estimated in Steps 9 and 10.

Step 1 – Define Seismic Input Parameters

Seismic hazards at TVA dam sites were quantified in a 2004 study by AMEC/Geomatrix. The New Madrid fault zone and several area source zones contribute to the seismicity of the region, as represented schematically in Figure 1. The New Madrid seismic zone is characterized by a large linear, combined reverse/strike-slip fault. Earthquakes in the area source zones are more diffuse (less concentrated in clusters) and tend to occur in zones of weakness of large crustal extent rather than along narrow, well-defined faults. Earthquakes occurring within the New Madrid Seismic Zone and in area sources outside of it will be considered in developing seismic input parameters for each CCP facility. However, only seismic source zones that contribute significantly to the ground motion hazard at a particular site will be used to develop seismic input parameters.

The national USGS seismic hazard model will not be used in these seismic risk assessments; instead, TVA will ask AMEC/Geomatrix to compute the site-specific seismic hazards for each closed CCP facility. The needed information can be obtained from the existing seismogenic model, but will need to separately consider the hazards associated with the New Madrid events and all other seismic sources (Figure 2), hereafter referred to in this white paper as the "earthquake scenarios". The following parameters are needed for each earthquake scenario:

- Uniform hazard spectra for frequencies from 0.25 to 100 Hz (100 Hz value is equivalent to peak ground acceleration, PGA) at the top of rock for a range of return periods from 100 to 2,500 years.
- De-aggregation for relevant ground motion frequencies (one or more of the following: 0.5, 1.0, 2.5, 5.0, and 100 Hz) at each return period. The de-aggregation results will be used to select appropriate, representative earthquake parameters (magnitude and distance from the site), from which inputs needed for liquefaction analyses can be developed.

In the Phase A effort, the project team (including seismologists designated by TVA) will meet to consider the earthquake hazard data produced by the AMEC/Geomatrix model for each site. The team will reach consensus on the appropriate parameters (return period, earthquake magnitude, and peak ground acceleration) to be used in evaluating each facility, before proceeding with work on subsequent steps of the analysis. The seismic parameters to be tabulated (Table 4) will then be used in both the Phase A and Phase B assessments.

Ground motion time histories will be needed for the detailed Phase B calculations, and TVA will need to ask AMEC/Geomatrix to provide:

- Representative acceleration time histories (two orthogonal components), representing ground motions at the top of the rock profile for the specified earthquake return periods.

Given the results of the Phase A assessment, the Phase B analyses will focus on a narrower range of possible earthquakes. Hence, acceleration time histories will not be needed for every seismic event listed in Table 4.

Step 2 – Review Site and Facility Information

To meet the requirements for closure of TVA ash storage facilities, the closed condition may involve placement of compacted ash behind a strengthened dike, drainage of pond water to the levels of the surrounding groundwater table, and capping of the area with native soils. The collection of available site information for each facility will be reviewed from a seismic performance perspective. For the Phase B assessment, this information will be augmented with new data that becomes available during the closure design process.

The project information needed for each storage facility includes:

- Planned geometry of the closed storage facility, as needed to meet current design criteria and regulatory requirements.
- Geologic mapping and related information about the site geology.
- Historical records and other information related to site development.
- Boring logs, SPT data, CPT data, shear wave velocities, etc. from field explorations.
- Laboratory data from testing of site materials, including classification, Atterberg limits, moisture content, particle size, specific gravity, unit weight, compaction tests, and other relevant test data.
- Laboratory data on measured strength properties, for both drained and undrained conditions.
- Previously completed slope stability analyses, where available, will be modified for calculations in the risk assessments.

Step 3 - Evaluate Potential for Soil Liquefaction

The potential for soil liquefaction may be the greatest contributor to failure risk at many of the TVA storage sites. Liquefaction will thus be considered first in the assessment of seismic performance at each closed facility (Figures 3 and 4).

The Phase A assessment will utilize empirical charts and back-of-the-envelope calculations to judge if liquefaction would be likely for a given earthquake scenario. For example, Ambraseys (1988) compiled magnitude, epicentral distance, and whether or not liquefaction was observed in past earthquakes, and then suggested a threshold boundary (in terms of magnitude and epicentral distance) where liquefaction might occur in natural soil deposits. Selected, parametric calculations with the simplified procedure outlined by Youd et al (2001) will also be useful in judging what earthquakes would cause liquefaction in the Phase A Portfolio Assessments. These empirical methods may be unconservative for evaluating saturated CCPs, which are often more prone to liquefaction than a sandy soil, but the results will still provide useful guidance in the Phase A assessment.

For the Phase B liquefaction evaluations, detailed engineering analyses will be undertaken to obtain estimates of cyclic loading, soil resistance, and factor of safety as described below. Potentially liquefiable soils include saturated alluvial soils, loose granular fills, and sluiced ash. The detailed analyses will focus on critical cross sections of the closed facilities; liquefaction safety factors will not be computed for all boring locations at a site.

(a) Soil Loading from Earthquake Motions

The magnitude of the cyclic shear stresses induced by an earthquake are represented by the cyclic stress ratio (CSR). The simplified method proposed by Seed and Idriss (1971) will be used to estimate CSR in the Phase A parametric analyses (ground response analyses will not be completed in Phase A).

In Phase B, the CSR at specific locations (borings and depths where in situ penetration resistance are measured) will be computed using one-dimensional, equivalent-linear elastic methods as implemented in the ProSHAKE software. Using an acceleration time history at the top of rock (obtained from the seismic hazards study in Step 1), the computer program will model the upward propagation of the ground motions through a one-dimensional soil profile. For cases where the one-dimensional assumption is inadequate, the calculations can be accomplished using QUAKE, a two-dimensional finite element program that implements the same dynamic modulus reduction curves and damping relationships as used in ProSHAKE.

The cyclic stresses imparted to the soil will be estimated from the earthquake parameters described in Step 1, representing earthquakes on the New Madrid fault and local crustal events.

(b) Soil Resistance from Correlations with Penetration Resistance

The resistance to soil liquefaction, expressed in terms of the cyclic resistance ratio (CRR), will be assessed using the NCEER empirical methodology (Youd et al. 2001). Updates to the procedure from recently published research will be used where warranted. The analyses will be based on the blowcount value (N) measured in the Standard Penetration Test (SPT) or the tip resistance (q_c) measured in the Cone Penetration Test (CPT). In Phase A, typical or representative values will be used in parametric hand calculations; detailed data from site-specific explorations will be analyzed in Phase B.

The NCEER procedure involves a large number of correction factors. Based on the site-specific conditions and soil characteristics, engineering judgment will be used to select appropriate correction factors consistent with the consensus recommendations of the NCEER panel (Youd et al. 2001). To avoid inappropriately inflating the CRR, the NCEER fines content adjustment will not be applied where zero blowcounts (“weight of hammer” or “weight of rod”) are recorded. The magnitude scaling factor (MSF) is used in the empirical liquefaction procedure to normalize the representative earthquake magnitude to a baseline 7.5M earthquake. The earthquake magnitude (M) considered to be most representative of the liquefaction risk will be determined by applying the MSF to the de-aggregation data (from Step 1) for each selected earthquake return period.

Saturated fly ash, where it remains following closure, is likely to be more susceptible to liquefaction than indicated by these empirical methods. Values of CRR determined via the NCEER procedure are related to the observation of liquefaction in natural soils, mostly silty sands. Given the spherical particle shape and uniform, small grain size of fly ash, the NCEER procedure may give CRR values that are too high for saturated fly ash.

Lacking better methods of analysis, the lower-bound, "clean sand" base curve (Youd et al. 2001) will be assumed to apply for fly ash in the Phase A assessment. Within the liquefaction calculations, this will be accomplished for these materials by neglecting the fines content adjustment to the normalized penetration resistance. For Phase B, published and unpublished data from cyclic laboratory testing on similar materials will be sought to augment the indications of liquefaction resistance obtained from in situ penetration tests.

(c) Factor of Safety Against Liquefaction

The factor of safety against liquefaction (FS_{liq}) is defined as the ratio of the liquefaction resistance (CRR) over the earthquake load (CSR). Following TVA design guidance and the precedent set by Seed and Harder (1990), FS_{liq} is interpreted as follows:

- Soil will liquefy where $FS_{liq} \le 1.1$.
- Expect substantial soil softening where $1.1 < FS_{liq} \le 1.4$.
- Soil does not liquefy where $FS_{liq} > 1.4$.

Using this criteria for guidance, values of FS_{liq} computed throughout a soil deposit or cross section (at specific CPT-q_c and SPT-N locations) will be reviewed in aggregate. Occasional pockets of liquefied material in isolated locations are unlikely to induce a larger failure, and are typically considered tolerable. Instead, problems associated with soil liquefaction are indicated where continuous zones of significant lateral extent exhibit low values of FS_{liq}. Engineering judgment, including consideration for the likely performance in critical areas, will be used for the overall assessment of each facility. A determination of "extensive" or "insignificant" liquefaction will then lead to the appropriate stability analyses in the next stage of the evaluation, as indicated in Figures 3 and 4.

Step 4 – Characterize Post-Earthquake Soil Strengths

The post-earthquake shearing resistance of each soil and CCP will be estimated, with consideration for the specific characteristics of that material. The full, static shear strength will be assigned to unsaturated soils. Excess pore pressures will not develop in an unsaturated soil during seismic loading, so drained strength parameters can be used. The undrained strengths of saturated soils will be decreased to account for the softening effects of pore pressure buildup during the earthquake. Specifically:

- In saturated clays and soils with $FS_{liq} > 1.4$, 80% of the static undrained strength will be assumed.
- In saturated, low-plasticity, granular soils with $1.1 < FS_{liq} \le 1.4$, a reduced strength will be assigned, based on the excess pore pressure ratio, r_u (Seed and Harder 1990).

Typical relationships between FS_{liq} and r_u have been published by Marcuson and Hynes (1989).

- In saturated, low-plasticity, granular soils with $FS_{liq} \leq 1.1$, a residual (steady state) strength (S_{us}) will be estimated for the liquefied soil. Values of S_{us} can be obtained from the empirical correlations published by Seed and Harder (1990), Castro (1995), Olson and Stark (2002), Seed et al. (2003), and Idriss and Boulanger (2008).

Subsequent stability and deformation analyses will be accomplished using these reduced strength parameters. No attempt will be made to model the cyclic reduction in soil shear strength during an earthquake. In the deformation analyses, the fully reduced strengths will be assumed at the start of cyclic loading, which will yield conservative estimates of slope displacements.

Step 5 – Analyze Slope Stability

The next step in the performance evaluation (Figures 3 and 4) will consider slope stability, for conditions with or without significant liquefaction. Slope stability will be evaluated using two-dimensional, limit equilibrium, slope stability methods. Reduced soil strengths (from Step 4), conservatively representing the loss of shearing resistance due to cyclic pore pressure generation during the earthquake, will be used in the stability calculations. The analyses will be accomplished using Spencer's method of analysis, as implemented in the SLOPE/W software, considering both circular and translational slip mechanisms.

Input files for static stability calculations, where previously completed for a particular facility, will be updated to represent seismic conditions. These stability analyses may be not available, or the closure geometry may be undefined, for the Phase A assessment of some sites. In those cases, simplified or approximate geometries will be developed for approximate analysis in Phase A. Engineering experience will also be useful in judging likely seismic stability. For example, a complete failure is likely if liquefaction undermines the foundation of the outslope. In the absence of liquefaction, a slope that exhibits adequate safety factors under static conditions is unlikely to fail in an earthquake. Back-of-the-envelope hand calculations can be useful in assessing stability where extensive liquefaction occurs in the saturated materials within or below CCPs retained by a stable perimeter dike. Detailed slope stability calculations, which accurately represent the planned closure geometry, will be used in the Phase B facility assessments.

(a) Slope Stability if Extensive Liquefaction

If extensive liquefaction is indicated, stability will be evaluated for the static conditions immediately following the cessation of the earthquake motions. Residual or steady state strengths will be assigned in zones of liquefied soil, with reduced strengths that account for cyclic softening and pore pressure build up assumed in non-liquefied soil. In both Phase A and B, complete failure (large, unacceptable displacements) will be assumed if the safety factor (FS_{slope}) computed in this step is less than one (Figures 3 and 4).

For slopes where the post-earthquake $FS_{slope} \geq 1$, deformations will be estimated in the Phase B assessment (Step 6 and Figure 4). Slope deformations will not be estimated in the Phase A portfolio assessment, where ground motion time histories will not be available. In Phase A, slopes exhibiting $FS_{slope} \geq 1$ with liquefaction will be assumed stable with tolerable deformations; this condition may exist, for example, where liquefied ash at the base of a closed storage facility is contained within a stable perimeter dike.

Note that pseudostatic stability analyses are not useful for evaluating a factor of safety where extensive liquefaction is expected, because appropriate pseudostatic coefficients can not be defined.

(b) Slope Stability if No Significant Liquefaction

If no significant liquefaction is expected, seismic stability will be analyzed in Phase A using approximate, pseudostatic stability methods (Figure 3). The added inertial loads from the earthquake will be represented with a simple, horizontal pseudostatic coefficient (k_h), which provides an approximate representation of the dynamic loads imposed by an earthquake. The horizontal pseudostatic coefficient will be set to one-tenth of the peak ground acceleration in rock ($k_h = 0.1 \cdot PGA_{rock}$). In Phase A, tolerable deformations (less than about 5 meters) will be assumed if the pseudostatic $FS_{slope} \geq 1$, and failure will be assumed if the pseudostatic $FS_{slope} < 1$.

This approach and criteria are based on the work of Hynes-Griffin and Franklin (1984). They performed Newmark deformation analyses, integrated over 350 ground motion time histories, used an amplification factor of three to represent peak accelerations at the base of an earth embankment, and assumed a displacement of 1 meter would be tolerable for an embankment dam. For a typical CCP facility, assuming no pool is retained following closure, "failure" would imply displacements significantly greater than 1 meter. A tolerable displacement of about 5 meters will be assumed here, for the Phase A risk assessments. From the upper bound curve plotted by Hynes-Griffin and Franklin (1984), a displacement of 5 meters would correspond to a yield acceleration of about 0.03 times the peak acceleration along the slip surface. Then, assuming an amplification factor of 3 for the ground motions at the base of the embankment, this suggests $k_h = 0.1 \cdot PGA_{rock}$ can be used conservatively in the pseudostatic analysis to judge failure, as described above.

Pseudostatic factors of safety will not be computed in the Phase B assessment. Instead, where a liquefaction failure is not predicted, potential slope displacements will be computed as described in Step 6.

Step 6 – Predict Deformations

In the Phase A Portfolio Assessment, closed facilities that are expected to remain stable (pseudostatic $FS_{slope} \geq 1$ with no liquefaction, or post-earthquake $FS_{slope} \geq 1$ with liquefaction) will be assumed to have tolerable displacements. Dynamic slope deformations are difficult to estimate without detailed analysis; the available empirical or approximate methods do not represent the conditions of interest, or the level of effort is not consistent with the goals of the first phase of risk assessments. In addition, earthquake ground motion time histories will not be available for the Phase A analyses.

In the Phase B Facility Assessments, the potential deformation of stable slopes will be evaluated as indicated in Figure 4. Conventional methods of analysis will be implemented to estimate potential slope displacements that accumulate during earthquake shaking; movements are assumed to stop when the earthquake ends, consistent with a post-earthquake safety factor greater than one. The acceleration time histories obtained from the ground response analyses in Step 3a will be used as inputs for computing deformations with one of the following simplified methods:

- Newmark's (1965) method involves double integration of accelerations greater than the yield acceleration (k_y), which will be determined from a succession of pseudostatic slope stability analyses in which k_h is varied. The value of k_h where the pseudostatic FS_{slope} = 1.0 corresponds to the yield acceleration.
- The Makdisi-Seed (1978, 1979) procedure, which better accounts for the dynamic response of embankments. This procedure was developed based on parametric numerical simulations for earthen dams. The procedure is iterative, considers the fundamental periods of the embankment response, and can be completed in steps using published charts. Results from QUAKE can also be used as input in this procedure.

The slope deformations predicted in Phase B will be conservative, because the yield acceleration will be computed based on reduced, post-earthquake soil strengths. In reality, the yield acceleration declines in successive cycles of seismic loading, as pore pressures accumulate and saturated soils become weaker. The analysis outlined in Figure 4 assumes reduced strengths and, where liquefaction is predicted, residual strengths at the start of the earthquake. Detailed numerical simulations can be used to track the progressive softening and liquefaction of soil within an embankment during an earthquake; such analyses are expensive and time consuming. Rigorous analyses of this type will not be justified except in a "Phase C" analysis, or where performance in a given seismic design event must be demonstrated. Note that the logic in Figure 4 might appear to assume a slope will be stable if there is no significant liquefaction; however, the deformation analysis will indicate unlimited deformations and certain failure if $FS_{slope} < 1$ for static, post-earthquake conditions.

Step 7 – Consider Other Potential Failure Modes

For most of the closed facilities, soil liquefaction, slope instability, and slope deformations will be the most likely seismic failure modes. However, depending on the unique configuration of each CCP facility, other potential failure modes may contribute significantly to the seismic risks. For example, the loss of critical drainage structures or retaining walls could lead to a failure condition. Other potential failure modes will be identified and evaluated quantitatively in this step.

As a secondary objective of the Phase A effort, the assessment team will consider the potential for failure of the active storage facilities, due to an earthquake occurring prior to closure. Many of the wet CCP storage facilities retain large pools of water, so this assessment will need to consider additional failure modes such as seepage and embankment cracking. The objective here will be to identify operating facilities that may have unusually high seismic risks, and might deserve more study or higher priority in the closure program.

Step 8 – Estimate Annual Probability of Seismic Failure

As indicated in the flowcharts in Figures 3 and 4, the assessments of seismic performance (in both the Phase A and Phase B efforts) will consider a range of potential earthquakes with differing return periods. The analyses will be repeated until the limiting (lowest) earthquake return period (from the candidate events defined in Step 1) that predicts failure of a particular CCP storage facility is obtained. Interpolation may be used, as appropriate, to narrow the definition of the limiting earthquake.

The return period for each earthquake scenario (Table 4) represents the annual probability of exceedance for the associated ground motion parameter. Hence, for each earthquake scenario, the event with the smallest return period that causes failure represents a limiting case, where all events having longer return periods would also cause failure. The inverse of the limiting return period thus represents the annual probability of seismic failure due to that earthquake scenario.

Step 9 – Estimate Potential Consequences of Failure

The potential consequences of a failure at each closed facility will be estimated in this step. The potential consequences will be unique to each site, but may include any of the following:

- restoration of the site and storage facility,
- clean-up to address environmental impacts,
- off-site disposal of released materials,
- damages and loss of use for transportation routes, including buried or overhead utilities,
- damages to buildings and other infrastructure,
- economic losses from the possible shutdown of power generation, and
- loss of human life (expected to be unlikely at most sites following closure).

Except for the potential loss of life, the failure consequences will be expressed in terms of present day costs. Detailed cost estimates of the potential consequences of failure will not be attempted in the Phase A assessments; instead, the potential magnitude of total consequence costs will be estimated using broad categories (< $100K, < $500K, < $1M, < $5M, < $10M, < $50M, < $100M). Cost estimates that better reflect the local site conditions will be produced by the closure design teams during the Phase B assessments.

Step 10 – Estimate Possible Mitigation Costs

The final step in the process will involve estimating the costs to mitigate seismic risks, perhaps by altering the closure design to withstand stronger earthquakes. Examples of possible mitigation measures include:

- ground improvements to reduce liquefaction potential (stone columns, deep soil mixing, jet grouting, or other appropriate technology),
- altering the geometry of outslopes (setbacks, benches, or flatter slopes) to improve

stability,

- adding buttresses or other supporting structures at the toe of slopes,
- enhanced drainage features, and
- relocation of infrastructure or people away from potential impact zones.

These mitigation approaches generally involve higher construction costs, which can be quantified in terms of present dollars. As with the consequence costs, detailed estimates of mitigation costs will not be attempted in the Phase A assessments. The potential magnitude of mitigation will be estimated in categories (< $100K, < $500K, < $1M, < $5M, < $10M, < $50M, < $100M). Mitigation cost estimates that better reflect the local conditions and facility layout will be developed by the closure design teams during the Phase B assessments.

Table 1. Expected Results from the Phase A and B Seismic Risk Assessments

TVA Facility	Prob. Failure	Econ. Costs	Loss of Life	Mitigat. Costs	Data Quality
ALF East Ash Disposal					
ALF East Stilling Pond					
BRF Dry Fly Ash Disposal					
BRF Fly Ash Pond And Stilling Basin Area 2					
BRF Bottom Ash Disposal Area 1					
BRF Gypsum Disposal Area 2a					
COF Disposal Area 5					
COF Ash Pond 4					
CUF Dry Ash Stack					
CUF Ash Pond					
CUF Gypsum Storage Area					
GAF Fly Ash Pond E					
GAF Bottom Ash Pond A					
GAF Stilling Pond B, C & D					
JSF Dry Fly Ash Stack					
JSF Bottom Ash Disposal Area 2					
JOF Ash Disposal Area 2					
KIF Dike C					
PAF Scrubber Sludge Complex					
PAF Peabody Ash Pond					
PAF Slag Areas 2a & 2b					
SHF Consolidated Waste Dry Stack					
SHF Ash Pond					
WCF Ash Pond Complex					
WCF Gypsum Stack					

Prob Failure = Annual probability of failure due to earthquakes
Econ. Costs = Economic costs resulting from a failure
Loss of Life = Potential loss of life resulting from a failure
Mitigat. Costs = Costs to mitigate seismic risks in closure design
Data Quality = Qualitative indication of how well conditions in the facility are characterized

Table 2. Risk Severity Scoring (Draft) used by TVA

as of 4/22/2009

TVA Risk Event Consequence Rating Scale (Work-In-Progress)						
Strategic Objective	Success Factor	5 Worst Case	4 Severe	3 Major	2 Moderate	1 Minor
Customer	Public Image	International media attention; nearly unanimous public criticism	National media attention; federal, state officials, and customers publicly critical	Regional / local media attention; customers voice concern	Minimal media attention; letters / emails to executive leadership voicing concern	No media attention; sparse criticism
	Rate Impact	Average total retail rate increases by 15%, relative to peers	Average total retail rate increases by 10%-15%, relative to peers	Average total retail rate increases by 5%-10%, relative to peers	Average total retail rate increases by 2%-5%, relative to peers	Average total retail rate increases by 0-2%, relative to peers
People	Safety	Fatalities	Wide spread injuries	Major injuries	Significant injuries	Minor injuries
	Employee Confidence	Widespread departures of key staff with scarce skills or knowledge	Sharp, sustained drop in CHI results; departures of key staff with scarce skills or knowledge	Sharp decline in CHI results	Modest decline in CHI results	No effect on CHI results
Financial	Cash Flow Impact	>$500M	$100M - $500M	$25M - $100M	$5M - $25M	<$5M
	Credit Worthiness	Credit rating downgrade to below investment grade	Credit Rating Downgrade	TVA put on credit watch	TVA put on negative outlook	Credit rating agencies and bondholders express concern
Assets and Operations	LNS (Load not served)*	10% of System Daily Sales (48,000 MWhrs)	1% of System Daily Sales (4,800 MWhrs)	0.1% of System Daily Sales (480 MWhrs)	0.05% of System Daily Sales (240 MWhrs)	140 MWhrs
	CPI (Connection Point Interruptions)	10% of CPs are down simultaneously	5% of CPs are down simultaneously	CPI totaling 10% of current CP count (124 for FY09)	CPI totaling 7.5% of current CP count (93 for FY09)	CPI totaling 5% of current CP count (62 for FY09)
	Duration (in Hours) of Service Interruption	3,000 cumulative hours for CPs	1,000 cumulative hours for CPs	500 cumulative hours for CPs	150 cumulative hours for CPs	50 cumulative hours for CPs
	Delivered Cost of Power	Sustained increase in delivered cost of power >1 year	Increase in delivered cost of power <1 year	Increase in delivered cost of power <1 month	Increase in delivered cost of power <1 week	Delivered cost of power not effected
	Damage to environment; type and magnitude of contamination / discharge	Major coal, nuclear plant accident or dam failure	Significant hazardous waste discharged; nuclear plant accident; dam integrity failure resulting in drawdown of pool elevation	Hazardous materials / waste discharge; clean up / remediation time takes approximately two weeks	Localized environmental damage, no impact to wildlife, clean up / remediation time less than two weeks	Minimal environmental damage, no hazardous discharge; clean up time takes a few days

Table 3. Risk Likelihood Scoring used by TVA

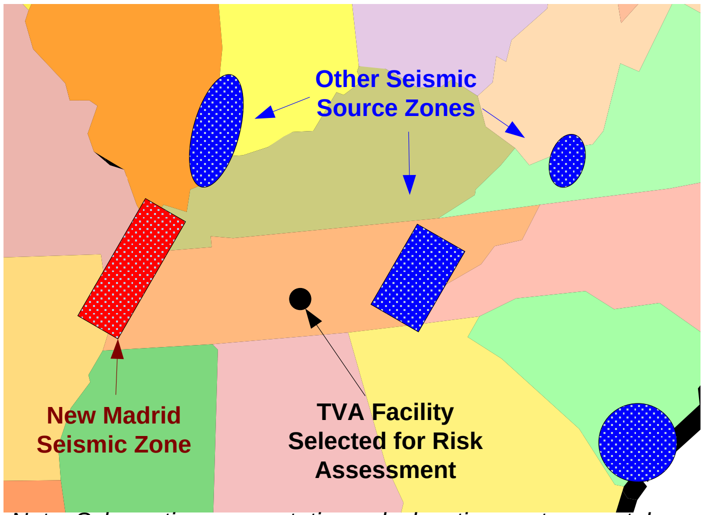
TVA Risk Event Probability Rating Scale		
Score	Rating	Description
5	Virtually Certain	95% probability that the event will occur in the next 3 years /10 years
4	Very Likely	75% probability that the event will occur in the next 3 years/10 years
3	Even Odds	50% probability that the event will occur in the next 3 years/10 years
2	Unlikely	25% probability that the event will occur in the next 3 years/10 years
1	Remote	5% probability that the event will occur in the next 3 years/10 years

- *The 3-year timeframe will be the primary focus for the business unit risk maps*
- *The 10-year risks will be collected by the ERM organization and charted separately for the enterprise*

Table 4. Seismic Hazard Input Data for Probabilistic Assessment of TVA Facilities

Seismic Sources	Return Period (years)	Annual Probability of Exceedance	Peak Ground Acceleration (g)	Earthquake Magnitude
New Madrid Seismic Zone	2,500 1,000 500 250 100	0.0004 0.001 0.002 0.004 0.01	*Values to be determined from the seismic hazard curves*	*Values to be determined from the hazard de-aggregation data**
All Other Seismic Sources	2,500 1,000 500 250 100	0.0004 0.001 0.002 0.004 0.01		

** Representative magnitude corresponding to the maximum contribution to the seismic hazard for liquefaction, as determined from the de-aggregation data weighted by the magnitude scaling factor (maximum PGA / MSF)*



Note: Schematic representation only, locations not accurately depicted, some sources omitted.

Figure 1. Schematic Representation of Seismic Source Model for TVA Facilities

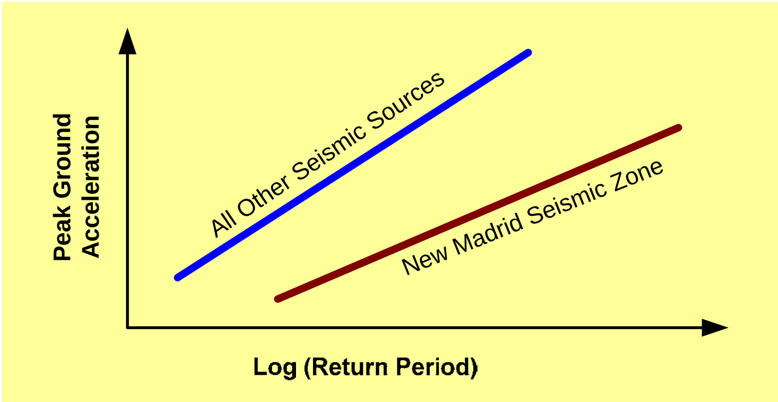


Figure 2. Typical Seismic Hazard Curves for Proposed Probabilistic Assessment of TVA Facilities

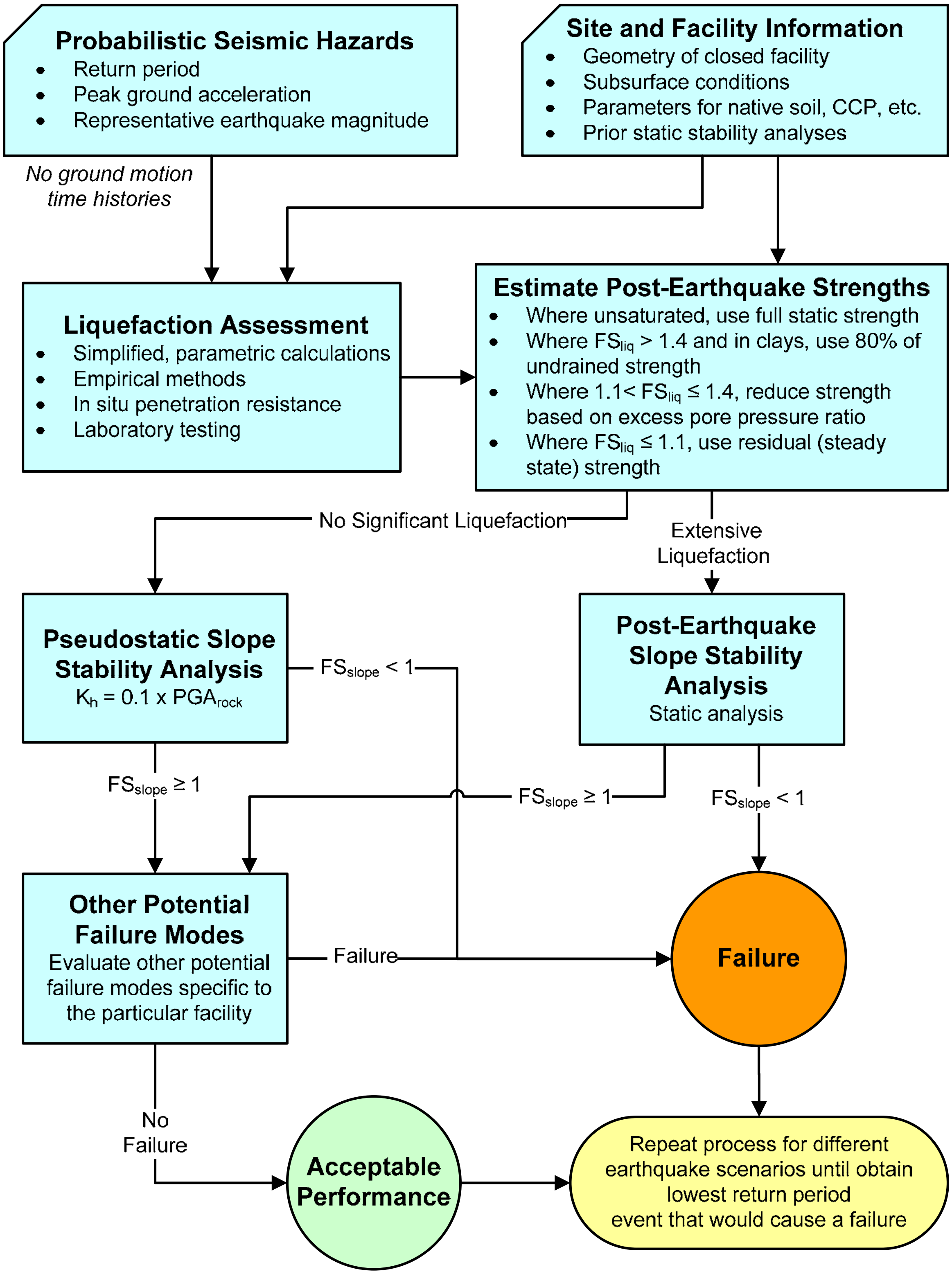


Figure 3. Simplified Flowchart for Assessing Facility Performance During a Probabilistic Seismic Event in Phase A

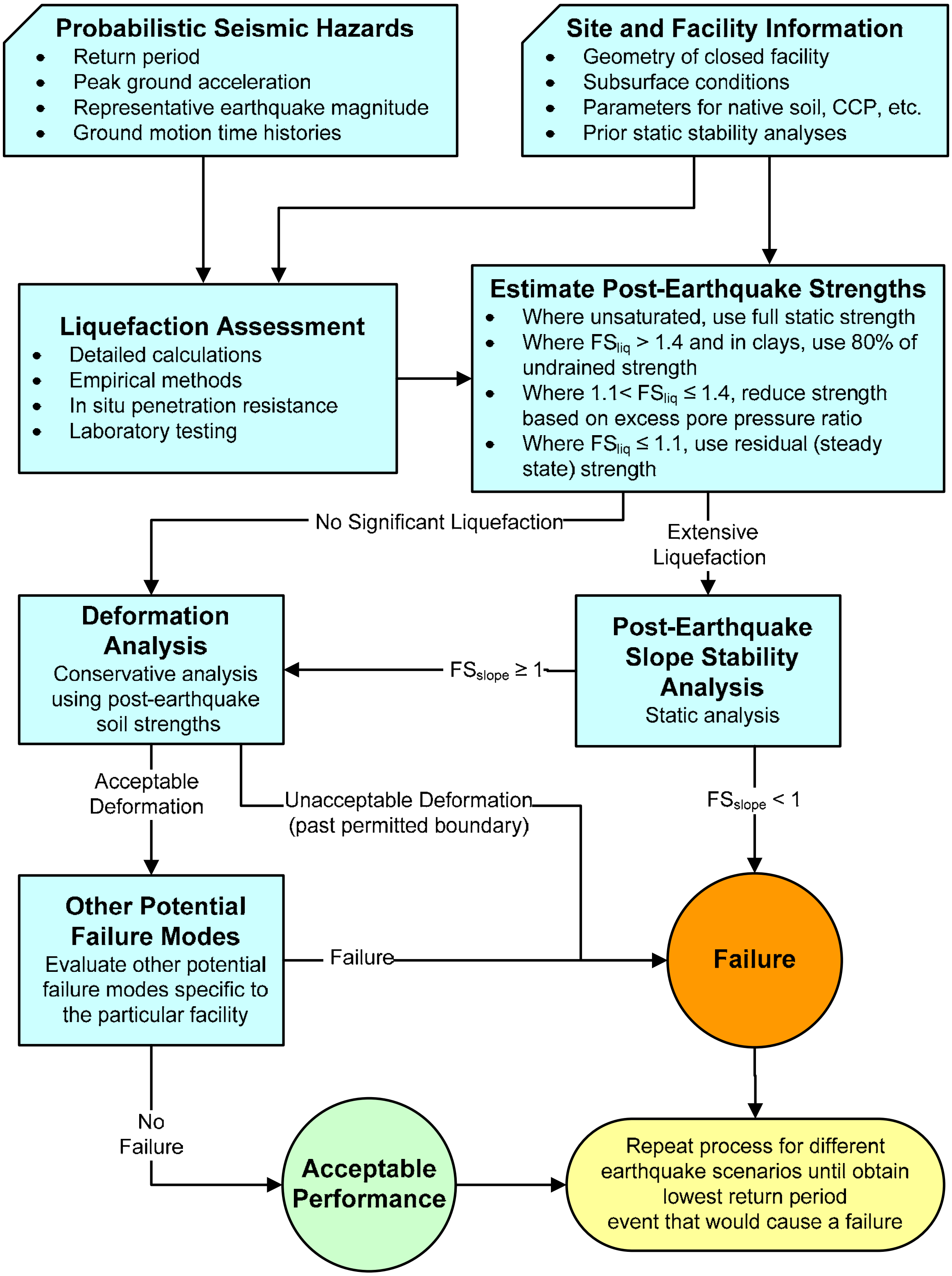


Figure 4. Simplified Flowchart for Assessing Facility Performance During a Probabilistic Seismic Event in Phase B

APPENDIX B

Document 6

Dam Inspection Check List Form

Site Name:	**Watts Bar Fossil Plant**	Date:	**September 15, 2011**
Unit Name:	**Stormwater BMP**	Operator's Name:	**Tennessee Valley Authority**
Unit I.D.:	**Old Ash/Ash Stilling Pond**	Hazard Potential Classification:	**High** ☐ **Significant** ☐ **Low X**
	Inspector's Name:	Stan Notestine, PE and Jim Filson, PE	

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)?	704		19. Major erosion or slope deterioration?		X
3. Decant inlet elevation (operator records)?	704		20. Decant Pipes:		
4. Open channel spillway elevation (operator records)?		X	Is water entering inlet, but not exiting outlet?		X
5. Lowest dam crest elevation (operator records)?	713		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)?		X	From underdrain?		N/A
9. Trees growing on embankment? (If so, indicate largest diameter below)	X		At isolated points on embankment slopes?		X
10. Cracks or scarps on crest?		X	At natural hillside in the embankment area?		X
11. Is there significant settlement along the crest?		X	Over widespread areas?		X
12. Are decant trashracks clear and in place?		X	From downstream foundation area?		X
13. Depressions or sinkholes in tailings surface or whirlpool in the pool area?		X	"Boils" beneath stream or ponded water?		X
14. Clogged spillways, groin or diversion ditches?		X	Around the outside of the decant pipe?		X
15. Are spillway or ditch linings deteriorated?		X	22. Surface movements in valley bottom or on hillside?		X
16. Are outlets of decant or underdrains blocked?		X	23. Water against downstream toe?		X
17. Cracks or scarps on slopes?		X	24. Were Photos taken during the dam inspection?	X	

Major adverse changes in these items could cause instability and should be reported for further evaluation. Adverse conditions noted in these items should normally be described (extent, location, volume, etc.) in the space below and on the back of this sheet.

Issue #	Comments
#1	Inspection – Annual, monthly, weekly, and special conditions (Note Plant has been closed for over 25 years)
#4	Never Overtopped
#5	Freeboard of 9'
#6	Monitoring wells and Piezometers – Not reading (Plant closed for over 25)
#9	3-4 trees in embankment (dia 3-4')

Coal Combustion Waste (CCW)

Impoundment Inspection

Impoundment NPDES Permit TN0005461 **INSPECTOR**

Date September 1, 2011

Impoundment Name SWM BMP – Ash/Stilling Pond

Impoundment Company TVA-Watts Bar Fossil Plant (WBF)

EPA Region Region 4

State Agency (Field Office) Address Tennessee Department of Environment and Conservation

Name of Impoundment Old Ash/Stilling Pond

(Report each impoundment on a separate form under the same Impoundment NPDES Permit number)

New ☐ **Update** X

	Yes	No
Is impoundment currently under construction?	☐	X
Is water or ccw currently being pumped into the impoundment?	☐	X

IMPOUNDMENT FUNCTION: Surface runoff

Nearest Downstream Town Name: Dayton, TN

Distance from the impoundment: Approximately 20 miles

Latitude	35	Degrees	36	Minutes	13.77	Seconds	**N**
Longitude	84	Degrees	469	Minutes	49.72	Seconds	**W**

State Tennessee **County** Rhea

	Yes	No
Does a state agency regulate this impoundment?	☐	X

If So Which State Agency? N/A

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.

X **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.

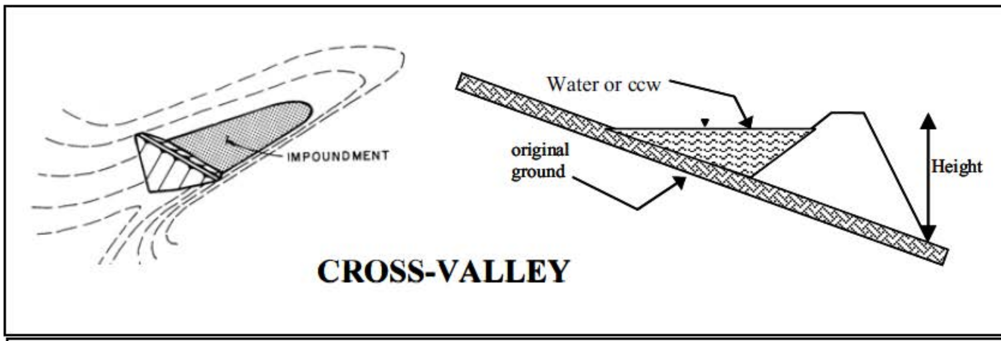
☐ **SIGNIFICANT HAZARD POTENTIAL:** Dams assigned the significant hazard potential classification are those dams where failure or misoperation results in no probable loss of human life but can cause economic loss, environmental damage, disruption of lifeline facilities, or can impact other concerns. 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.

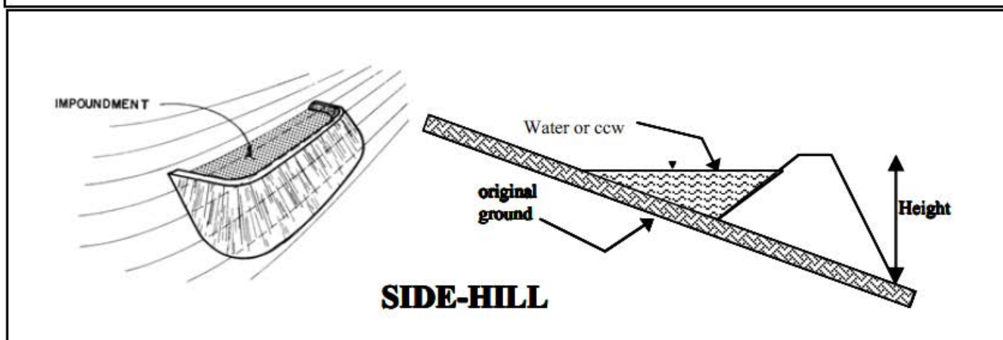
☐ **HIGH HAZARD POTENTIAL:** Dams assigned the high hazard potential classification are those where failure or misoperation will probably cause loss of human life.

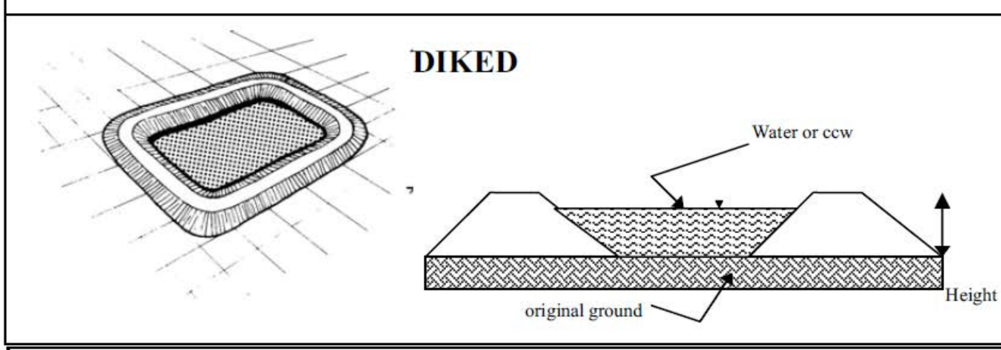
DESCRIBE REASONING FOR HAZARD RATING CHOSEN:

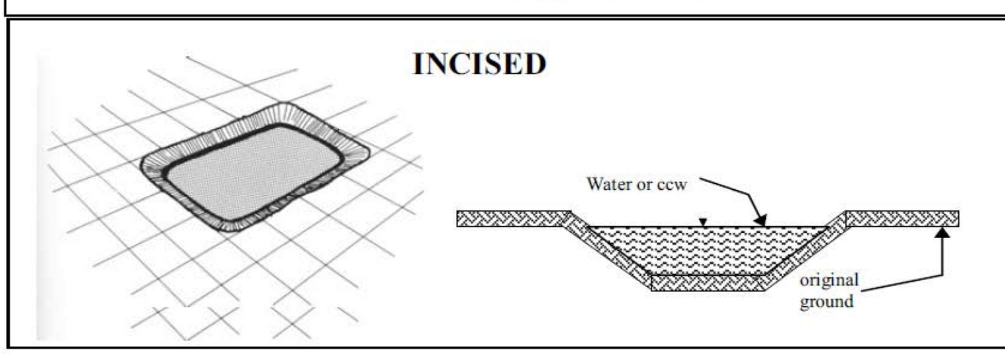
Low hazard based on no loss of human life with low environmental impacts.

CONFIGURATION:









☐ Cross-Valley X Side-Hill ☐ Diked

☐ Incised (form completion optional) ☐ Combination Incised/Diked

Embankment Height (ft) – 11 **Embankment Material** Clay

Pool Area (ac) 15 **Liner** N/A

Current Freeboard (ft) 9 ft **Liner Permeability** N/A

TYPE OF OUTLET (Mark all that apply)

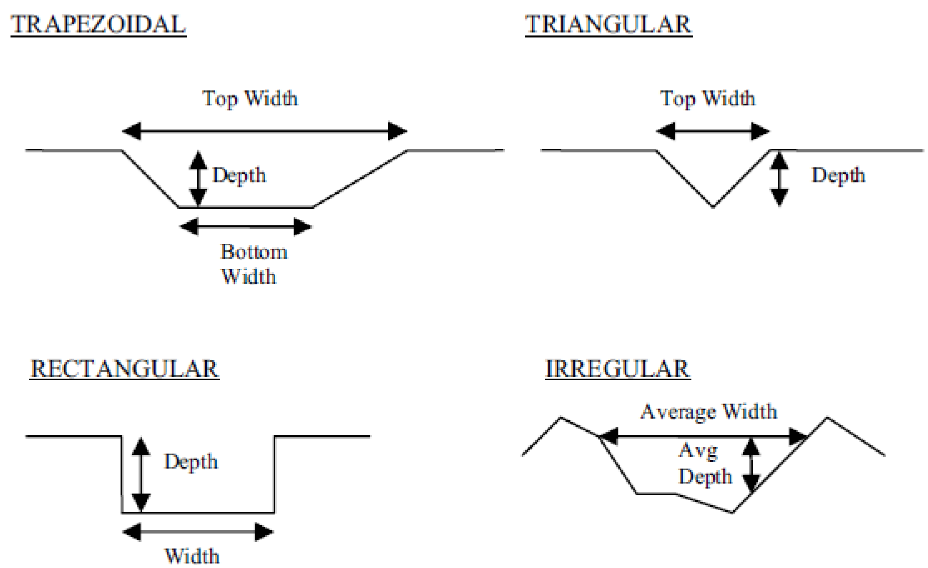
X **Open Channel Spillway**

- X Trapezoidal
- ☐ Triangular
- ☐ Rectangular
- ☐ Irregular

depth (ft) = 2

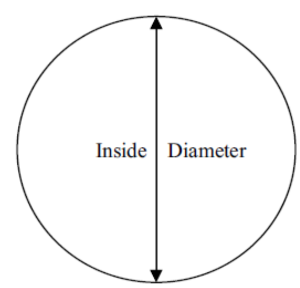
average bottom width (ft) = 15

top width (ft) = 20



X **Outlet**

3 -36 inch Pipes (inside diameter)



Material

- ☐ corrugated metal
- ☐ welded steel
- X concrete
- ☐ plastic (hdpe, pvc, etc.)
- ☐ other (specify):

	Yes	No
Is water flowing through the outlet?	X	☐

☐ **No Outlet**

☐ **Other Type of Outlet** (specify):

The Impoundment was Designed By **Tennsessee Valley Authority**

	Yes	No
Has there ever been a failure at this site?	☐	X
If So When?		

If So Please Describe :

	Yes	No
Has there ever been significant seepages at this site?	☐	X
If So When?		

If So Please Describe :

	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?	☐	X
If so, which method (e.g., piezometers, gw pumping,...)?		

If So Please Describe :

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 – based on best records

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