

# INACTIVE CCR SURFACE IMPOUNDMENT

Design and Operating Criteria

**CFR 257.71; 257.73; 257.82**

Pond 1

Clinch River Plant

April, 2018  
(Revised June 20, 2018)

Prepared for: Appalachian Power Company – Clinch River Plant

Carbo, Virginia

Prepared by: American Electric Power Service Corporation

1 Riverside Plaza

Columbus, OH 43215



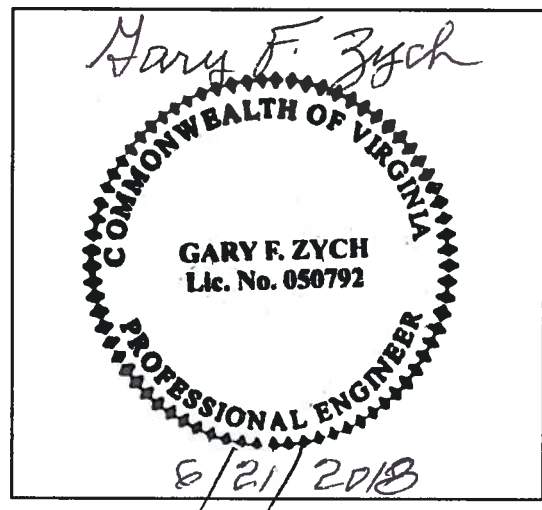
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INACTIVE CCR SURFACE IMPOUNDMENT  
Design and Operating Criteria  
CLINCH RIVER PLANT  
POND 1

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I certify to the best of my knowledge, information and belief that the information contained in this summary meets the requirements of 40 CFR 257.71; 257.73 and 257.82

## Table of CONTENTS

<b>1.0 OBJECTIVE.....</b>	<b>4</b>
<b>2.0 NAME AND DESCRIPTION OF INACTIVE CCR SURFACE IMPOUNDMENT .....</b>	<b>4</b>

Attachment A: Closure Design Report (report text only)  
Attachment B: Figures and drawings

Revised April 30, 2018 – deleted the last sentence related to Post Closure.  
Revised June 20, 2018 – added location map and design drawings of the impoundment prior to Closure.

## **1.0 OBJECTIVE**

This report was prepared by AEP- Geotechnical Engineering Services (GES) section to fulfill requirements of CFR 257 Subpart D for Inactive CCR Surface Impoundments.

## **2.0 NAME AND DESCRIPTION OF INACTIVE CCR SURFACE IMPOUNDMENT**

The Clinch River Power Plant is located near Carbo, Russell County, Virginia.

It is owned and operated by Appalachian Power Company (APCO). The facility owns an inactive CCR surface impoundment that was used for permanent disposal of sluiced CCR materials, referenced as the Ash Pond Complex (Pond 1).

At the Clinch River Plant, the Ash Pond Complex consists of Ash Pond 1A, Ash Pond 1B and the Reclaim Pond as shown in Figure 1. The two ash ponds are formed by earthen embankments approximately 60-ft. high on the west, south and east sides; a splitter embankment in the center; and natural high ground along the north side. The embankments have interior slopes of approximately 3 Horizontal to 1 Vertical (3H to 1V) and exterior slopes of approximately 2H to 1V. The exterior dike on the south side has an underdrain and finger drain system installed downstream of the toe road to control and collect seepage. The Reclaim Pond is an incised pond.

The dam is classified as a High Hazard Dam and is an unlined surface impoundment.

The operation of the CCR unit ceased on October 9, 2015 and therefore was not operating as of October 14, 2015. The closure of the CCR unit started in 2014 with a site investigation and engineering report. The closure report was filed with applications to the State of Virginia Department of Conservation and Recreation, Dam Safety Group and the Department of Environmental Protection to close the ash pond.

Approval was obtained and construction activities started in August 2016. The construction was substantially completed by the March 30, 2018. The closure of the CCR Unit was closure in place with a CCR compliant cap system, consisting of a geomembrane and two-foot thick cover soil and vegetation.

The closure report referenced above addressed all items related to Structural Integrity as per CFR 257.73, CFR 257.74 and CFR 257.82. As part of the closure design, new discharge channels were constructed such that the dam no longer impounds any stormwater runoff from the watershed. The channels were designed to pass the peak discharge of the 0.9 PMP design storm as required the Virginia Dam Safety.

The text portion of the design report for the closure is in Attachment A.

Attachment B contains the Location Map (Figure 1) and copies of the design plan and cross sections of Pond 1 dikes prior to Closure.



**ATTACHMENT A**

**Closure Design Report (report text only)**

# Closure Plan

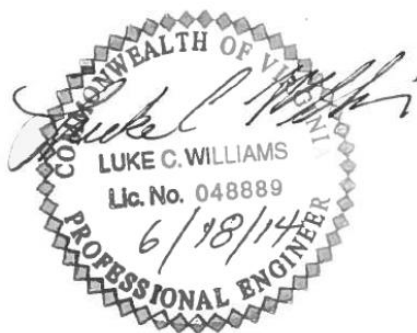
Ash Pond 1  
Clinch River Plant

Appalachian Power Company  
Clinch River Plant, Carbo, Virginia

April 2014 (Revised November 2016)

Prepared by: American Electric Power Service Corporation  
and Appalachian Power Company

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## Table of Contents

I.	Closure Plan .....	2
A.	Closure Activities.....	2
A.1	Closure Plan Time-Frames .....	2
A.2	Closure Performance Standard.....	2
A.3	Inventory Removal and Disposal .....	3
A.4	Leachate/Seep Management .....	3
A.5	Closure .....	3
A.6	Schedule for Closure .....	4
A.7	Posting .....	4
A.8	Notification.....	4
A.9	Certification.....	4
A.10	Closure Cost Estimate .....	4
A.11	Post Closure Cost Estimate .....	4
B.	Closure Calculations .....	5
B.1	Cover System Stability and Liquids Management.....	5
B.2	Settlement, Subsidence and Displacement .....	5
B.3	Freeze and Thaw Effects.....	5
C.	Construction Specifications .....	5
D.	Groundwater .....	5

Attachment A: Pond Closure Drawings

Attachment B: Geotechnical Analyses

Attachment C: Engineering Report and Calculations

Attachment D: Construction Quality Assurance Plan

Attachment E: Construction Specifications

Attachment F: Closure Cost Estimate

Attachment G: Alternate Final Cover Demonstration

Attachment H: CCR Closure Plan Compliance (40 CFR 257.102)

## **I. Closure Plan**

Appalachian Power Company (APCo), doing business as American Electric Power (AEP), is submitting a landfill application for Closure of Ash Pond 1 at the Clinch River Plant in Russell County, Virginia. Ash Pond 1 was used to dispose of bottom ash produced at the Clinch River Plant. As of October 19, 2015, ash will no longer be disposed of in Pond 1 in order to be an inactive surface impoundment per the Federal Coal Combustion Residual rules. Pond 1 will receive water and solids from the Plant's advanced waste water treatment plant until April of 2016. The pond will be dewatered shortly after and will undergo closure.

### **A. Closure Activities**

#### **A.1 Closure Plan Time-Frames**

This application anticipates closure to be completed before September of 2018.

#### **A.2 Closure Performance Standard**

This closure plan satisfies the requirements of 40 CFR 257.102 of the Federal CCR Rule, which is evidenced by and further detailed in Attachment H.

Closure of the site will consist of covering Ash Pond 1 with a geomembrane cap, a geocomposite drainage net, and soil cover to minimize infiltration of precipitation.

The Closure Plan has been developed to minimize post-closure maintenance. Ash Pond 1 will be covered with soil and seeded using a mixture of perennial cool-season grasses, a perennial legume, and for quick cover, annual grain species. This mixture should produce a thick, maintenance-free vegetative cover to minimize erosion. Erosion resistant materials are also used as needed to prevent erosion in surface drainage conveyances.

Stormwater runoff from the Pond 1 watershed will be diverted away from the final grades of the protective cap as shown on the plans. Rainfall runoff from the watershed flows overland (and concentrated at one location) towards Pond 1 from the north side of the pond. These flows will be collected above Pond 1 in either the existing Pond 1A Diversion Channel or the proposed Pond 1B Diversion Channel.

The existing Pond 1A Diversion Channel will divert the stormwater runoff away from the protective cap by routing around the western perimeter of the pond. This diversion channel conveys flow to the south side of Pond 1 at VA Rt. 665 and into the Clinch River.

The Pond 1B Diversion Channel will divert the stormwater runoff away from the protective cap by routing around the eastern perimeter of the pond. At the end of this diversion channel, flow will be collected at a drop inlet structure and discharged into a proposed 48" HDPE pipe. This pipe will convey the flows through an existing tunnel under VA Rt. 616 and into Dumps Creek.

Rainfall falling directly onto the protective cap will be routed to a central collection channel at Pond 1A that intercepts the Pond 1A Diversion Channel above the concrete flume. This channel will be created by the final grading and will convey non-erosive flows. Runoff collected on the Pond 1B protective cap will flow to the Pond 1B Diversion Channel.

### **A.3 Inventory Removal and Disposal**

The Temporary Ash Storage area is located along the northern perimeter of Pond 1A and contains ash that was excavated from Pond 1A. The ash was placed at this location for use as fill during grading operations for the Pond 1 closure. The ash may be used to reach the grades shown in the PVC Liner Subgrade Plan as shown on the drawings. Once the stockpiled ash has been moved from the Temporary Ash Storage Area and placed to achieve subgrade throughout the Pond 1 closure limits, the Temporary Ash Storage Area will also be covered with both the PVC Liner and final cover soils as shown on the drawings.

None of the coal combustion residuals, which were historically sluiced to Ash Pond 1, will be removed prior to the closure activities.

### **A.4 Leachate/Seep Management**

The Reclaim Pond currently collects leachate from the Pond 1 toe seep drains. Leachate is conveyed from the Reclaim Pond to the Clinch River Plant's water treatment facility for treatment. Following closure of Pond 1, the Reclaim Pond will continue to collect and convey seep leachate from the pond.

### **A.5 Closure**

The general concept for site closure, the requirements for soil cover, vegetation, and post-closure maintenance are explained herein and are shown on the Pond Closure Drawings in Attachment A. Cover soil will be obtained from both on-site and off-site sources. The designed cap meets the requirements of the alternate final cover as noted in the demonstration of Attachment J.

#### **A.4.1 Cap Description.**

The cap system consists of:

- 30 mil PVC flexible membrane liner (FML), covered by
- A double-sided geocomposite drainage net (GDN), covered by
- 24-inch thick vegetated soil cover (or approved alternate).

#### **A.4.2 Engineering calculations and stability calculations of the final cover system are provided in Attachment B & C.**

#### **A.4.3 Maintenance Needs.**

The cover system is designed to function effectively with minimum maintenance needs. The top surface will be graded to provide positive drainage and to prevent ponding. The vegetative cover specified will be monitored closely, particularly in the establishment year and will be reseeded and mulched as necessary. The vegetation species were chosen so as not to require maintenance. Fertilizer-nutrient cycling and biological nitrogen fixation (by Birdsfoot trefoil) will maintain and build fertility levels. Woody plants will not be allowed to grow on the closed pond. The calculated soil loss from the soil erosion control layer was determined to be 0.28 tons per acre per year. These calculations are contained in

Attachment C of this document. Erosion control blankets will be used to control erosion while vegetation is establishing.

#### A.4.4 Construction Quality Assurance Plan.

This plan is included in Attachment D.

### **A.6 Schedule for Closure**

Closure is anticipated to be completed before September of 2018. Please refer to Section 7.0 of Attachment H for additional details pertaining to the pond closure schedule.

### **A.7 Posting**

Signs will be posted at the south access gate of the Pond 1 facility. These signs will indicate that the site is closed and unauthorized entrance is prohibited. No customer notification was required due to APCo being the only user of the facility.

### **A.8 Notification**

The land is already recorded with Russell County, Virginia. Upon closure, the Russell County Board of Supervisors in Lebanon, Virginia will be notified.

### **A.9 Certification**

The required certification by a registered professional engineer will be provided at the appropriate time following completion of closure activities.

### **A.10 Closure Cost Estimate**

An estimate of closure costs has been produced and is available in Attachment F.

### **A.11 Post Closure Cost Estimate**

An estimate of post closure costs has been produced and is available in a separate document titled: Post Closure Care Plan.

## ***B. Closure Calculations***

### **B.1 Cover System Stability and Liquids Management**

Attachment B & C contains calculations for cover system stability, peak storm run-off and volumes, surface water management facilities (ditches and pipes), and soil erosion during construction and throughout the post-closure period.

### **B.2 Settlement, Subsidence and Displacement**

Issues pertaining to settlement, consolidation, bearing capacity, liquefaction and stability for Ash Pond 1 are addressed in Attachment B. No adverse effects due to these issues are anticipated.

### **B.3 Freeze and Thaw Effects**

According to the U.S. Weather Bureau, the depth of maximum frost penetration for the Carbo area is approximately 15 inches. Since the FML used in the cap system will be covered with soils material, freeze/thaw cycles will not affect the integrity of the FML.

## ***C. Construction Specifications***

The site and cap system construction requirements/specifications are included in Attachment E.

## ***D. Groundwater***

Information regarding the groundwater is provided in a separate document titled: Groundwater Monitoring Plan Pond 1A/1B.

## Attachment B

# Geotechnical Analysis

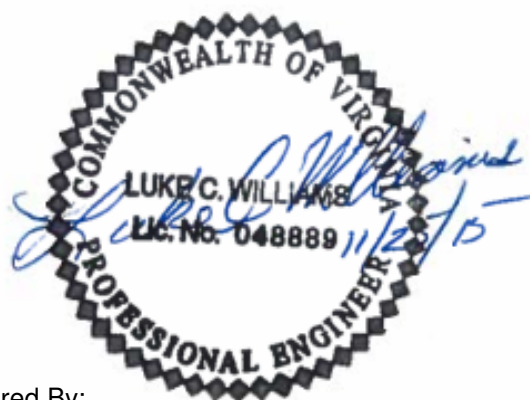


**Attachment B**  
**Revised Geotechnical Analysis**  
**AEP Clinch River Power Plant**  
**Pond 1 Closure**

Carbo, Virginia

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## TABLE OF CONTENTS

LIST OF TABLES.....	ii
LIST OF FIGURES .....	iii
1. INTRODUCTION .....	1
1a. Project Description.....	1
1b. Purpose of the Work .....	1
1c. Proposed Conditions.....	2
2. SUBSURFACE INFORMATION.....	2
2a. Available Geotechnical Data .....	2
2b. Current Subsurface Investigation .....	3
2c. Laboratory Testing .....	3
3. DESIGN CROSS-SECTIONS FOR ANALYSES .....	5
3a. Geometry and Soil Layers.....	5
3b. Soil Parameters .....	10
3c. Groundwater Levels.....	12
4. ANALYSES AND RESULTS .....	14
4a. Seepage Analyses .....	14
4b. Slope Stability Analyses.....	15
4c. Liquefaction Potential.....	20
4d. Settlement Analyses .....	25
5. Conclusions .....	26

## APPENDICES

APPENDIX 1 Boring & Stability Section Plan; Cross Section A-A; Cross Section B-B; Cross Section C-C; Cross Section D-D

APPENDIX 2 Drill Logs

APPENDIX 3 In-Situ Testing

APPENDIX 4 Laboratory Testing Results

APPENDIX 5 Seepage, Stability, and Settlement Results; SEEP/W Material Parameters

APPENDIX 6 Cover Slope Stability; Anchor Trench Design

**LIST OF TABLES**

Table 1: Permeability Results with Depth for Borings 13B-01, 13B-02, and 13B-03 ..... 4

Table 2: CU Results with Depth for Borings 13B-01, 13B-02, and 13B-03 ..... 4

Table 3: Hydraulic conductivity values obtained from the current subsurface exploration.....11

Table 4: Predicted seepage rates compared to the findings from the previous study. ....15

Table 5: Summary of stability factors of safety for each loading condition and location. ....16

Table 7: Relative Density .....23

Table 8: Modulus of volume compressibility. ....25

Table 9: Settlement Calculation Results.....26

## LIST OF FIGURES

Figure 1: Idealized Section A-A with Model Properties .....	6
Figure 2: Idealized Section B-B with Model Properties .....	7
Figure 3: Idealized Section C-C with Model Properties.....	8
Figure 4: Idealized Section D-D with Model Properties.....	9
Figure 5: Strength value results (drained and undrained) showing the average values and the values used in the analyses. ....	11
Figure 6: Groundwater levels as recorded at each piezometer location along the base of the embankment, shown by date.....	12
Figure 7: Groundwater levels as recorded at each piezometer location along the crest of the embankment, shown by date.....	13
Figure 8: Groundwater levels as recorded at each piezometer location along the existing hillside, shown by date. ....	14
Figure 9: USGS Hazard Mapping results for the project location.....	18
Figure 10: Determination of the $PGA_{adjusted}$ value from Idriss (1990).....	19
Figure 11: Liquefaction resistance of materials encountered in borings SPCT-01, SPCT-02, and SPCT-03 compared to published results from Andrus & Stokoe, 1999.....	21
Figure 12: Shear wave velocities of materials encountered in borings SPCT-01, SPCT-02, and SPCT-03 compared to published results from Bachus and Santamarina (2012). ....	24

## 1. INTRODUCTION

### 1a. Project Description

This Report of Geotechnical Analysis presents our assessment of the post-closure stability of Pond 1 at the Clinch River Power Station. This report utilizes previous geotechnical data and the recent data obtained during our exploration in July 2013. With the design of an impervious cap, the impounding capabilities of the pond will be eliminated, thus making the previous stability analyses performed by AEP more critical (lower FS) than this study.

Pond 1 at the Clinch River Power Station was constructed between 1955 and 1956 to the east of the plant and near to the confluence of Dump Creek and the Clinch River. The main dike that forms Pond 1 was raised several times between 1955 and 1971. The current crest elevation of the main dike is approximately 1,570 ft. The original 1955 dike was made of relatively impervious silty clay soil with a mixture of shale and sandstone fragments to a crest elevation of 1,540 ft. A mixture of fly ash and bottom ash material was used in the first raising to a crest elevation of 1,550 ft. Shale rock fill was used in subsequent raisings to the current crest elevation of 1,570 ft. The second and third dikes first were constructed above and behind the original dike following the upstream construction method. The last raising was constructed on the downstream side of the previous dike raisings with a downstream slope graded to 2 horizontal to 1 vertical. A cement bentonite fly ash cutoff wall 65 feet deep was installed throughout the crest in the winter of 1991 to address a high phreatic surface at that time.

Pond 1 is separated by a splitter dike as shown on Drawing 1, "Boring & Stability Section Plan" (Appendix 1), with the separation defining Pond 1A and Pond 1B. Bottom ash material produced at the plant is sluiced into Pond 1A with occasional fly ash where it is deposited by settlement as it traverses a serpentine channel to Pond 1B. The clarified water is then discharged through a riser and reused at the plant. The operating pool levels at Pond 1A and Pond 1B are currently 1,565 feet and 1,558 feet, respectively.

### 1b. Purpose of the Work

The scope of work for this analysis was to analyze the stability of Ponds 1A and 1B, considering the effects of the pond closure on the long-term geotechnical stability. Previous studies have examined the current conditions of the embankment including the in-place material properties of the embankment material. These previous studies utilized material properties of the sluiced material gathered from other sites. Therefore, Amec Foster Wheeler's geotechnical exploration focused on obtaining material properties of the in-place sluiced material at the Clinch River facility. The geotechnical analyses in this report summarize the liquefaction potential, seepage predictions, stability, and settlement of the cross-sections using the previously defined embankment geometry and soil properties. Steady-state seepage analyses were conducted to determine the pore water pressure distribution throughout the cross-sections. The stability analyses were conducted using these pore water pressures at four cross sections under static and pseudo-static loadings. Deep failure slip circles as well as shallow failure slip circles were

examined. The predicted phreatic surface was determined for each section, as well as the expected factor of safety based upon our analyses. Using these results, conclusions were made about the overall stability of this structure upon closure.

Amec Foster Wheeler's judgment of adequate safety factors were based on the Army Corps of Engineers (ACOE) document EM-1110-2-1902 (2003) and from Hynes-Griffin and Franklin (ACOE-1984). The ACOE specifies for new dam construction a minimum safety factor (FS) of 1.5 with respect to static loading. The ACOE indicate that this value may be lowered for existing structures where a performance history has been obtained. In this instance, we have used the FS=1.5 value on the existing Clinch River structure for conservative and comparative purposes. For pseudo-static loadings, the Hynes-Griffin and Franklin study recommends a pseudo-static safety factor of 1.0, in which earth dams would not be subjected to "dangerously large" earthquake deformations.

### **1c. Proposed Conditions**

The proposed pond closure design will utilize cut and fill methods with existing coal ash and approved borrow materials placed as structural fill to achieve a grade sloping away from the hillside to allow surface water to drain from the 21 acre site. The dikes will not be altered except to be lowered in some instances. Maximum cut and fill depths, relative to the existing ash surface, will be approximately 8 and 13 feet, respectively, to achieve the final subgrade elevations. The completed work will then be capped with a 30-mil PVC FML flexible membrane liner, double-sided geocomposite drainage net, and 24 inches of cover soil. Surface water will be directed via sheet and channel flow to the west (Pond 1A) and the east (Pond 1B), where it will ultimately be discharged to the Clinch River and Dumps Creek, respectively, as shown in Drawing 1.

## **2. SUBSURFACE INFORMATION**

### **2a. Available Geotechnical Data**

In 2010, AEP conducted extensive seepage and stability analyses for the active phase of Ponds 1A and 1B and documented them in a report titled *CLINCH RIVER POWER PLANT BOTTOM ASH DISPOSAL FACILITY - Pond 1 Stability Analysis*, December 2010. In that report, the construction and operation of Pond 1 was reviewed and much of the previous drilling and material strength properties for Pond 1 were consolidated. As part of the 2010 investigation, six additional borings and piezometers were installed through the embankment; and with the construction documents and site topographic maps, the report provided cross sections of the embankment and their material properties. In the 2010 report, Sections A-A, B-B, and C-C were developed through the embankment with a crest elevation of 1,570 feet. AEP analyzed static and pseudo-static limit equilibrium slope stability and seepage along these three sections and concluded that they were acceptable.

Amec Foster Wheeler examined the 2010 AEP report and utilized the information for the embankment in this 2013 study. Generally, the 2010 report determined that the stability of the embankment was satisfactory with a crest at 1,570 feet and a constant pool of water at

approximately elevation 1,560 feet behind the embankment for both static and pseudo-static conditions. Because of the extensive efforts to define the embankment by AEP, Amec Foster Wheeler's investigation concentrated on the characteristics of the sluiced ash behind the embankment.

## **2b. Current Subsurface Investigation**

As part of this geotechnical investigation, field work was performed at three locations upstream of the embankment and along the previously defined cross sections A-A and B-B. The exploration locations are shown on Drawing 1. At each location, the auger borings were advanced and undisturbed samples were collected using both GUS samplers and piston samplers. S&ME, as a subcontractor to Amec Foster Wheeler, conducted the drilling operations from July 22, 2013 to July 30, 2013, with Amec Foster Wheeler personnel logging the borings and coordinating the work. The drilling program was designed to obtain a relatively complete section at the drill locations through the sluiced ash and successfully terminated in native materials at depths ranging from about 65 feet to about 80 feet.

All borings were performed with a truck mounted drill rig and were advanced between sampling events using 3-1/4 inch I.D. hollow stem augers. Relatively undisturbed samples were obtained with a GUS or Piston Sampler at approximately 5 foot intervals according to ASTM D 6519. Once these samples were retrieved from the bore hole, Amec Foster Wheeler measured the recovery and weighed the sample to determine the in-place density of the material. We then sealed the tube according to ASTM D 1587 and returned the tube to our laboratory in Abingdon, Virginia. Vane shear testing (ASTM D 2573-08) or SPT sampling (ASTM D 1586) was performed following the relatively undisturbed sampling. During the drilling, Conetec, as a subcontractor to Amec Foster Wheeler, performed vane shear testing between the collections of undisturbed samples at 13B-01. At subsequent holes, SPT samples were collected. Drill logs are provided in Appendix 2.

Amec Foster Wheeler subcontracted with Conetec to perform CPTu testing, SCPTu testing, downhole dilatometer testing, and at selected holes and intervals, vane shear testing and pore pressure dissipation measurements. The data from this in-situ testing is provided in Appendix 3. In addition, the drill log and CPT data from Amec Foster Wheeler's data are presented graphically on Drawings 2 and 3 (Appendix 1), representing cross-sections A-A and B-B, respectively.

## **2c. Laboratory Testing**

The soil/ash samples were analyzed in the laboratory under consolidation, permeability, and CU testing. The permeability and strength parameters were of interest in this study and are shown in Table 1. In this table, the permeability and strength parameters are displayed according to depth. While both of these parameters are shown to vary with depth, it should be noted that there is no discernible trend of the strength values associated with increasing depth.

The permeability testing yielded values of hydraulic conductivity that varied from a low of 5.0E-05 cm/sec to a high of 1.1E-03 cm/sec, with an average value of 3.94E-04 cm/sec for the 4 samples

analyzed (Table 1). The consolidated undrained testing provided effective stress friction angle values from 23.3° to 41.2°, with an average of 32.1°. Effective cohesion values ranged from 0 psi to 8.4 psi, with an average of 2.7 psi. The total stress friction angle values ranged from 8.2° to 25.7°, with an average of 16.1°. The total cohesion values ranged from 0 psi to 11.4 psi, with an average of 4.2 psi. (Table 2)

**Table 1: Permeability Results with Depth for Borings 13B-01, 13B-02, and 13B-03**

	Depth (ft)	Boring 13B-01	Boring 13B-02	Boring 13B-03
Permeability Testing	16.5'			k = 5.0E-05 cm/sec = 0.0059 ft/hr
	20.0'			k = 1.1E-03 cm/sec = 0.1300 ft/hr
	22.9'		k = 3.7E-05 cm/sec = 0.0044 ft/hr	
	40.0'			k = 3.9E-04 cm/sec = 0.0460 ft/hr

**Table 2: CU Results with Depth for Borings 13B-01, 13B-02, and 13B-03**

	Depth (ft)	Boring 13B-01		Boring 13B-02		Boring 13B-03	
		Effective	Total	Effective	Total	Effective	Total
CU Testing	5.0'	$\Phi'=32.5^\circ$ c'=3.4psi	$\Phi=25.3^\circ$ c=6.6psi				
	17.4'			$\Phi'=26.7^\circ$ c'=2.1psi	$\Phi=12.4^\circ$ c=0.0psi		
	20.3'					$\Phi'=32.5^\circ$ c'=3.7psi	$\Phi=9.0^\circ$ c=8.2psi
	21.5'	$\Phi'=32.0^\circ$ c'=1.3psi	$\Phi=25.7^\circ$ c=0.4psi				
	30.0'	$\Phi'=29.4^\circ$ c'=2.3psi	$\Phi=17.0^\circ$ c=0.0psi				
	40.0'					$\Phi'=41.2^\circ$ c'=0.0psi	$\Phi=23.0^\circ$ c=1.2psi
	47.3'			$\Phi'=23.3^\circ$ c'=8.4psi	$\Phi=8.2^\circ$ c=11.4psi		
	49.6'					$\Phi'=37.1^\circ$ c'=2.0psi	$\Phi=13.0^\circ$ c=3.2psi
	60.2'			$\Phi'=34.4^\circ$ c'=2.4psi	$\Phi=9.8^\circ$ c=6.7psi		
	65.4'					$\Phi'=31.5^\circ$ c'=1.4psi	$\Phi=17.5^\circ$ c=4.1psi



### 3. DESIGN CROSS-SECTIONS FOR ANALYSES

#### 3a. Geometry and Soil Layers

The geometry and material properties used in Amec Foster Wheeler's models for this 2013 study are based on AEP's 2010 Stability Report for the same site. In that report, the geometry was defined based on 2006 aerial topography, from AEP's drawing 13-10585-3107-4, and from the 2009 boring logs. The shear strength parameters and hydraulic conductivities for each zone were determined through AEP's field and laboratory testing conducted at that time. Three sections were defined in the 2010 study; and following Amec Foster Wheeler's review were deemed applicable to represent the conditions of the impoundment. These sections are denoted as Sections A-A, B-B, and C-C. Their locations, as well as a newly defined Section D-D, are shown on Drawing 1, and their geometries and soil properties are shown in Figures 1 through 4 for Sections A-A through D-D, respectively.

Amec Foster Wheeler obtained the existing surface profile at these sections from updated mapping (March 2012), and modified it to reflect the proposed final grades for the pond closure. We also obtained the 2010 GeoSlope files from AEP with the embankment soil and material property information and applied it to our model for this study. The soil layers for the embankments were not modified, but the impounded ash layers were added based on our drilling and testing and extended to the natural ground north of the pond.

Section D-D is a new cross-section that was not defined previously in AEP's 2010 Stability Report. For this section, Amec Foster Wheeler defined its geometry and soil layers based upon information obtained from boring MW0915A and the layout of the closest existing cross-section, Section A-A. Figure 4 shows the geometry and soil layers used for this section. It should be noted that the depth to bedrock for this section is less than in Sections A-A through C-C shown in Figures 1 through 3, respectively.

Figure 1: Idealized Section A-A with Model Properties

Soil Type	$\gamma_{sat}$ (pcf)	Static & Seismic Drained		Seismic Undrained		Saturated k (ft/hr)
		$\Phi'$ (deg)	C' (psf)	$\Phi$ (deg)	C (psf)	
1- Clayey silty sand and gravel	136	36	0	36	0	0.05
2- Silty sand	136	36	0	31	1006	5.0E-4
3- Compacted ash	100	30	0	30	0	0.069
4- Clay w/ gravel	134	25	700	12	1810	3.5E-6
5- Clayey silty sand w/ gravel	135	34	0	29	580	0.07
6- Sluiced ash	97	32	0	16	600	0.069
7- Sandy lean clay	140	32	850	21	1270	3.5E-6
8 -Clayey silt	98	39	0	20	400	1.58E-5
9- Silty sand	91.5	31	700	15	1800	5.8E-3
10- Sandy silt	100	28	0	20	400	2.13E-5
11- Lean clay w/ sand	88	30	0	18	200	3.46E-6
12- Weathered shale	140	35	1000	35	1000	2.1E-5
13- Bedrock						0.001
14- Cutoff wall	150	0	1200	0	1200	6.0E-4
15- Protective cover	105	30	0	30	0	0.069

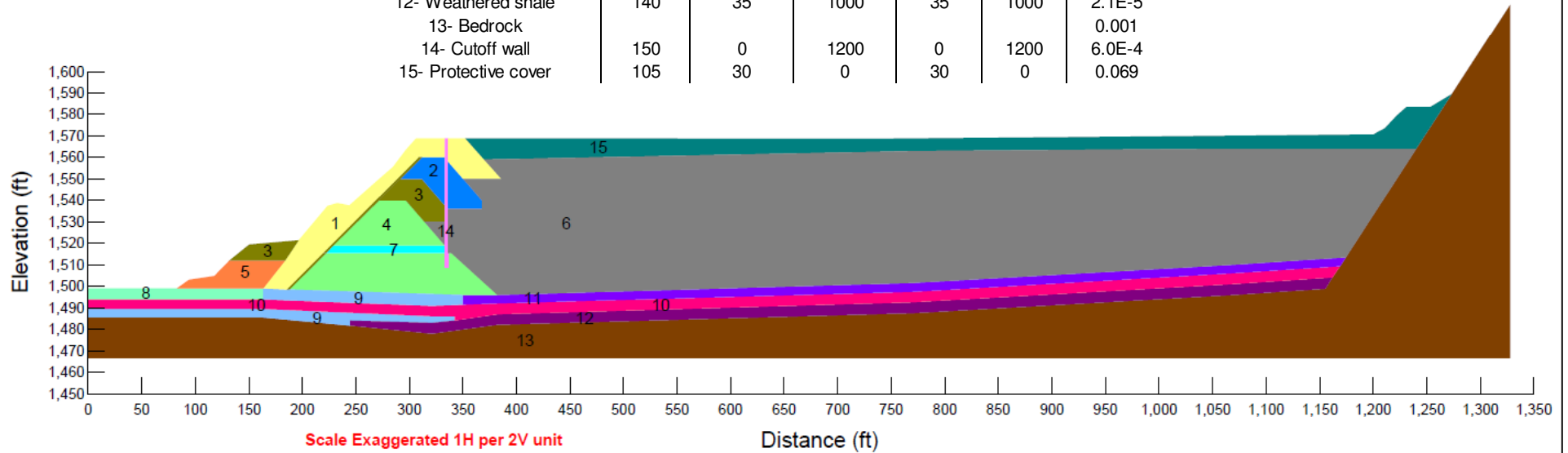


Figure 2: Idealized Section B-B with Model Properties

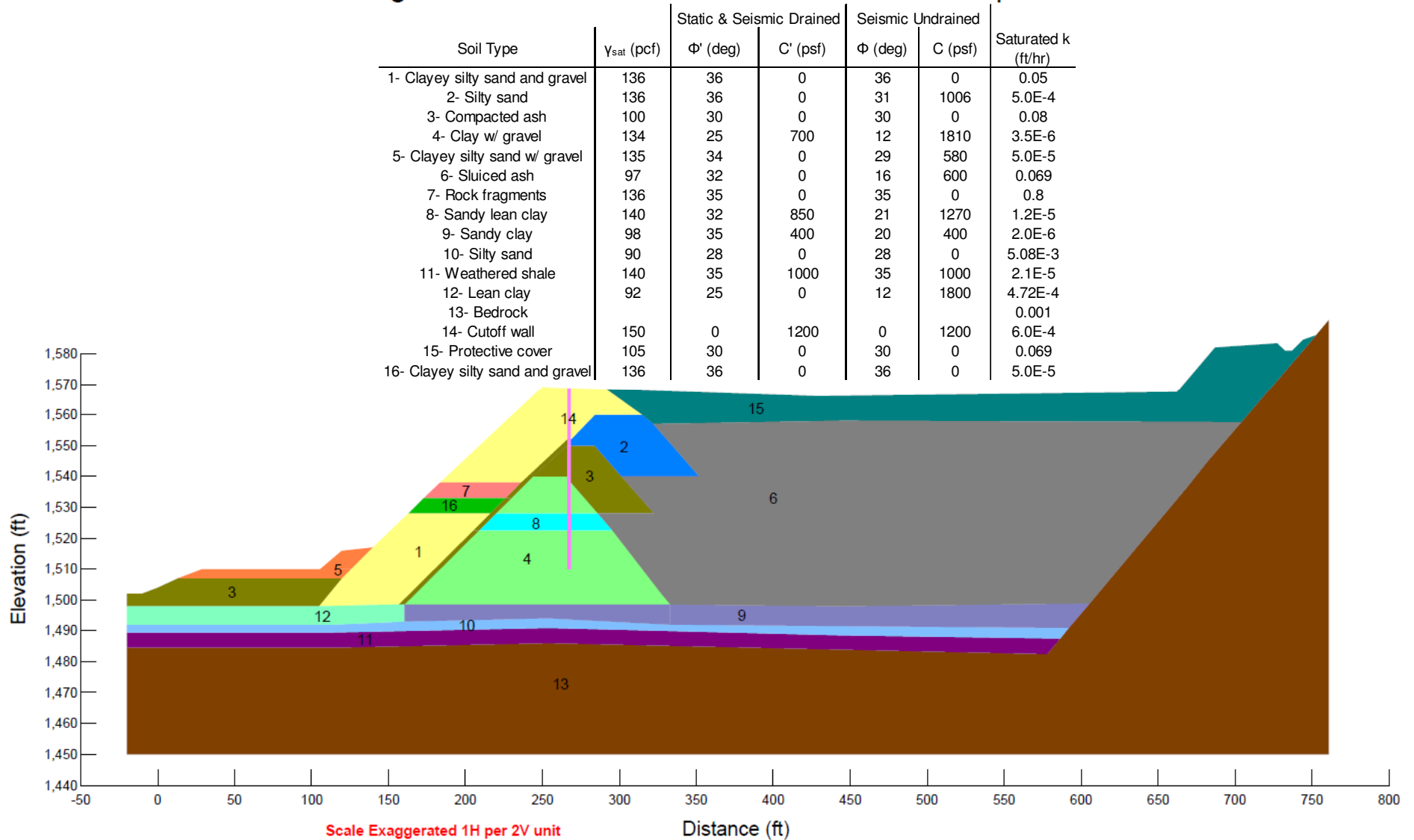


Figure 3: Idealized Section C-C with Model Properties

Soil Type	$\gamma_{sat}$ (pcf)	Static & Seismic Drained		Seismic Undrained		Saturated k (ft/hr)
		$\Phi'$ (deg)	$C'$ (psf)	$\Phi$ (deg)	$C$ (psf)	
1- Clayey silty sand and gravel	136	36	0	36	0	0.05
2- Silty sand	136	36	0	31	1006	5.0E-4
3- Compacted ash	100	30	0	30	0	0.08
4- Clay w/ gravel	134	25	700	12	1810	3.5E-6
5- Clayey silty sand w/ gravel	135	34	0	29	580	5.0E-5
6- Sluiced ash	97	32	0	16	600	0.069
7- Clayey sand	97	36	100	22	900	6.08E-6
8- Silty sand	90	18	400	6	400	0.005
9- Silty clay	110	36	100	22	900	3.26E-6
10- Weathered shale	140	35	1000	35	1000	2.1E-5
11- Bedrock						0.001
12- Cutoff wall	150	0	1200	0	1200	6.0E-4
13- Protective cover	105	30	0	30	0	0.069

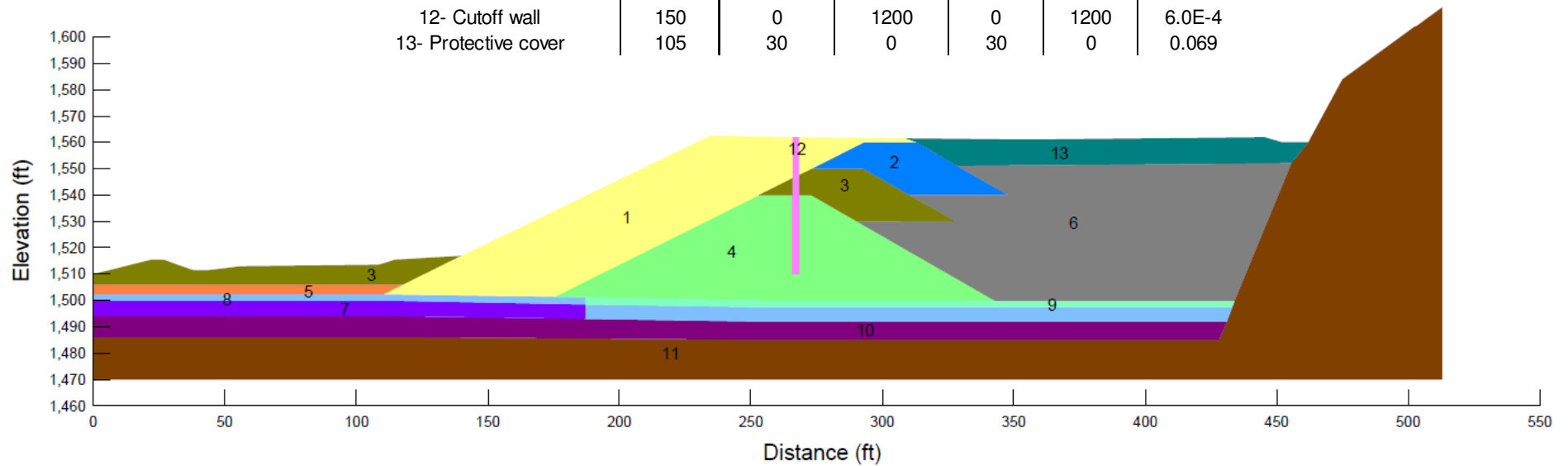
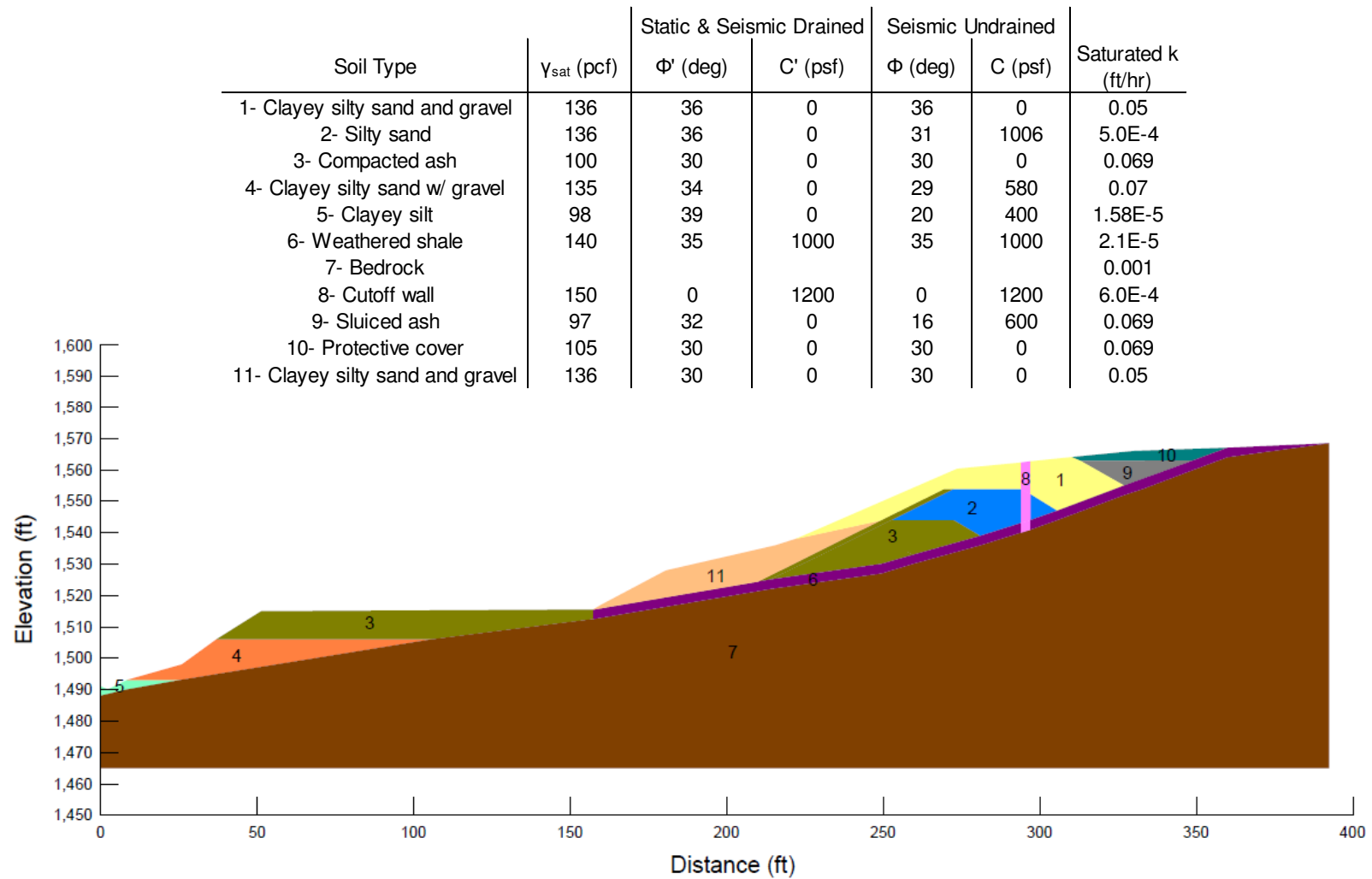


Figure 4: Idealized Section D-D with Model Properties



### **3b. Soil Parameters**

#### *Strength Parameters*

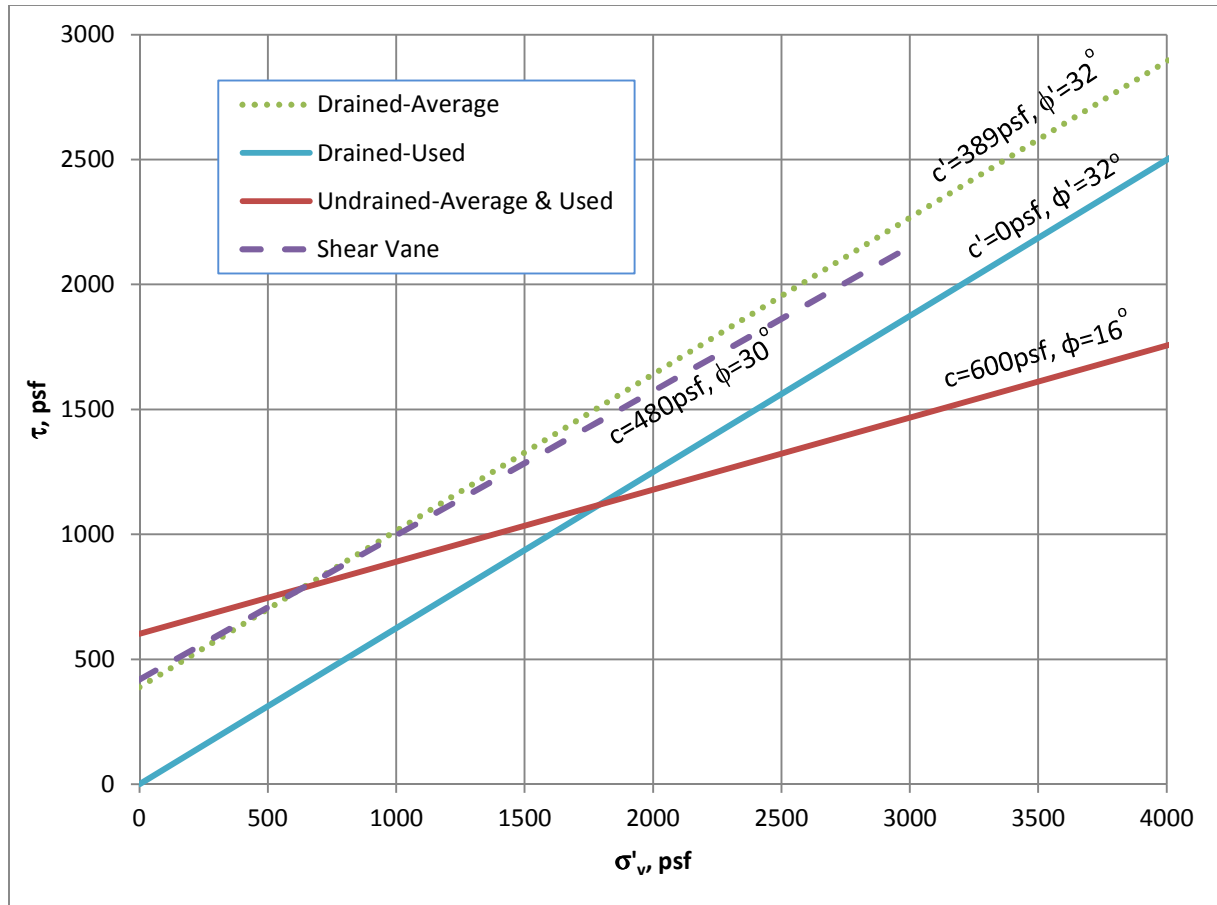
The shear strength parameters used in our analyses are also shown on Figures 1 through 4. For these plans, no additional drilling was performed on the embankment and the shear strength and hydraulic conductivity values for the embankment materials were taken from AEP's 2010 study.

The drained and undrained strengths of the impounded ash were determined from the recent exploration and CU w/ pore pressure measurement testing on representative relatively undisturbed samples and in-situ vane shear methods. The results from these analyses are shown in Figure 5.

As discussed in Section 2c, the minimum value of  $\Phi'$  was determined to be  $23.3^\circ$ , with average values of  $32.1^\circ$  and 2.7 psi (389 psf). The average effective shear strength failure envelope is shown in Figure 5. For this analysis, a slightly lower failure envelope was used for drained conditions  $c' = 0$  psf.

The undrained shear strengths of the fly ash were analyzed using CU w/ pore pressure testing and with in-situ vane shear testing. The results from both of these methods are shown in Figure 5. The average undrained shear strength from the CU w/ pore pressure laboratory testing resulted in an average  $\Phi$  of  $16^\circ$  and  $c$  of 600 psf. These values are significantly lower than the in-situ vane shear measurements ( $\Phi$  of  $30^\circ$  and  $c$  of 480 psf) and, the more conservative average values from the CU w/ pore pressure testing were used.

Finally, the protective cover material used for the pond closure was assumed. This material was defined with a  $\phi$  angle of  $30^\circ$  and cohesion of 0 psf. The stability analysis did not include the strength of the PVC lining material; however, a soil – fabric interface analysis is provided in the appendices.



**Figure 5: Strength value results (drained and undrained) showing the average values and the values used in the analyses.**

### Seepage analysis Parameters

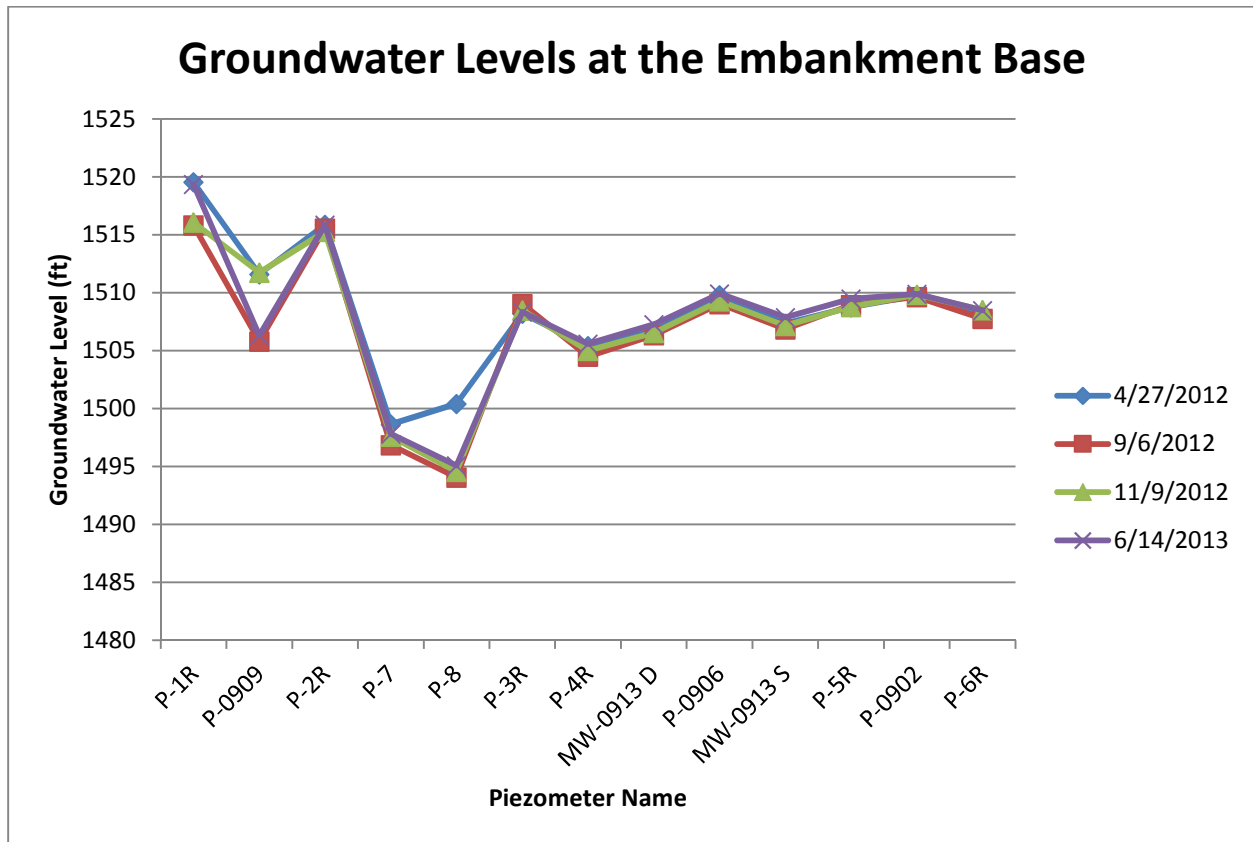
The permeability test results for the ash are included in Table 3. The values are shown to vary with depth and location, but an average for boring 13B-03 was calculated as 0.061 ft/hr. This value is in agreement with the previous conductivity value of 0.069 ft/hr used for the impounded ash layer in AEP's 2010 report. Therefore, the analysis was performed using the previous value of 0.069 ft/hr for the impounded ash.

**Table 3: Hydraulic conductivity values obtained from the current subsurface exploration.**

Depth (ft)	Hydraulic Conductivity (ft/hr)	
	13B-01	13B-02
16.5		0.006
21-23	0.004	0.130
41		0.046
<b>Average</b>		<b>0.061</b>

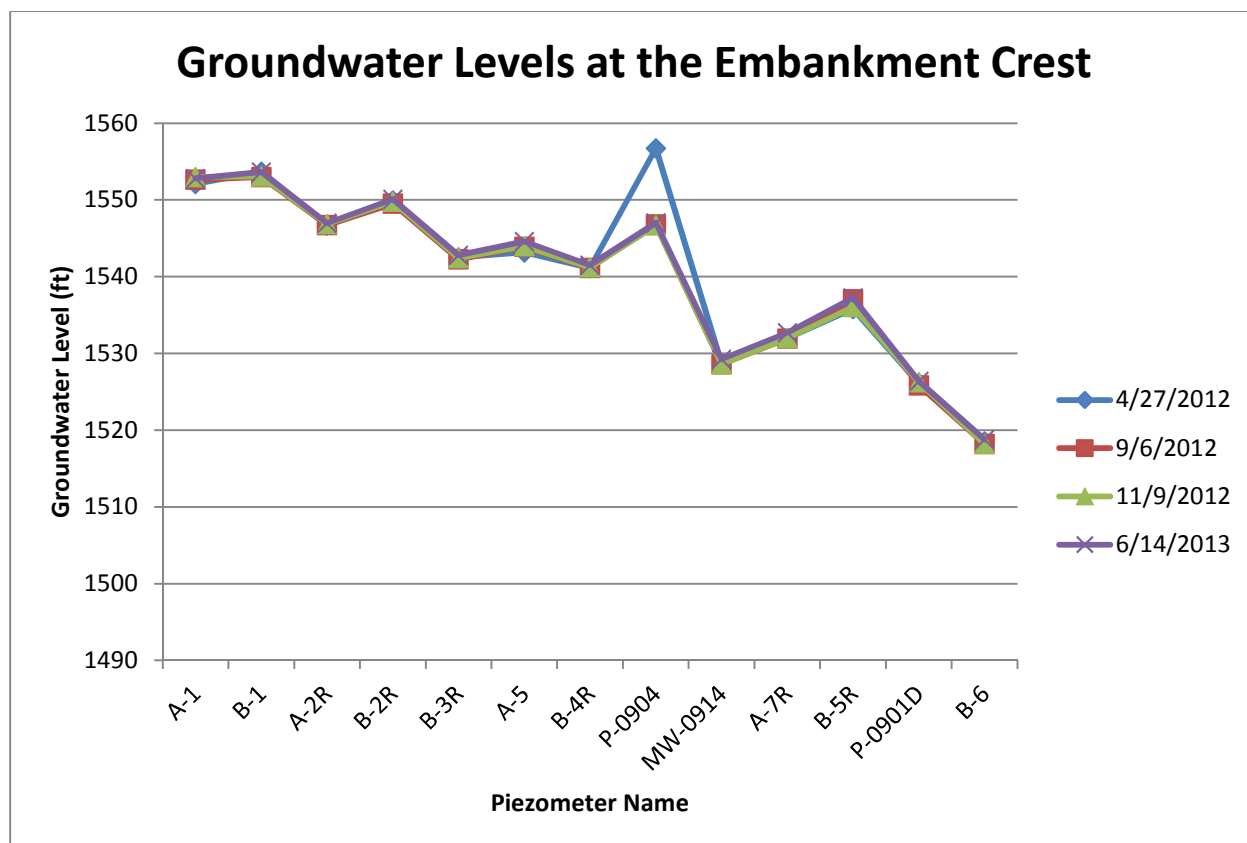
### 3c. Groundwater Levels

AEP regularly monitors groundwater wells and piezometers in and around the embankment. The historical ground water levels, indicating the active phase of disposal phreatic surface around the embankment and pool are provided in Figures 6-8 below. These graphs illustrate the ground water levels at the toe, crest and back of the impoundment, respectively. Also the wells are shown from west to east.

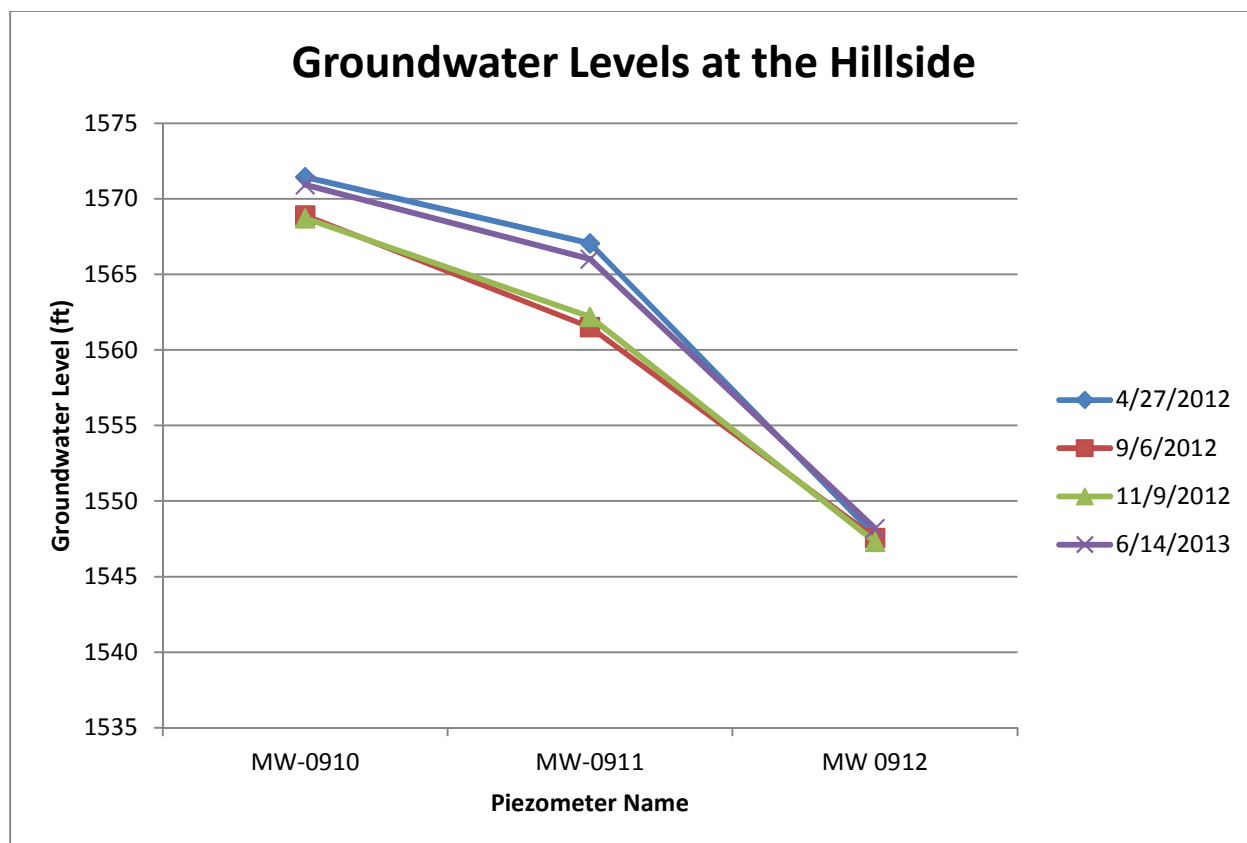


**Figure 6: Groundwater levels as recorded at each piezometer location along the base of the embankment, shown by date.**





**Figure 7: Groundwater levels as recorded at each piezometer location along the crest of the embankment, shown by date.**



**Figure 8: Groundwater levels as recorded at each piezometer location along the existing hillside, shown by date.**

## 4. ANALYSES AND RESULTS

### 4a. Seepage Analyses

Geo-Studio's SEEP/W software program was used to analyze the groundwater flow through the defined reclaimed impoundment and embankment. This program provides an estimation of the phreatic surface, to be used in the following stability analyses. The analyses were performed using the steady-state model, in which the flow into the system (through the hillside) is equal to the flow out of the system (through the toe of the embankment and/or seepage at the ground surface). To conduct a steady-state seepage analysis, the hydraulic conductivity function vs. soil suction was defined based upon material type. The SEEP/W software has methods that predict these functions, and these parameters are included in the SEEP/W output files shown in Appendix 5.

For this analysis, Amec Foster Wheeler assumed current groundwater levels at the base of the hillside, monitoring wells MW0910 and MW0911. Groundwater levels for Sections C-C and D-D were assumed based on the results from the previous sections. These piezometers show a water

level that are influenced by the existing pool and are therefore conservative for post closure analyses.

Appendix 5 includes the graphical results obtained from the seepage and stability analyses, as will be discussed throughout the remainder of the report. The results of the seepage analyses for Sections A-A through D-D are shown in Figures 5.1 through 5.4, respectively, and show the calculated phreatic surface. As stated previously, the seepage analyses were performed based upon a groundwater level from the hillside immediately to the north of the pond. Therefore, on the sections boundary conditions were applied on the right side of the model to match the groundwater levels recorded at the monitoring wells. Boundary conditions on the downstream portion of the slope were also defined as a seepage face to allow the model to choose the downstream heads.

The predicted seepage rates for each cross-section were also calculated and are shown in Table 4. In this table, the seepage rates are compared to those calculated in the 2010 AEP stability report. The current analyses show slight deviations from the original values, with Section B-B showing increased flow and Sections A-A and C-C showing decreased flow. Consequently, it is predicted that the existing seepage control appurtenances will continue to manage the anticipated seepage adequately.

**Table 4: Predicted seepage rates compared to the findings from the previous study.**

	Predicted Seepage Rates (ft <sup>3</sup> /hr)	
	AEP	Amec Foster Wheeler
Section A-A	0.078	0.053
Section B-B	0.032	0.044
Section C-C	0.046	0.032
Section D-D		0.020

#### **4b. Slope Stability Analyses**

The slope stability of each cross-section under varying loading conditions and material properties was conducted using Geo-Studio's SLOPE/W software program. SLOPE/W uses a 2-dimensional limit equilibrium formulation by analyzing the forces on a determined number of slices within a slip surface. The Morgenstern-Price method of analysis was used to determine the factors of safety, in which both force and moment equilibrium are satisfied using interslice shear and normal forces. This method of analysis is more rigorous than simpler methods (such as Bishop's, Janbu's, Army Corps of Engineers), because it satisfies both equations of equilibrium, while the simpler methods do not. A half-sine function was also used to specify the lambda values used in the formulation.

After selecting the analysis type, the pore water pressures and phreatic surface calculated by the SEEP/W software were loaded into and used for the SLOPE/W stability analyses for each section.

The results for each analysis are shown in Figures 5.5 through 5.19, with the computed factors of safety shown in Table 5.

**Table 5: Summary of stability factors of safety for each loading condition and location.**

Analysis	Computed FS
<b>Static Loading</b>	
<b>Section A-A</b>	
Deep Failure (Fig. 5.5)	1.84
Shallow Failure (Fig. 5.6)	1.51
<b>Section B-B</b>	
Deep Failure (Fig. 5.7)	1.62
Shallow Failure (Fig. 5.8)	1.64
<b>Section C-C</b>	
Deep Failure (Fig. 5.9)	1.65
Shallow Failure (Fig. 5.10)	1.64
<b>Section D-D</b>	
General Failure (Fig. 5.11)	1.57
<b>Pseudo-static Loading</b>	
<b>Section A-A</b>	
Drained Conditions (Fig. 5.12)	1.24
Undrained Conditions (Fig. 5.13)	1.26
<b>Section B-B</b>	
Drained Conditions (Fig. 5.14)	1.13
Undrained Conditions (Fig. 5.15)	1.24
<b>Section C-C</b>	
Drained Conditions (Fig. 5.16)	1.23
Undrained Conditions (Fig. 5.17)	1.12
<b>Section D-D</b>	
Drained Conditions (Fig. 5.18)	1.15
Undrained Conditions (Fig. 5.19)	1.15

As shown in Table 5, static loading of both deep and shallow surfaces were considered on each cross-section. Effective strength parameters were used in the static loading case, as the materials are assumed to be in a drained condition.

The entry and exit method was used to determine the slip surface location and geometry. The stability analysis results (Appendix 5) show the critical slip surface in as a white circular arc with its accompanying factor of safety. Also, a safety map is shown as a shaded red region. This region represents the region encompassing the slip surfaces with factors of safety within 0.05 of the critical slip surface and illustrates the extents of the critical surface search.

In addition to the static loading analyses, a pseudo-static analysis was performed according to the procedures outlined in the *Geotechnical Earthquake Engineering Handbook* (2002). The procedures followed based on these recommendations are presented below:

- a. Determine the Peak Ground Acceleration at the rock interface ( $PGA_{rock}$ ) for the project location using the USGS hazard mapping tool located at <http://www.geohazards.usgs.gov/hazardtool>, shown in Figure 9. The  $PGA_{rock}$  for the site was based upon a Class B/C rock boundary with an annual frequency of exceedance of 2% in 50 years. The  $PGA_{rock}$  value for the site is 0.145g.
- b. The  $PGA_{rock}$  at the Class B/C boundary was then used to estimate the value of the earthquake magnitude due to the further amplification through the overlying soils/embankment materials. This value is referred to as the adjusted PGA ( $PGA_{adjusted}$ ) and was determined using relationships developed from Idriss, I.M. (1990), "*Response of Soft Soil Sites During Earthquakes*." Figure 10 includes the chart used from this paper to develop  $PGA_{adjusted}$ . Using a  $PGA_{rock}$  value of 0.145g, the  $PGA_{adjusted}$  value was determined as 0.25g.
- c. The earthquake acceleration of the site, "a," is then determined based upon the earthquake record and dam geometry, in addition to other factors. For this analysis, "a" was determined using the following equation:  
$$a = 0.5 * PGA_{adjusted}$$

Therefore, a was determined as 0.125 g.
- d. The pseudo-static coefficient, "k," is then input into the Geo-slope program to model the effects of pseudo-static loading. The pseudo-static coefficient is represented by the following equation:  
$$k = a / g; k = 0.125g / g$$

Therefore, k is equal to the unitless value of 0.125.
- e. Use a minimum factor of safety of 1.0.

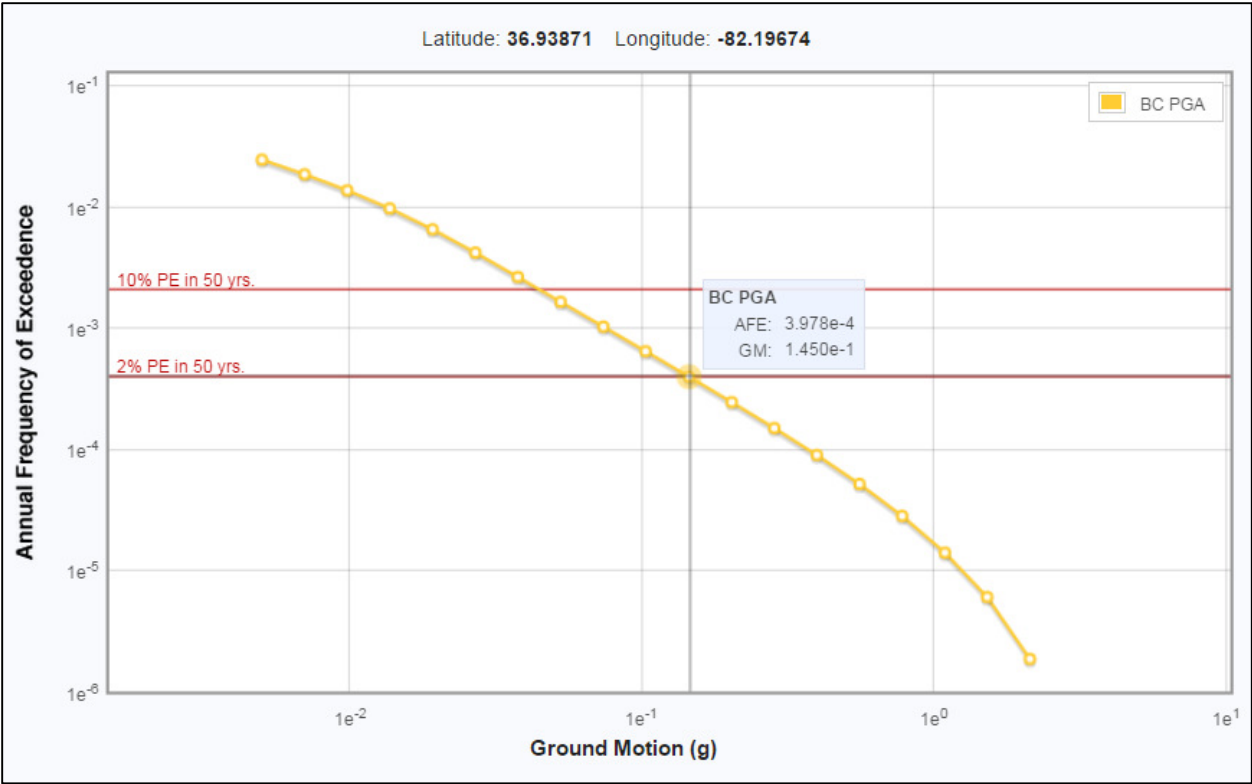
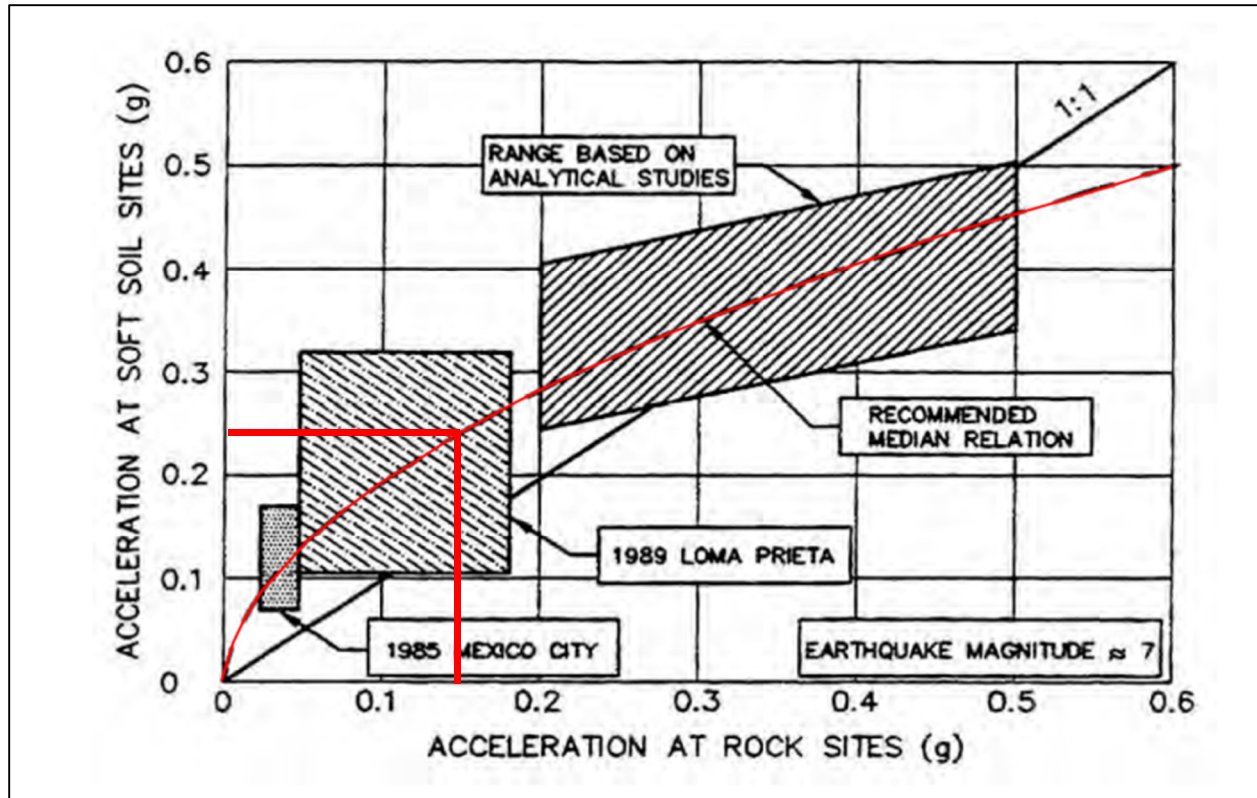


Figure 9: USGS Hazard Mapping results for the project location.



**Figure 10: Determination of the  $PGA_{adjusted}$  value from Idriss (1990).**

Most of the loading cases in the current stability analyses produced similar results (slip surfaces and factors of safety) to AEP's 2010 stability report. Because the circles were similar between the two studies and generally show no influence by the ash we have not analyzed the previously non-critical wedge failure analyses in this report. The extension of the sections, to include the impounded ash, yielded new failure surfaces through the impounded ash for the undrained seismic case at sections A-A and C-C (Figures 5.13, and 5.17). The factor of safety values were compared to the Army Corps of Engineers recommended value of  $FS=1.5$  for conservative and comparative purposes. For pseudo-static loadings, the Hynes-Griffin and Franklin study recommends a pseudo-static safety factor of 1.0, in which earth dams would not be subjected to "dangerously large" earthquake deformations. As shown in Table 5, the analyses results indicate factors of safety greater than the minimum required factor of safety for each case. Therefore, it can be concluded from this analysis that the stability of each cross-section is adequate.

However, these procedures should not be used where materials in either the embankment or foundation are susceptible to liquefaction under the design cyclic loading. Therefore, a liquefaction susceptibility analysis was performed and is presented in the following section.

#### 4c. Liquefaction Potential

The cyclic (dynamic) liquefaction potential of the hydraulically placed materials at the Clinch River Pond 1 was analyzed following the Simplified Procedure after Andrus and Stokoe (1999). This procedure uses the measured in-situ shear wave velocities to determine the liquefaction potential of materials subjected to a Magnitude 7.5 seismic event. Shear wave data at 1 meter intervals was obtained, at the three borings indicated as SCPT-01, SCPT-02, and SCPT-03.

In this procedure, the Cyclic Stress Ratio (CSR) is first computed according to the following equation:

$$CSR = 0.65 \left( \frac{a_{\max}}{g} \right) \left( \frac{\sigma_v}{\sigma'_v} \right) r_d$$

where  $a_{\max}$  is a peak ground acceleration,  $g$  is the acceleration due to gravity,  $\sigma_v$  is the vertical total stress,  $\sigma'_v$  is the vertical effective stress, and  $r_d$  is the shear stress reduction coefficient ( $1 - 0.015 \cdot \text{depth}$ ). The vertical stresses are based on the CPT data and corrected for unit weights obtained through the undisturbed sampling. For this analysis  $a_{\max} = 0.18 \text{ m/sec}^2$  determined from the USGS program Hazard Curve for PGA analysis a 2% exceedance in a 50 year exposure time was selected.

The shear wave velocity measurements taken in the field also have to be analyzed to correct for overburden stress. These calculations were performed according to the following equation:

$$V_{s1} = V_s \left( \frac{P_a}{\sigma'_v} \right)^{0.25}$$

where  $V_{s1}$  is the shear wave velocity corrected for overburden stress,  $V_s$  is the measured shear wave velocity, and  $P_a$  is the atmospheric pressure (around 100 kPa).

Once the CSR and  $V_{s1}$  have been calculated at each data point location, the results are plotted on a graph where CSR is the ordinate and  $V_{s1}$  is the abscissa, shown in Figure 11. A boundary line as defined in Andrus and Stokoe (1999) is also plotted on the graph. This line defines the boundary between a high liquefaction potential and a low liquefaction potential. The boundary is determined by using equations 2.13 and 4.3 with  $a = 0.022$ ,  $b = 2.8$ , and  $n = -2.56$  from Andrus and Stokoe (1999). When comparing the results from the shear wave velocity measurements obtained in the ash pond to this line, it can be seen that all values indicate a low risk for liquefaction. Because all the data points plot to the lower right of the liquefaction line, it can be concluded from this Simplified Procedure that the materials in the pond will not liquefy when subjected to a peak acceleration of  $0.18 \text{ m/sec}^2$  as a result of a Magnitude 7.5 seismic event.



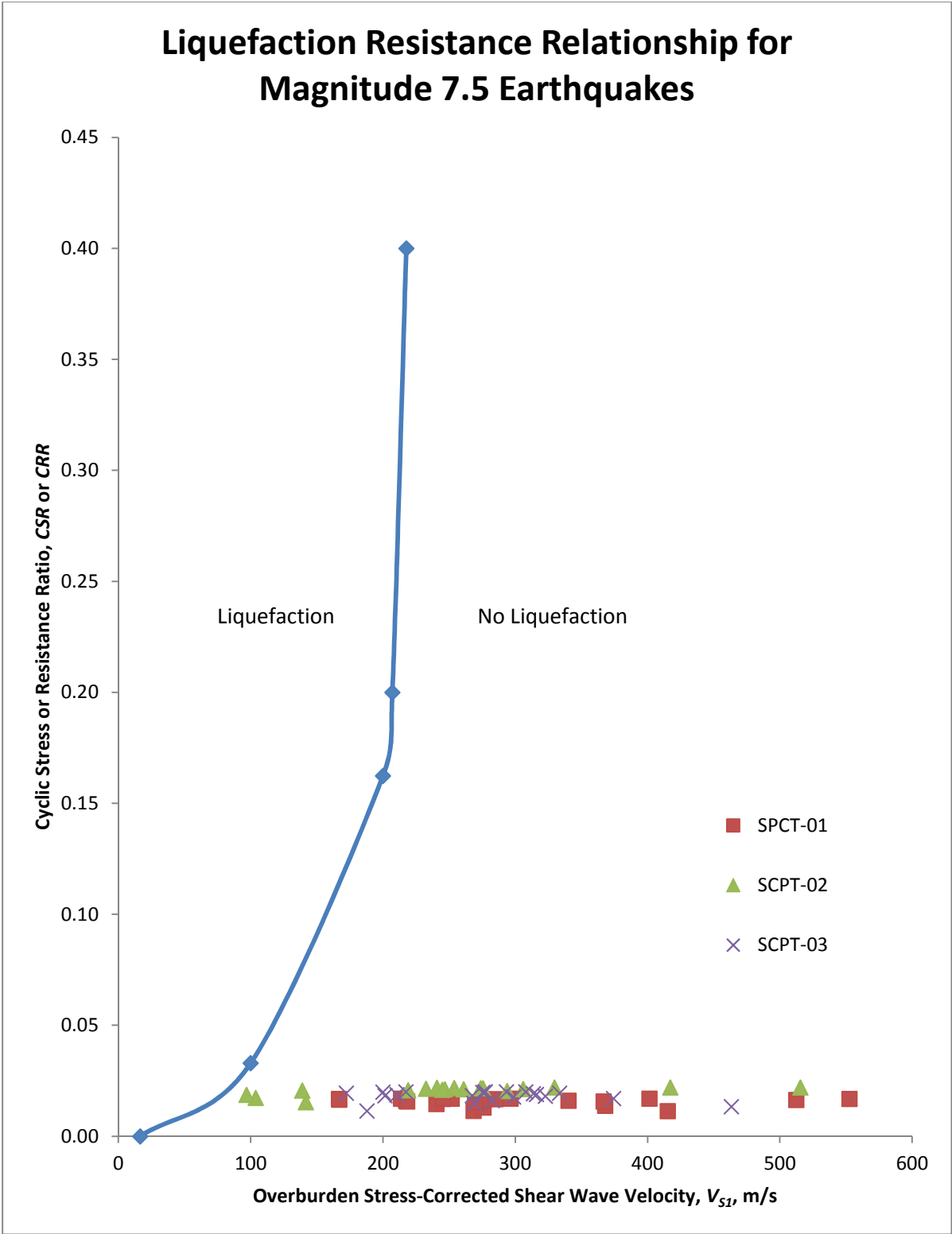


Figure 11: Liquefaction resistance of materials encountered in borings SPCT-01, SPCT-02, and SPCT-03 compared to published results from Andrus & Stokoe, 1999.

## Static Liquefaction

Static liquefaction is the undrained load response of a loose and **contractive** granular material that experiences positive pore pressure generation. A significant study was presented by the Electric Power Research Institute<sup>1</sup> (EPRI) which investigated the static liquefaction potential of coal ash. The study concluded that ....

“...the results of this study indicate that water-deposited fly ash is inherently anisotropic and it **generally** exhibits a dilative load-deformation behavior that hinders static liquefaction in ponds shallower than 20 to 25m.

The construction for the Clinch River Pond 1 Closure will not involve significant increase in fill heights that would potentially increase the pore pressures and induce static liquefaction. However, an analysis was still performed to analyze the potential for static liquefaction according to the following three areas as outlined in the EPRI report: (1) triaxial testing, (2) relative density, and (3) diagenic behavior.

### (1) Triaxial test

The triaxial results are provided in Appendix 4. As seen from the P-Q curves, the materials exhibit a dilative behavior. Exceptions to this occur in 13B-2: 17.4 to 23.7 feet of depth and 13B-3: 49.6 to 51.4 feet of depth. However, the samples generally showed little reduction in post-peak strength. The triaxial test observations indicate that the material is generally dilative and has a low potential for static liquefaction.

### (2) Relative density

The relative density results are summarized below in Table 7. In general, the results indicate that the soils sampled in our investigation are less than the desired 60% relative density. According to this metric, contractive behavior is possible at these horizons if the pore pressures are not adequately relieved.

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<sup>1</sup> *Geotechnical Properties of Fly Ash and Potential for Static Liquefaction: Volume 1 – Summary and Conclusions*. EPRI, Palo Alto, CA: 2012. 1023743.

**Table 7: Relative Density**

Sample	Depth (ft)	In-place unit wt (pcf)	Dry unit wt (pcf)	Minimum Density (pcf)	Relative Density %
<b>13B-01</b>					
GUS 2	0-11	95	59	45	0.57
GUS 5	18.5-23.5	96	58	53	0.28
GUS 7	28.5-35.5	96	60	48	0.54
<b>13B-02</b>					
GUS 2 and 3	15.5-20.5	92	51	50	0.05
PS 8	40.5-45.5	95	55	47	0.33
PS 10	57-63.5	85	47	35	0.35
<b>13B-03</b>					
PS2	18.5-23.5	96	51	50	0.03
PS 6	38.5-43.5	82	62	50	0.58
PS 8	48.5-53.5	87	53	26	0.61

(3) Diagenic behavior (Cementation).

Diagenic behavior will tend to inhibit dilative behavior and occurs when the pH of the ash is in the 9.0 to 10.0 range. All of the ash tested for pH was less than these values. Therefore, based upon the pH of the ash tested, it is not likely that the ash at this facility has undergone diagenesis and, thus, has a low potential for static liquefaction.

Diagenesis can also be predicted based upon shear wave velocity data and compared to threshold values defined by Bachus and Santamarina (2012). According to this evaluation, diagenesis is possible when the materials exhibit shear wave velocities higher than the threshold limits proposed by Bachus and Santamarina (2012). Figure 12 shows that the shear wave velocities of the materials encountered are higher than the threshold boundaries at certain locations throughout the ash. However, this is most likely due to the stiffening of the fly ash as it underwent consolidation in response to addition of further ash through the pond-filling process and subsequent expulsion of water through the pore spaces.

Based upon this analysis, the ash materials have a low potential for static liquefaction. The triaxial tests indicate that the material is generally dilative and not prone to static liquefaction. While the relative density results and shear wave velocities were inconclusive, the pH levels of the ash indicate a low potential diagenesis, which could inhibit static liquefaction as well.

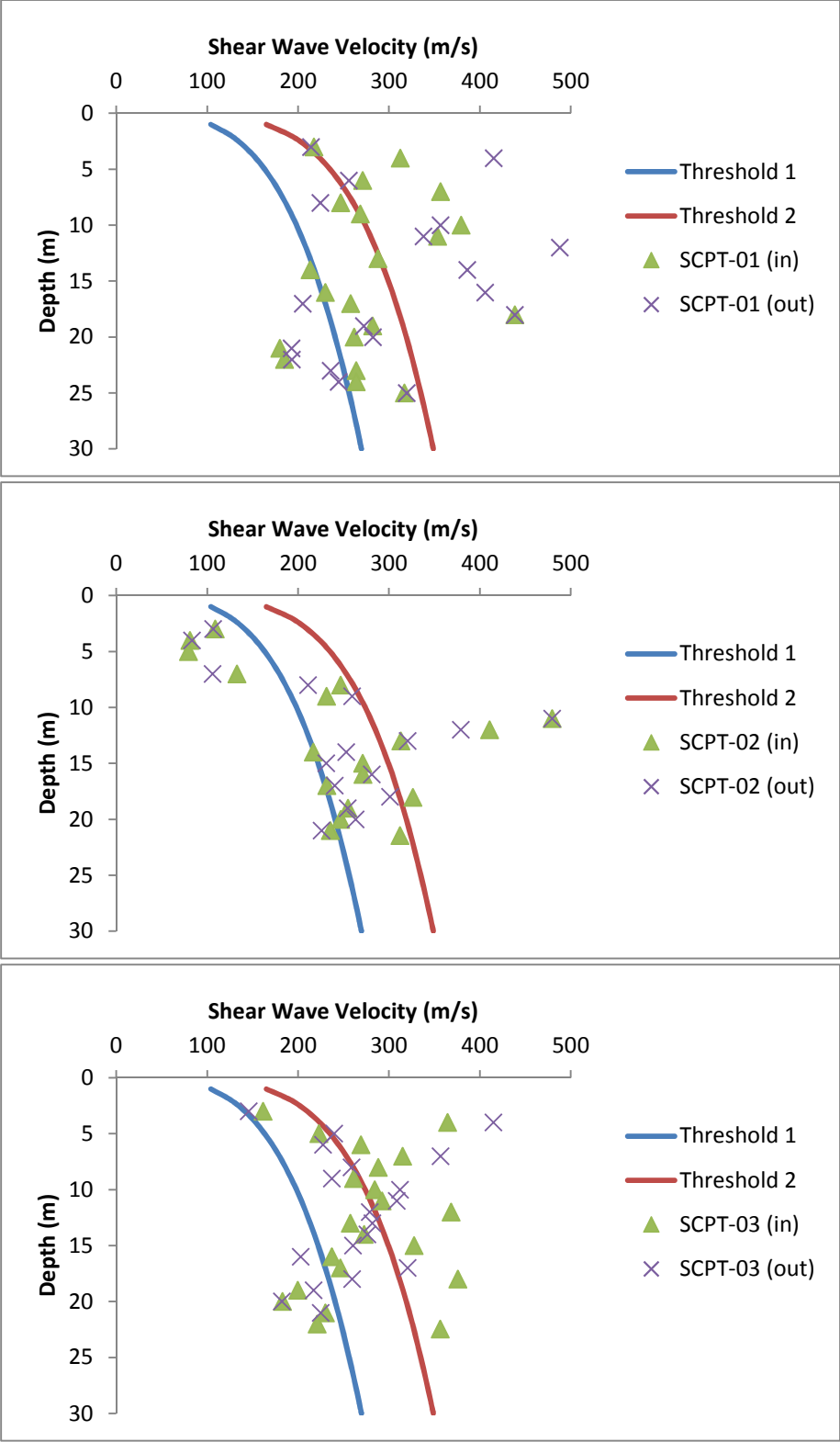


Figure 12: Shear wave velocities of materials encountered in borings SPCT-01, SPCT-02, and SPCT-03 compared to published results from Bachus and Santamarina (2012).

#### 4d. Settlement Analyses

Settlements were calculated at each boring location with the formula:

$$S = m_v \times t \times \Delta E'$$

Where:  $S$  = settlement

$m_v$  = modulus of compressibility

$t$  = thickness

$\Delta E'$  = change in effective stress

A total of eight consolidation tests were conducted at various depths in the impounded ash in borings 13B-01, 13B-02, and 13B-03. The consolidation test results (Strain vs. Log P) were used to determine the appropriate the modulus of volume compressibility ( $m_v$ ).

**Table 8: Modulus of volume compressibility.**

	% strain	Load TSF	$m_v$ sf/Ton		$m_v$ sf/lb		1/ $m_v$ psf
13B-1 27 feet	3.95	2	0.0047		2.35E-06		425,230
	18.06	32					
13B-2 23 feet	1.22	0.5	0.0325		1.62E-05		61,566
	12.59	4					
13B-2 46.5 feet	6.1	2	0.0042		2.12E-06		472,069
	18.81	32					
13B-2 61.7 feet	2.45	0.5	0.0387		1.93E-05		51,737
	15.98	4					
13B-3 16.8 feet	10.79	1	0.0041		2.07E-06		482,866
	23.63	32					
13B-3 21.8 feet	5.59	8	0.0034		1.69E-06		593,325
	13.68	32					
13B-3 41.7 feet	2.33	8	0.0018		9.17E-07		1,090,909
	6.73	32					
13B-3 51.4 feet	5.7	8	0.0037		1.83E-06		545,455
	14.5	32					
Average			0.0042		5.82E-06		481405

An average value of  $m_v$  was used to calculate the order of magnitude of settlements at each boring location.

The CPT logs in a spreadsheet were used for the calculation, and  $t$  (*thickness*) was obtained from the sampling interval (approximately 0.165 feet).  $\Delta E'$  (change in effective stress) is calculated from the increased load of the backfill at 105 pcf and the increased effective stresses as a result of lowering of the water table. Therefore, calculated settlement of the ash was attributed to both the increased backfill load and lowering of the water table. To calculate the settlement due to lowering of the water table, the drop in phreatic surface was conservatively estimated as 30' in SCPT-01 and SCPT-02, and 20' in SCPT-03.

The results of the analysis indicate that settlement of the fill at these locations ranges from 5 inches to 9 inches. The 9 inch value was calculated from the  $E_s$  values collected using CPT results.

**Table 9: Settlement Calculation Results**

		Totals from lower water table	Totals from increased backfill load	Total Settlement (S)
SCPT-01	Feet	0.23767	0.30683	<b>0.545</b>
	Inches	2.85205	3.68196	<b>6.53</b>
SCPT-02	Feet	0.196331	0.234791	<b>0.431</b>
	Inches	2.356	2.817	<b>5.17</b>
SCPT-03	Feet	0.165	0.277	<b>0.442</b>
	Inches	1.983	3.325	<b>5.31</b>

Settlement profiles were not calculated due to the complexities of the existing site configuration (i.e. irregular shape of serpentine channels and preloading of some areas to proposed fill heights). Nevertheless, we do anticipate a bowl like settlement profile centered on the existing serpentine channel areas, with maximum settlements of this magnitude near the center of the pond. This area is also traversed by the proposed collection channel. The proposed channel has a slight grade (necessarily to limit fill volumes) that may be susceptible to settlements of this magnitude. To limit the settlement of the channel after final grading, pre-loading of the area is recommended.

## 5. Conclusions

With the removal of the free water surface and installation of a liner, the phreatic surface will drop with a corresponding decrease in flow volume from the current rates. The level of the final phreatic surface will be controlled by ground water level in the natural hillsides behind the embankment.

Nevertheless, the piezometers currently installed around Pond 1 should be maintained and monitored until the phreatic surface through the pond has stabilized.

Amec Foster Wheeler has provided a conservative stability analyses that uses a high phreatic surface. The results indicate a factor of safety greater than the minimum recommended factor of safety for static and pseudo static conditions. Therefore, it can be concluded from this analysis that the stability of each cross-section is sufficient under these conditions. If the phreatic surface does drop as discussed above the FS for the embankment will increase. Moreover, the liquefaction potential analysis indicates a low potential for dynamic liquefaction.

Our analyses indicate a low potential for static liquefaction. However, we have provided the following measures to address the potential. First, static liquefaction is generally manifested during construction or other significant event that would result in a pore pressure increase. AEP does not propose a significant amount of fill above the current fines level. The maximum fill heights will occur in the existing pond area where they may reach 10 to 15 feet in height above the fines level. At these locations there is sufficient crest width to keep the potential failure surfaces confined to the localized areas of the pond and therefore the ash will continue to be contained.

During construction the contractor should anticipate localized failures (i.e. mud waves, bearing failures, etc) of ponded material in advance of the backfilling in these areas. To reduce the potential for this occurrence, we recommend removal of free surface water as far in advance of the backfilling as possible. Also, the backfill materials should be placed in very thin lifts after a relatively stable base is established. If the free water is not removed in a time frame that allows for self weight consolidation of the fines, some ash displacement and mixing of the fill material with the ash is anticipated.

Amec Foster Wheeler anticipates and our settlement analysis has demonstrated that consolidation of the fines will occur and thus the final surface may settle. The settlement, though minor, may affect the grades of the proposed collection channels. To mitigate the settlements the free water surface should be removed and the ponded areas preloaded as far in advance of final grading as practical.

## Attachment C

# Engineering Report and Calculations



**Engineering Report and Calculations  
Clinch River Pond Closure  
Pond 1A/1B – Clinch River Power Plant**

Russell County, Virginia

***Prepared For:***

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***Prepared By:***

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Amec Foster Wheeler Project No: 3050130220

September 8, 2015  
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## TABLE OF CONTENTS

APPENDICES .....	i
TABLES .....	i
PROJECT SUMMARY .....	ii
1.0 INTRODUCTION .....	1
1.1 Background.....	1
1.2 Purpose and Scope of Current Design.....	1
2.0 SITE DESCRIPTION .....	1
2.1 Existing Site Characteristics.....	1
2.2 Contributing Drainage Areas .....	2
3.0 DESIGN CONSIDERATIONS .....	2
3.1 General.....	2
3.2 Waste Products and Production Rates.....	4
4.0 ENGINEERING ANALYSES AND DESIGN .....	6
4.1 Stability .....	6
4.2 Hydraulic and Structural Design.....	6
5.0 CONSTRUCTION REQUIREMENTS .....	9
5.1 Sequence of Construction.....	9
5.2 Dewatering of In-Place Ash.....	10
5.3 Subgrade Contours with Subsurface Collection Drain .....	11
5.4 PVC Liner, Double Sided Drainage Net and Cover Material.....	11
5.5 Cover Soil Placement .....	11
5.6 Diversion Channel Construction.....	11
5.7 Pipe Abandonment .....	12
5.8 Tunnel Grouting .....	13
6.0 EROSION AND SEDIMENT CONTROL ANALYSIS AND CALCULATIONS .....	13
7.0 CONSTRUCTION QUALITY CONTROL MONITORING.....	16
8.0 CLOSURE .....	16

## APPENDICES

Appendix A – Construction Plans  
 Appendix B – Hydrology and Hydraulic Calculations  
 Appendix C – Channel Lining Calculations  
 Appendix D – Pipe Buoyancy Calculations  
 Appendix E – Temporary Sediment Basin Calculations

## TABLES

Table 1: Basic Characteristics of Pond 1 .....	2
Table 2: Anticipated Volumes to Reclaim Impoundment Summary .....	5
Table 3: Drainage Areas .....	7
Table 4: Rainfall Depths.....	8
Table 5: Design Flows.....	8
Table 6: Summary Channel Hydraulics .....	9

## **PROJECT SUMMARY**

Location: Town of Cleveland, Russell County, Virginia

Type: Engineering Design

Agency: Virginia Department of Conservation and Recreation (VDCR)

## **1.0 INTRODUCTION**

The Clinch River Power Plant is owned and operated by Appalachian Power Company (APCO), which is part of American Electric Power (AEP), and is located on the Clinch River in Russell County, Virginia off Route 665 near the Town of Cleveland. This facility currently uses an ash pond (Pond 1) to dispose of the bottom ash produced from the coal fired plant. In the future, AEP plans to close the currently active ash pond as they convert the Clinch River plant to be fueled by natural gas. With this design report and accompanying design plans, Amec Foster Wheeler aims to provide the information needed to assist AEP in the closure of the ash pond.

### **1.1 Background**

The Clinch River plant was built in 1958 and originally included three generating units, each having a capacity of 235 megawatts. In early 2015, Unit 3 was shut down, resulting in a total plant capacity of 470 megawatts. In the spring of 2015, the Clinch River plant began conversion of the remaining two generation units to natural gas. The plant was initially constructed with two ash ponds for disposing and storing coal combustion byproducts: Ash Pond 1 and Ash Pond 2. Ash Pond 1 is an active multi-cell pond composed of two cells (Pond 1A and Pond 1B) and will be the focus of this report. This report will provide the engineering calculations for the closure of Pond 1. Ash Pond 2 is currently closed and has been excluded from this analysis.

In 2013, Amec Foster Wheeler prepared 2 reports for Pond 1 which was, at the time, classified as a SIGNIFICANT hazard embankment. A report titled "Clinch River Diversion Berm" was prepared to divert water from the approximately 53 acre watershed west of Pond 1A. This channel was originally designed on a 0.5 Probable Maximum Precipitation (PMP) event according to the current regulations at that time. The second report titled "Clinch River Pond Closure" was prepared for the closure of the pond. Again, the 0.5 PMP and/or the 100 year precipitation events were used to design the channels that controlled runoff from the closed impoundments.

Subsequent to the submittal of these plans, the pond was reclassified as a HIGH hazard embankment, which required passage of the 0.9 PMP.

### **1.2 Purpose and Scope of Current Design**

Amec Foster Wheeler has designed a closure plan to grade and cap Ponds 1A and 1B to meet the requirements indicated above. The closure plan will eliminate impounding of water as well as conveying upstream stormwater runoff safely away from the toe of the impoundment. In addition, a PVC liner will cover the entire disposal area and 2' feet of clean fill will be installed above the liner. The existing toe drains will convey the contact water to the existing wastewater treatment plant, and surface runoff (i.e. non-contact water) will be conveyed to two NPDES outfalls. The two outfalls essentially divide the stormwater from Pond 1A and Pond 1B and convey stormwater either to the Clinch River (Pond 1A) or Dumps Creek (Pond 1B).

## **2.0 SITE DESCRIPTION**

### **2.1 Existing Site Characteristics**

Pond 1 is considered a side-hill impoundment. The Pond is used for sluicing and settling of ash byproducts. Pond 1 is separated by a splitter dike to form two cells: Pond 1A and Pond 1B. The pond is used for sluicing and settling of ash byproducts. Water and ash are currently

sluiced into Pond 1A. To facilitate settling, the sluiced ash is conveyed through serpentine channels that form the primary impounding volumes that classify the structure as an impoundment. Water is conveyed from Pond 1A to Pond 1B via a 30" pipe through the splitter dike. The principal spillway outlet consists of a 42" weir inlet structure located in Pond 1B. Under the normal operation of the Pond, the ash is removed by dredging and used as beneficial reuse materials.

As part of the closure for the pond, AEP has ceased dredging of the ash to allow the serpentine in Pond 1A to be partially filled. The majority of Pond 1A is in a dry state and is only impounding a small area of water near the splitter dike. There are similar plans to relocate the sluice lines to Pond 1B and partially fill the current impoundment area in that basin. The Pond 1A diversion channel is currently being constructed to the west of Pond 1A. Table 1 summarizes the characteristics of Pond 1.

**Table 1: Basic Characteristics of Pond 1**

Pond 1A/1B	
Dam Height (ft)	65
Crest Width (ft)	35
Length (ft)	3,150
Maximum Pool Surface Area (Acre)	21.0
Normal operating Pool (ft)	1,566.6/1,557.2
Nominal Crest Elevation (ft)	1,570

## 2.2 Contributing Drainage Areas

The total watershed area for the entire Pond 1 site is approximately 156.7 acres. As part of the closure plan for Pond 1, stormwater runoff from upstream drainage basins is to be diverted away from Pond 1 by means of diversion channels. The Pond 1A diversion channel will divert stormwater runoff from the drainage area west of Pond 1A to the Clinch River.

There are two separate drainage basins north of Pond 1A and Pond 1B. These drainage basins currently drain to diversion culverts located at the lower end of the drainage basin. The diversion culverts are 18" in diameter and convey stormwater flows under the Pond 1 splitter dike. During larger storm events the diversion culverts reach capacity, causing stormwater runoff to flow into Pond 1B. As part of the proposed Pond 1 closure, these diversion inlets will be abandoned and flows will be directed to the proposed Pond 1B diversion channel.

Further details on the drainage configuration associated with the closure of Pond 1 can be found in Appendix A – Construction Plans.

## 3.0 DESIGN CONSIDERATIONS

### 3.1 General

Pond 1 impounds water and material and is regulated as a dam by the Virginia Department of Conservation and Recreation (VDCR). The hazard classification of Pond 1 as described on its current certificate is a High Hazard. High Hazard structures are required to safely pass stormwater runoff for the 0.9 Probable Maximum Precipitation (PMP) event as documented in Virginia Administrative Code Chapter 20 Impounding Structure Regulations. The 0.9 PMP

storm depth for the project site is 25.9 inches for the 6 hour storm duration and 33.3 inches for the 24 hour duration, per NOAA HMR-51. The drainage features within the Pond 1 site are required to safely pass the stormwater runoff from the 0.9 PMP in a non-erosive manner.

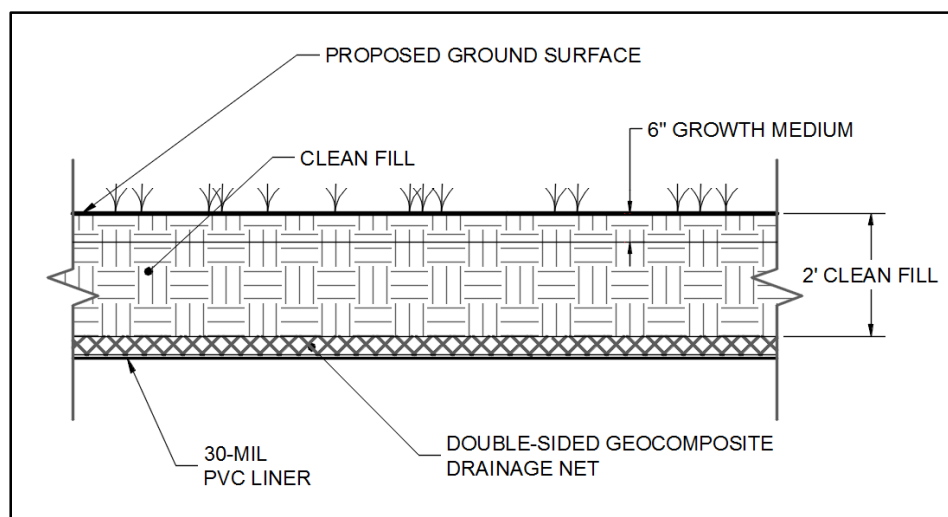
The high flows produced by these storms result in high velocities and, thus, substantial channel linings. Typically, these types of channels are constructed in rock; however, the rock on the side hill where the diversions are located has been impacted by a fault, and it is difficult to hold steep slopes. The majority of the hillside ditches for this project are sloped at a 2H to 1V side slope. While the slopes are assumed to be geotechnically stable, the rock is assumed to be erodible and various solutions for channel lining have been provided. These solutions include lining the channels with either riprap, gabion mattresses, grouted riprap, or a combination thereof. In the lower reach of the 1B Lower Diversion Channel, an exception to the 2 to 1 slope occurs. To maintain the best configuration for stormwater flows in this location, the slopes approach 1.5H to 1V and are to be stabilized with gabion mattresses. Detailed calculations for channel lining design can be found in Appendix C.

The proposed 48" stormwater pipe that will convey flows from the Pond 1B diversion channel to the existing tunnel sump will cross under the existing toe drain. The crossing will be located near the existing tunnel sump as shown on the construction drawings. Reconstruction of the toe drain will be needed if the toe drain system is disturbed during installation of the stormwater pipe.

The tunnel beneath SR616 and railroad will be modified to allow for the conveyance of stormwater from the Pond 1B diversion channel to Dumps Creek. Modification to the tunnel bottom will be required as the tunnel does not slope towards Dumps Creek. Grouting of the tunnel bottom will be required for positive drainage.

The Pond 1 site is a side-hill impoundment and, by nature, steep slopes exist on the site. The Pond 1A and 1B diversion channels each have very steep lower sections where the channel traverses the embankment slope of the dam. These steep channel sections require energy dissipation and containment of flood flows to the toe of the dam.

To lower the phreatic surface and reduce seepage contact water, the cap should provide a relatively impermeable barrier for infiltration water. The standard cap liner is shown in Figure 1 below.



**Figure 1: Soil Liner Detail**

As shown, the ash is initially covered with an impermeable PVC liner. Above the PVC liner, a double sided geocomposite drainage net is provided to control infiltration from the cap surface. The geocomposite drains to a central subsurface drainage pipe to allow the infiltration (non-contact) water to be drained off-site. At that point, clean fill is placed to serve as a protective cover and to provide a minimum of 1.5 feet of soil material above the ash.

### **3.2 Waste Products and Production Rates**

Table 2 provides a summary of anticipated volumes to grade the ash pond for the purpose of closure. In this table, volumes for both the subgrade materials (ash) and cap materials (clean fill and vegetative cover) are calculated. The subgrade volume calculations include the required fill and cut that will be associated with the proposed PVC Liner grading contours. In addition, 23,000 yd<sup>3</sup> of ash from the Temporary Ash Storage area (shown on the design drawings) will be included as part of the subgrade fill quantities. A total of approximately 3,000 yd<sup>3</sup> of additional fill will be needed (from an offsite location) to complete the subgrade grading.

The pond cap volume calculations include the required clean fill and vegetative cover that will be associated with the proposed final grading contours. Clean fill material will be generated from the construction of the proposed Pond 1B Upper and Lower Diversion Ditches. An expansion factor was first applied to this volume, and a 50% reduction factor was applied assuming that much of this material will be unsuitable for clean fill requirements. This unsuitable, clean material will need to be disposed of in a spoil area onsite. After accounting for the material generated from the proposed diversion ditch, approximately 34,000 yd<sup>3</sup> of clean fill material will be required and approximately 17,000 yd<sup>3</sup> of vegetative cover material will be required to complete the pond cap construction.

**Table 2: Anticipated Volumes to Reclaim Impoundment Summary**

SUBGRADE VOLUME CALCULATIONS	1A		1B	
	factor	cubic yards	factor	cubic yards
Subgrade Fill Volume Required		48,335		48,787
Ash Volume Added Between Now and Closure				(9,500)
Losses to Subgrade	0.2	9,667	0.2	7,857
<b>Net Subgrade Fill Volume Required</b>		<b>58,002</b>		<b>47,144</b>
Ash Cut Volume Available Onsite		61,700		29,900
Anticipated Earthwork Shrinkage Factor	0.1	6,170	0.2	5,980
<b>Net Ash Cut Volume Available Onsite</b>		<b>55,530</b>		<b>23,920</b>
<b>Additional Subgrade Volume Required</b>		<b>2,472</b>		<b>23,224</b>
Ash Fill Available Onsite (Temporary Ash Storage area)		23,000		
<b>Total Pond 1A/1B Subgrade Volume Shortage</b>	<b>(2,696) yd<sup>3</sup></b>			
2' CAP VOLUME CALCULATIONS	factor	cubic yards	factor	cubic yards
<b>Net Clean Fill Volume Required</b>		<b>36,750</b>		<b>15,975</b>
<b>Net Vegetative Cover Volume Required</b>		<b>12,250</b>		<b>5,325</b>
Clean Fill Cut Volume Available from 1B Diversion				31,000
Expansion Factor			1.2	37,200
% Not Meeting Clean Cover Requirements			0.5	(18,600)
<b>Net Clean Fill Cut Volume Available</b>				<b>18,600</b>
<b>Total Pond 1A/1B Clean Fill Volume Shortage</b>	<b>(34,125) yd<sup>3</sup></b>			
<b>Total Pond 1A/1B Vegetative Cover Volume Shortage</b>	<b>(17,575) yd<sup>3</sup></b>			



## **4.0 ENGINEERING ANALYSES AND DESIGN**

### **4.1 Stability**

In April 2014, Amec Foster Wheeler prepared a *Report of Geotechnical Analysis AEP Clinch River Power Plant Pond 1 Closure April 11, 2014*. This report detailed the geotechnical investigations and provided stability analyses of the proposed closure. Previous studies examined the current conditions of the embankment including the in-place material properties of the embankment material. These previous studies utilized material properties of the sluiced material gathered from other sites. The geotechnical analyses in that report summarize the liquefaction potential, seepage predictions, stability, and settlement of the cross-sections using the previously defined embankment geometry and soil properties.

Steady-state seepage analyses were conducted to determine the pore water pressure distribution throughout the cross-sections. The stability analyses were conducted using these pore water pressures at four cross sections under static and pseudo-static loadings. Deep failure slip circles as well as shallow failure slip circles were examined. The predicted phreatic surface was determined for each section, as well as the expected factor of safety based upon our analyses. Using these results, conclusions were made about the overall stability of this structure upon closure. Because the proposed grades for this closure plan are equal to or less than those analyzed previously, additional stability analyses demonstrating the new configuration were not necessary.

Amec Foster Wheeler's judgment of adequate safety factors were based on the Army Corps of Engineers (ACOE) document EM-1110-2-1902 (2003) and from Hynes-Griffin and Franklin (ACOE-1984). The ACOE specifies a minimum safety factor (FS) of 1.5 with respect to static loading for new dam construction. The ACOE indicate that this value may be lowered for existing structures where a performance history has been obtained. In this instance, we have used the FS=1.5 value on the existing Clinch River structure for conservative and comparative purposes. For pseudo-static loadings, the Hynes-Griffin and Franklin study recommends a safety factor of 1.0 with an accompanying 20% material strength reduction.

### **4.2 Hydraulic and Structural Design**

#### **4.2.1 Proposed Pond Closure Drainage Characteristics**

A topographic survey was conducted in March of 2012 and was used for the previous closure design. Additional surveys for the top surface of the pond were conducted in 2014 (Pond 1B) and 2015 (Pond 1A). These latter surveys were incorporated into the 2012 surveys and are used for the basis of the design and hydraulics and hydrology of the pond closure. The contributing drainage to the project site was divided into six subareas as shown in Appendix B.

The Pond 1A Diversion Channel can be divided into upper and lower sections. Stormwater runoff from Subarea 001 will be conveyed to the Clinch River by the Pond 1A diversion channel that is currently being constructed. The upper section receives drainage from the area west of Pond 1A (Subarea 001). The lower section is the steeper section that receives flows from the upper section and the Pond 1A collection channel.

Stormwater runoff from Subarea 002 and the surface of Pond 1A will be conveyed by the 1A collection channel across the reclaimed area of the Pond 1A basin. The 1A collection channel will connect to the existing Pond 1A diversion channel as shown on the construction plans.

Stormwater runoff from Subarea 003, 004, and the surface of Pond 1B will be conveyed away from Pond 1 by the proposed Pond 1B diversion channel. The Pond 1B diversion channel is divided into upper and lower sections. The upper section receives drainage from Subarea 003. The lower section receives drainage from Subarea 004 and the flows from the surface of Pond 1B. The final grading of Pond 1B will direct runoff north to the Pond 1B diversion channel via sheet flow. The Pond 1B diversion channel will convey storm flows to the toe of the existing impoundment.

At the toe of the 1B embankment, an inlet catch basin will capture flows from the 1B diversion channel. A 48" HDPE pipe will connect this catch basin with the existing tunnel and outfall into Dumps Creek. To meet VDOT drainage requirements, the inlet catch basin is designed to contain flows up the 25-year storm event. Flows will overtop the catch basin for storms greater than the 25-year and will be conveyed along the historic drainage path via sheet flow.

Table 3 below summarizes the contributing drainage area for the site drainage features.

**Table 3: Drainage Areas**

Drainage Feature	Contributing Drainage Area (acres)
Upper Pond 1A Diversion Channel	63.6
Lower Pond 1A Diversion Channel	82.7
Pond 1A Collection Channel	19.1
Upper Pond 1B Diversion Channel	22.7
Lower Pond 1B Diversion Channel	74.0
Pond 1B Surface	8.7
<b>Total Site Drainage Area</b>	<b>156.7</b>

#### 4.2.2 Hydrologic Modeling

The hydrologic analysis was performed with USACE HEC-HMS hydrology model. The Pond 1A and 1B diversion channels are required to control and properly convey the 0.9 PMP storm. Design flood hydrographs were developed for the 6-hour and 24-hour PMP storm durations. Per VDCR guidance, the most severe flood hydrograph was selected for the design of the drainage features. **Table 4** shows the rainfall depth for the 25-year (24 hour duration), 0.9 PMP (6 hour duration), and 0.9 PMP (24 hour duration). The precipitation distributions for the PMP storm events were set using guidance from the NRCS Technical Release for Earth Dams and Reservoirs (TR-60). The 25-year storm event was used for the design of the inlet catch basin at the outlet of the Pond 1B diversion channel. The SCS Type II distribution was used for the 25-year event.

**Table 4: Rainfall Depths**

PMP	Precipitation Depth (in)	Precipitation Distribution
25-yr (24-hr)	4.36	SCS Type II
0.9 PMP (6-hr)	25.92	TR-60
0.9 PMP (24-hr)	33.3	TR-60

The peak flowrates from the hydrology model were used to design the drainage features. The peak flows for each of the project subareas are shown in **Table 5**.

**Table 5: Design Flows**

Drainage Feature	25-yr (24-hr)	100-yr (24-hr)	0.9 PMF (6-hr)	0.9 PMF (24-hr)
Pond 1A Collection Channel	104	135	370	120
Upper Pond 1A Diversion Channel	135	207	1,089	385
Lower Pond 1A Diversion Channel	166	245	1,421	505
Pond 1B Collection Channel	36	45	158	55
Upper Pond 1B Diversion Channel	52	79	398	138
Middle Pond 1B Diversion Channel	133	205	1102	395
Lower Pond 1B Diversion Channel	169	250	1,259	450

### 4.2.3 Hydraulic Analysis

A hydraulic model was set up through the entire project reach using the HEC-RAS 4.1 software. The model was used to evaluate water surface elevations in Pond 1A and 1B diversion channels along with other design parameters. The peak flowrates calculated from the hydrologic analysis were input into the hydraulic model. Design considerations included available freeboard from the HGL and EGL to the top of channel and the channel bed and side slope shear stress during the 0.9 PMP storm.

The SWMM5.0 model was used in the design of the catch basin inlet and outlet pipe. The catch basin was designed to retain flows onsite up to the 25-year event.

**Table 6** below summarizes the hydraulics of the Pond 1A and 1B diversion channels. Further details on the hydraulics of the diversion channels and catch basin can be found in Appendix B. Discussion on the construction and configuration of the channels is discussed in the construction section below.

**Table 6: Summary Channel Hydraulics**

Drainage Feature	Mean Channel Slope (%)	Flow (cfs)	Mean Flow Depth (ft)	Mean Velocity Cross Section (ft/s)	Mean Channel Bed Shear Stress (lb/ft <sup>2</sup> )	Mean Channel Side Slope Shear Stress (lb/ft <sup>2</sup> )
Pond 1 A Diversion Channel (Upper)	3.0%	1089	3.7	12.8	7.3	3.4
Pond 1 A Diversion Channel (lower flume section)	28.5%	1421	2.6	39.1	5.3	0.1
Pond 1A Collection Channel	0.7%	370	1.1	3.1	0.4	0.2
Pond 1B Diversion Channel (Upper)	2.0%	398	3.8	7.2	3.1	1.4
Pond 1B Diversion (Lower section)*	2.0%	1259	3.6	9.3	2.9	1.5
Pond 1B Diversion (Spillway through embankment)	16.0%	1259	1.9	23.0	12.5	5.5
*Pond 1B Diversion Channel upstream of spillway through embankment						

## 5.0 CONSTRUCTION REQUIREMENTS

The plans for the closure of Pond 1 are provided in the appendices. The requirements to close Pond 1 include capping the ash with an impermeable geosynthetic liner system and placing a minimum of 2 feet of clean cover material that will be regraded to eliminate impoundment of water. Current projections estimate that the plant will continue to operate through the fourth quarter of 2015 by discharging into Pond 1B. However, the closure of ponds (1A/1B) will be coordinated to allow for continued plant operations in the event that discharging to Pond 1B is required in 2016.

In general, the modifications will include the following major construction activities:

### 5.1 Sequence of Construction

1. Discharge to Pond 1A has ceased at the time of this report.
2. The construction of the Pond 1A Diversion Channel was close to completion at the time of this report.
3. The slurry discharge and pump plant discharge slurry has been relocated to Pond 1B at the time of this report.
4. Grading of Pond 1A.
  - a. Dewater Pond 1A and maintain conveyance of stormwater to Pond 1B.
  - b. Establish sediment trap and pump system above concrete flume.
  - c. Begin regrading of ash above this trap to attain the subgrade contours.
    - i. Grading is to progress from this downstream location upstream
    - ii. As sections are prepared grade subgrade to provide subsurface drainage channel.
    - iii. Install PVC geomembrane and double-sided geosynthetic drainage net
  - d. Import Pond 1A clean cover material and provide cover for the placed liner and drainage net as sections are prepared.

- i. Construct Pond 1B Upper Diversion Channel. Material from this excavation meeting the specified requirements may be used as Pond 1A cap cover material.
    - ii. As part of the Pond 1B Upper Diversion Channel, the diversion pipe in this hollow will be abandoned.
  - e. Import vegetative cover material for Pond 1A.
5. Begin closure of Pond 1B.
- a. Stop sluicing into Pond 1B.
  - b. Construct the Lower Diversion Channel to existing principal spillway.
  - c. Dewater Pond 1B and maintain capacity by pumping.
  - d. Abandon existing spillway pipe.
  - e. Complete construction of the Lower Diversion Channel to toe of dam.
  - f. The existing depression at the east end of 1B will be reconfigured to serve as the sediment trap for the closure. With this configuration the contractor can choose to regrade in either direction; upstream to downstream or downstream to upstream.
  - g. Regrade ash to attain the subgrade contours.
  - h. Install PVC geomembrane and double-sided geosynthetic drainage net.
  - i. Import Pond1B clean cover material.
  - j. Import vegetative cover material on Pond 1B.
  - k. Complete construction of cap.

This sequencing is not to be considered as imparting means and methods on the contractor. These are only a recommended work sequence to illustrate sediment control and constraints by the ongoing operations.

## 5.2 Dewatering of In-Place Ash

To provide a suitable subgrade for the fill ash, it will be necessary to dewater the sluiced ash. Without proper dewatering there is the potential for localized bearing failures in advance of the pushout, which can result in loss of equipment or mud waves. The failures are the result of generally weak slurried materials (not fully consolidated) and buildup of excess pore pressures in the sluiced ash. The contractor should provide additional details prior to construction on dewatering and stabilization of sluiced ash. We have provided some example methods below; however, the contractor should choose his own means and methods.

The most common method to place fill over soft subgrades is bridging. This method utilizes a thick lift of material, fill ash, repeatedly placed over the sluiced ash until the mixing and displacement of the two materials provides sufficient strength and distribution of bearing pressure to allow for the equipment to go over the recently placed fill and extend the pushout further into the pond area. This method can significantly increase the amount of material required for reclamation.

A second bridging method is to utilize a separation fabric, drainage layer and load distribution fabrics and thin lift construction. Generally, a geogrid is placed on the slurried ash followed by a geotextile drainage fabric and a thin drainage layer consisting of rock. This initial lift is followed by additional thin lifts of fill ash until sufficient load distribution of the equipment and fill material is provided.

In a worst case condition it may be necessary to provide dewatering wells to lower the phreatic surface in the sluiced ash. Because the ash is fine grained and may drain slowly, this operation

could delay the closure of the Pond. At the Clinch River Plant the dikes for the serpentines provide good areas for installing relatively shallow (5 to 10 feet below current ash level) well points.

### **5.3 Subgrade Contours with Subsurface Collection Drain**

Subgrade construction will include placement of the existing ash currently above the waterline to attain the subgrade contours shown on Drawing 16. The majority of available cut material will be located at the western end of Pond 1A. We anticipate that this material will be graded to the east and filling in the serpentines and sediment ponds lastly. Following preparation of the sluiced ash surface, the fill material should be placed in 8 inch lifts and compacted to 95% of the maximum Standard Proctor Density. Moisture conditioning of this fill may be necessary to attain the desired compaction.

After the subgrade contours are established the Subsurface Collection Drain can be constructed and lined as shown on Drawing 31. The subsurface collection channel is provided to collect infiltration through the 2 foot cap soils and convey the water collected above the PVC liner through the double sided drainage net. The Subsurface Drainage Pipe will daylight in the concrete flume to convey this water offsite.

### **5.4 PVC Liner, Double Sided Drainage Net and Cover Material**

The cap and cover system will be constructed as shown in Figure 1. The regraded ash is initially covered with an impermeable PVC liner. Above the PVC liner, a double sided geocomposite drainage net is provided to control infiltration from the cap surface. The geocomposite drains to a central subsurface drainage pipe to allow the infiltration (non-contact) water to be drained off-site. At that point, clean fill is placed and compacted to provide a minimum of 1.5 feet of material above the ash.

### **5.5 Cover Soil Placement**

The top surface of Basin 1A is approximately 15 acres. Again as prescribed by the regulations, 2 feet of clean cover soil is to isolate the ash from the surface. The cover soil will be obtained from an off-site borrow source free of gravel larger than 3 inches. The upper 6-inches is to be suitable as a growth medium.

As described in Section 3.2, the excavation of the 1B Upper and Lower Diversion Channels will provide approximately 18,600 cubic yards of material that may be suitable for the cover soil.

The cover soil should be placed in 8-inch loose lifts and compacted to 95% of the Standard Proctor Maximum Dry Density.

### **5.6 Diversion Channel Construction**

#### **1B Upper Diversion Channel**

The 1B Upper Diversion Channel is to be constructed during Pond 1A closure. This is to redirect the watershed above this channel from flowing to Pond 1A and direct it to Pond 1B. Pond 1B in the interim condition can pass this storm flow. The channel is primarily cut into rock, however due to the reported nature of the rock we anticipate that it will be rippable and will therefore require a channel liner to prevent erosion.

The 1B Upper diversion channel is a trapezoidal channel with 2H: 1V and 3H: 1V side slopes. Although cut into rock, the channel will require riprap because it is friable. This channel section



will be lined with Class I riprap to Station 5+80. At Station 5+80 the channel will temporarily discharge into Pond 1B until the lower section of diversion is constructed.

### 1B Lower Diversion Channel

The 1B Lower Diversion Channel is to be constructed during Pond 1B closure. The lower diversion channel will be constructed in two sections. The first section will extend to location of the principal spillway at Station 8+00. The principal spillway will be abandoned during construction of the second section which will require Pond 1B levels to be maintained by pumping. The second section is from Station 8+00 to Station 12+50 and will end at the toe of the 1B embankment.

Station 0+00 to Station 4+00 will be a trapezoidal channel with 2H:1V side slopes. This channel section will be lined with Class AI riprap. From Stations 4+00 to 5+50, the ditch bottom and right side slopes will be lined with Class 1 riprap. The left side slopes will be lined with gabion mattresses consisting of 4-6" fill stone at 1.0' thick.

From Station 5+50 to Station 11+00 the channel left side slope will cut into the existing hillside rock. The rock cut will be at 1.5H:1V. Due to the rippable nature of the rock and steeper side slopes, gabion mattresses will be used along the left side slopes and will consist of 4-6" stone at 1.0' thick. The channel bed and right side slope will be lined with Class I riprap.

At Station 11+00 the 1B diversion channel will enter a 17% grade to discharge at the toe of the existing embankment. From Station 11+00 to Station 12+20 the channel has a bottom width of 25 feet with 2H:1V side slopes. The channel will be lined with Class II grouted riprap approximately 3 feet deep.

During this time construction of the stormwater catch basin and stormwater pipe will also take place. The catch basin will be located at end of the 1B diversion channel. The catch basin will require construction of stormwater containment wall approximately 3 feet in height, and will also involve construction of a inlet structure that will receive storm flows and convey them the existing tunnel sump.

Upon completion of the 1B diversion channel flows from the upstream drainage areas will be diverted away from Pond 1 and will allow for completion of the Pond 1B cap.

## **5.7 Pipe Abandonment**

The construction of the upper 1B diversion channel will allow for the abandonment of the 18" stormwater pipe at the base of the western hollow (Subarea 003). This section of pipe will be removed to its connection point with the 18" pipe that drains the eastern hollow (Subarea 004) as shown on the construction drawings. Stormwater from the western hollow will be conveyed by the constructed upper 1B diversion channel into Pond 1B.

During construction of the lower 1B diversion channel the 18" stormwater pipe will removed to allow for construction of this channel. This section of pipe will be removed to its connection point with the existing manhole junction at the splitter dike. Stormwater from the eastern hollow will enter directly into Pond 1B during construction lower channel.

The pipe section the runs under the splitter dike will need to be abandoned by grouting. The pipe will be cut where it daylight at the toe of the Pond 1 embankment slope. A bulkhead will

be constructed at the location of the cut to allow for grouting of the pipe. Grout will be pumped into the pipe from the bottom to prevent air pockets from forming within the pipe.

The principal spillway located at Pond 1B will be abandoned during construction of the 1B diversion channel. The 36" spillway pipe will be abandoned by grouting. The pipe will be cut where it daylights at the toe of the Pond 1 embankment slope. A bulkhead will be constructed at the location of the cut to allow for grouting of the pipe. Grout will be pumped into the pipe from the bottom to prevent air pockets from forming within the pipe.

The water levels in Pond 1B will be maintained by pumping after the abandonment of the principal spillway.

## **5.8 Tunnel Grouting**

The tunnel that runs from the existing tunnel sump to Dumps Creek carries utilities underneath the railroad tracks. These utilities will be abandoned and demolished and the tunnel will be used to convey stormwater from the 48" HDPE pipe to Dumps Creek. The tunnel is oval in shape with an approximate height of 6' 6" and width of 8' 11". The tunnel slopes up as it runs toward Dumps Creek and will require modification. Grouting of the tunnel bottom will be required to obtain a positive slope through the tunnel for proper conveyance of stormwater to Dumps Creek. Construction activities will include abandonment of the utilities within the tunnel and grouting of the tunnel invert to gain positive drainage as shown on the construction plans.

## **6.0 EROSION AND SEDIMENT CONTROL ANALYSIS AND CALCULATIONS**

### ***Site Description***

The site consists of wooded and steeply sloped hollows, generally 20% to 40% slopes on the west and the remaining areas generally sloped from 10% to 30% slopes. The site is covered with volunteer heavy, wooded vegetation, predominantly hardwoods, and 15 to 20 feet high. There is no evidence of significant erosion under present site conditions.

### ***Soils***

The soil in all subareas are part of hydrologic soil group C as defined by Exhibit A-1 of TR-55 using the soil types derived from a site specific Web Soil Survey from the NRCS. The USDA NRCS RUSLE2 computer program was used to estimate the projected erosion rate on the ash disposal area cap system. The program estimates soil loss based on the Revised Universal Soil Loss Equation (RUSLE). The final cover slope described produces an estimated soil loss of 0.28 tons per acre per year for a silty clay loam type final soil cover.

### ***Ground Cover***

An onsite inspection was made to determine the existing vegetation. The drainage area is located in a rural area and is heavily wooded. There are areas of hardwood growth on the north, east, and west sides of the site. Tree lines are shown on the topographic map along with the type of cover on the rest of the site.

### ***Adjacent Property***

State Route 665 borders the property on the east side. On the north, south and east side is large tracts of heavily wooded land owned by different land owners. On the north side of the facility over the ridge is Lake Bonaventure, but is considered a separate watershed.



### ***Drainage Patterns***

The site consists of six subbasins identified as Subarea 001, Subarea 002, Subarea 003, Subarea 004, Pond 1A, and Pond 1B, as shown in Appendix B. The approximate acreage of each of these areas is also identified in Appendix B.

### ***Site Plan Development***

The maps developed for data collection and analysis were used to help determine the most suitable area for development and most critical areas from an erosion control standpoint. Erosion potential was one of the major factors which were considered in locating the channels.

The final site plan shown in Appendix A was developed through a balanced evaluation of such factors as convenience, drainage, maintenance, costs, aesthetics, erosion potential during construction, and stormwater runoff after construction.

### ***Plan for Erosion and Sediment Control***

The limits of grading are outlined in the plans presented in Appendix A so that the areas requiring erosion and sediment control practices could be determined. Unless otherwise indicated, all structural and vegetative erosion and sediment control practices shall be constructed and maintained according to minimum standards and specifications of the Virginia DCR handbook.

### ***Structural Measures***

- Sediment Basin
  - The proposed improvements are completely drained by a single channel. A sediment basin constructed across the swale below all construction will be the most effective method for removing sediment from the runoff before it leaves the site. The basin will be designed to accommodate the removal of accumulated sediment and will be used during the construction of the upstream channel until final stabilization. Calculations for the sediment basin are presented in Appendix E.
- Check Dams and Wattles
  - Rock check dams and straw wattles built across the drainage swale up-slope from the sediment basin and will greatly reduce the velocity of runoff from both the construction site and the adjacent property. This measure will reduce ditchline erosion and help increase the effectiveness by allowing more sediment to settle before the runoff reaches the basin.
- Silt Fence
  - Silt fence will be installed downslope of all disturbed areas with minimal slopes to filter sheet flow runoff before it enters the drainage swale.
- Construction Entrance
  - A construction entrance will be required to clean the tires of vehicles and equipment entering the site. There is potential to track sediment on Virginia State Route 665 therefore construction entrances will be required at all entrances to construction areas.
- Concrete Washout

- A concrete washout will be required to clean concrete trucks for the proposed flatwork. After concrete is poured at a construction site, the chutes of ready mixed concrete trucks must be washed out to remove the remaining concrete before it hardens. Concrete washout facilities should be inspected daily and after heavy rains to check for leaks, identify any plastic linings and sidewalls have been damaged by construction activities, and determine whether they have been filled to over 75 percent capacity. When the washout container is filled to over 75 percent of its capacity, the washwater should be vacuumed off or allowed to evaporate to avoid overflows. Then when the remaining cementitious solids have hardened, they should be removed and recycled.

### ***Vegetative Measures***

- Temporary Seeding
  - Certain areas of the site will be rough graded as a first stage of construction. Final grading will not occur until near completion of the project. These areas shall be seeded temporarily with fast germinating temporary grasses to reduce erosion potential. Diversion dikes and the sediment basin embankment shall also receive temporary seeding.
- Permanent Seeding
  - Immediately following final grading, permanent vegetation shall be applied in accordance with and overall landscape plan for the site. Seeding information is presented in the drawings in Appendix A. Erosion control blanket will be installed over fill slopes which have brought to final grade and have been seeded to protect the slopes from rill and gully erosion and to allow the seed to germinate properly. Mulch will be used on relatively flat areas. In all seeding operations seeding and fertilizer will be applied prior to mulching.
- Stabilization of Earthen Structures
  - All earthen structures such as sediment basins should be seeded and mulched immediately after being constructed with fast germinating temporary vegetation to help prevent structural damage or failure. This will also help to ensure that the structure itself will not become part of an erosion problem.

### ***Management Strategies***

- Construction traffic should be limited to access roads and areas to be graded. All traffic should be prohibited from crossing drainage swales except were absolutely necessary.
- The sediment basin will be installed as a first step in grading.
- All major grading should be completed with 90 days of the beginning of the project. Temporary seeding shall be applied immediately after the grading is completed on the respective areas.
- Responsibility for plan implementation should be given to the construction superintendent, and he/she should make all construction workers aware of the provisions of the plan.
- All erosion and sediment control measures shall be checked continuously and especially after each significant storm to locate damage and conduct maintenance operations.
- After achieving adequate stabilization, temporary erosion and sediment controls will be removed.

## **Maintenance**

In general, all erosion and sediment control measures will be checked daily and after each significant rainfall. The following items will be checked in particular:

1. The sediment basin will be cleaned out when the level of sediment buildup after each rain event.
2. The temporary riprap and construction entrance will be checked regularly for sediment buildup which will prevent drainage. If the rock is clogged with sediment, it shall be removed and cleaned or replaced.
3. The silt fence barrier will be checked regularly for undermining or deterioration of the fabric. Sediment shall be removed when the level of sediment deposition reaches halfway to the top of the barrier.
4. The seeded areas will be checked regularly to ensure that a good stand is maintained. Areas should be fertilized and re-seeded as needed.

## **7.0 CONSTRUCTION QUALITY CONTROL MONITORING**

The design techniques and construction drawings for the proposed modifications and appurtenant structures are based on data developed from our geotechnical, hydrologic and hydraulic analyses and designs, and stability analyses/designs. The actual layout and extent of many of the diversion channels and closure cap features may require modification in the field based on actual mass conditions exposed during construction. Therefore, our personnel should assist with the construction engineering and quality control monitoring throughout the proposed construction.

## **8.0 CLOSURE**

We appreciate this opportunity to be of continuing service to American Electric Power. We also look forward to assisting you throughout successful completion of the pond closure. If you have any questions or comments about the design or construction proposed herein, please call at your convenience.

Sincerely,

**Amec Foster Wheeler Environment & Infrastructure, Inc.**

A handwritten signature in blue ink, appearing to read "J. McDaniel".

Jon McDaniel, P.E.  
Project Manager

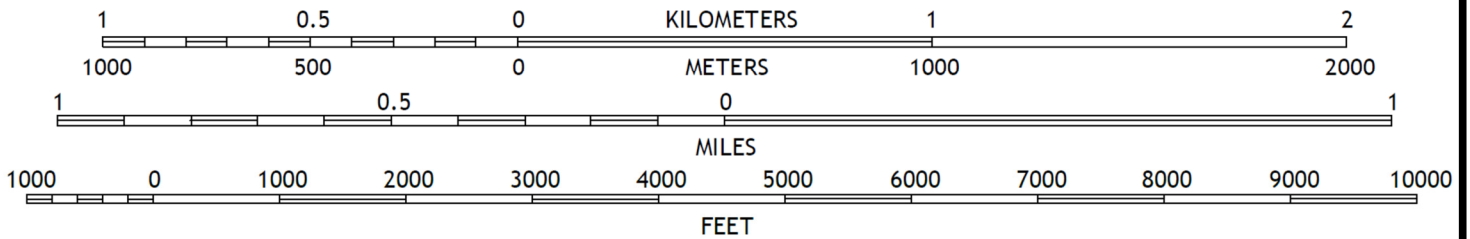
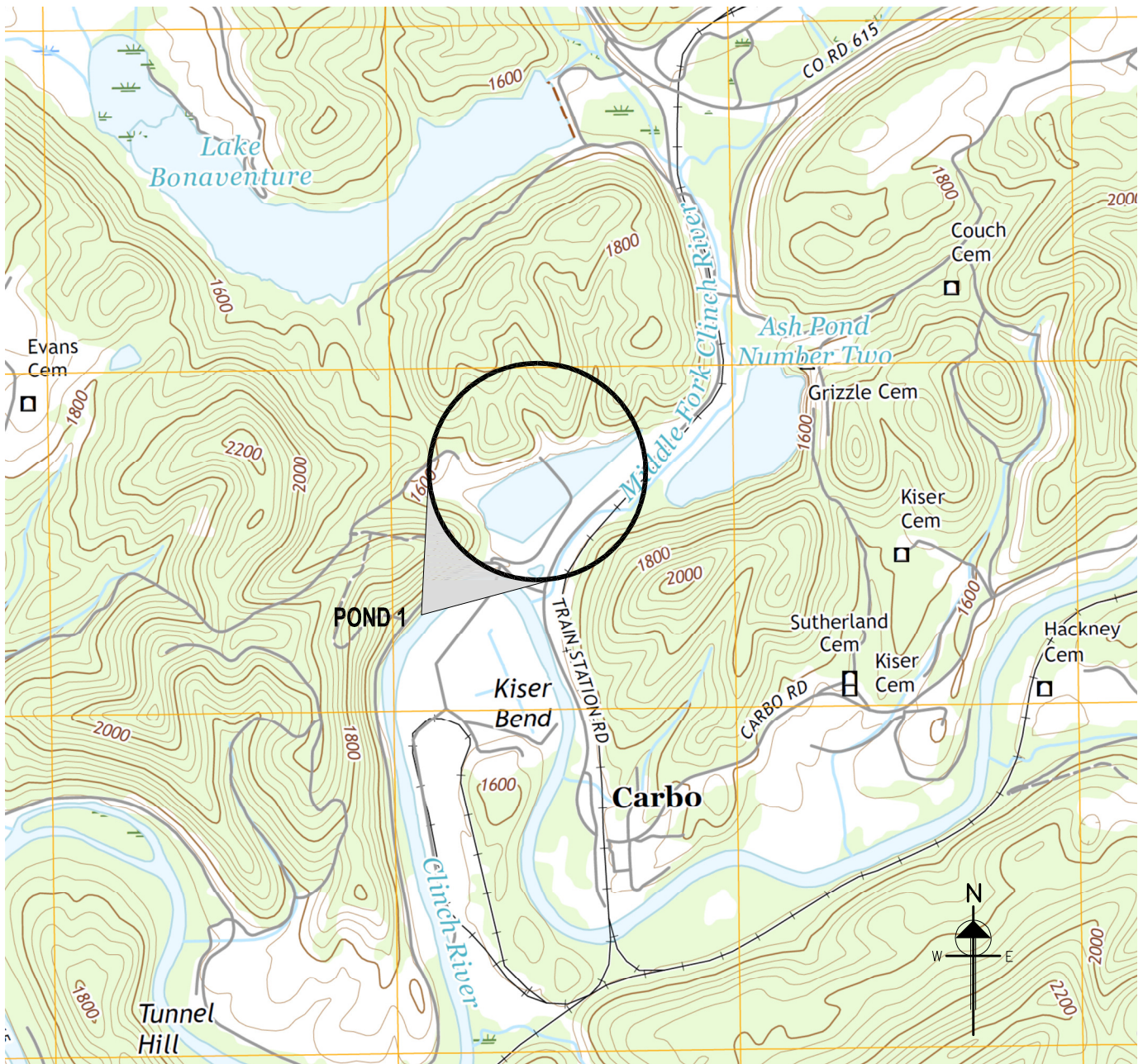
A handwritten signature in blue ink, appearing to read "Luke C. Williams".

Luke Williams, P.E.  
Senior Engineer

**ATTACHMENT B**

**Figures and drawings**





THIS DRAWING IS CLASSIFIED AS:

**AEP PUBLIC**

APPALACHIAN POWER COMPANY

**CLINCH RIVER PLANT**

REFERENCE AEP's CORPORATE INFORMATION SECURITY POLICY

CARBO

VIRGINIA

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POND 1  
**USGS TOPO MAP**  
7.5-MINUTE SERIES

UNIT:  
13

DRAWING NUMBER:  
LOCATION MAP

REV:  
1

SCALE: 1"=2000'

**CIVIL ENGINEERING**

DR:

CH:

SUP:

ENG:

DATE: June 20, 2018



**AEP SERVICE CORP.**  
1 RIVERSIDE PLAZA  
COLUMBUS, OH 43215

ASH FILL PEEP

Note:  
Field to provide a sand  
center pipe thru side of  
this Hwy 25.5 into Clinch River

Clinch River Plant, Ash Storage  
Ash Storage (10 Units, 10 Ash & Bottom Ash)  
Ash Production (100,000 Cu Yds. per Year)  
Assumed Bottom Storage (1000)  
To Store 1000 = 1,000,000 Cu Yds. - 20 Years  
To Store 1000 = 2,000,000 Cu Yds. - 40 Years (Based on building future Dike of Ash.)

Boring 12

Boring 17

12-3023A

12-3023A

12-3023A

12-3023A

12-3023A

12-3023A

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SECTION F-F  
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SECTION G-G  
Scale: 1/100'

SECTION D-D  
Scale: 1/100'

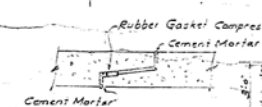
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SECTION C-C  
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SECTION A-A  
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SECTION B-B  
Scale: 1/100'

SECTION E-E  
Scale: 1/100'



CONCRETE PIPE JOINT DETAIL  
Cradle to be founded on material  
other than soil.  
Remove material where required  
and backfill with granular material.

TYPICAL SECTION THRU CRADLE

REFERENCE DRAWINGS  
12-3002 Location of Borings  
12-3011 Log of Borings  
State of Virginia - Dept. of Highways  
Plans & Sections

NO.	DESCRIPTION	DATE	BY	CHKD.	APPROVED
11	Added Hwy 616, 617, 618, 619, 620, 621, 622, 623, 624, 625, 626, 627, 628, 629, 630, 631, 632, 633, 634, 635, 636, 637, 638, 639, 640, 641, 642, 643, 644, 645, 646, 647, 648, 649, 650, 651, 652, 653, 654, 655, 656, 657, 658, 659, 660, 661, 662, 663, 664, 665, 666, 667, 668, 669, 670, 671, 672, 673, 674, 675, 676, 677, 678, 679, 680, 681, 682, 683, 684, 685, 686, 687, 688, 689, 690, 691, 692, 693, 694, 695, 696, 697, 698, 699, 700, 701, 702, 703, 704, 705, 706, 707, 708, 709, 710, 711, 712, 713, 714, 715, 716, 717, 718, 719, 720, 721, 722, 723, 724, 725, 726, 727, 728, 729, 730, 731, 732, 733, 734, 735, 736, 737, 738, 739, 740, 741, 742, 743, 744, 745, 746, 747, 748, 749, 750, 751, 752, 753, 754, 755, 756, 757, 758, 759, 760, 761, 762, 763, 764, 765, 766, 767, 768, 769, 770, 771, 772, 773, 774, 775, 776, 777, 778, 779, 780, 781, 782, 783, 784, 785, 786, 787, 788, 789, 790, 791, 792, 793, 794, 795, 796, 797, 798, 799, 800, 801, 802, 803, 804, 805, 806, 807, 808, 809, 810, 811, 812, 813, 814, 815, 816, 817, 818, 819, 820, 821, 822, 823, 824, 825, 826, 827, 828, 829, 830, 831, 832, 833, 834, 835, 836, 837, 838, 839, 840, 841, 842, 843, 844, 845, 846, 847, 848, 849, 850, 851, 852, 853, 854, 855, 856, 857, 858, 859, 860, 861, 862, 863, 864, 865, 866, 867, 868, 869, 870, 871, 872, 873, 874, 875, 876, 877, 878, 879, 880, 881, 882, 883, 884, 885, 886, 887, 888, 889, 890, 891, 892, 893, 894, 895, 896, 897, 898, 899, 900, 901, 902, 903, 904, 905, 906, 907, 908, 909, 910, 911, 912, 913, 914, 915, 916, 917, 918, 919, 920, 921, 922, 923, 924, 925, 926, 927, 928, 929, 930, 931, 932, 933, 934, 935, 936, 937, 938, 939, 940, 941, 942, 943, 944, 945, 946, 947, 948, 949, 950, 951, 952, 953, 954, 955, 956, 957, 958, 959, 960, 961, 962, 963, 964, 965, 966, 967, 968, 969, 970, 971, 972, 973, 974, 975, 976, 977, 978, 979, 980, 981, 982, 983, 984, 985, 986, 987, 988, 989, 990, 991, 992, 993, 994, 995, 996, 997, 998, 999, 1000				

REVISIONS

UNITS 100  
ASH DIKE AND  
RELOCATION OF DUMPS CREEK  
AND STATE HWY 616

APPALACHIAN ELECTRIC POWER CO.  
CLINCH RIVER PLANT  
CARBO, VIRGINIA

DR. NO. 12-3023-N

ARCH. ELEC. MECH. CIV. ST.

SCALE: 1/100'

DATE: 12/1/50

DATE: 12/1/50

DATE: 12/1/50

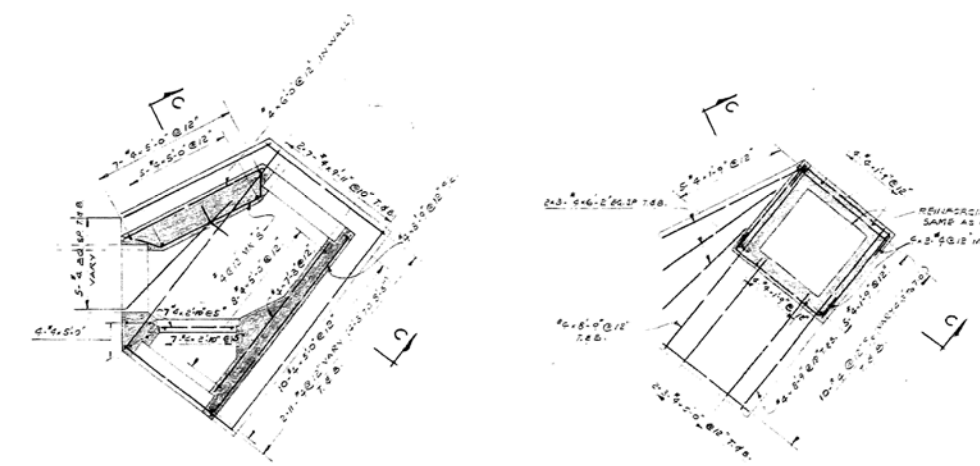
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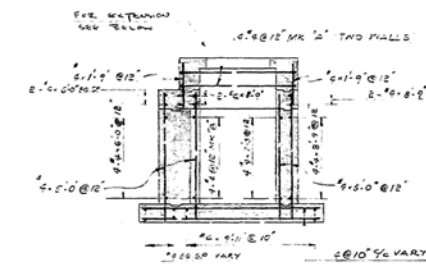
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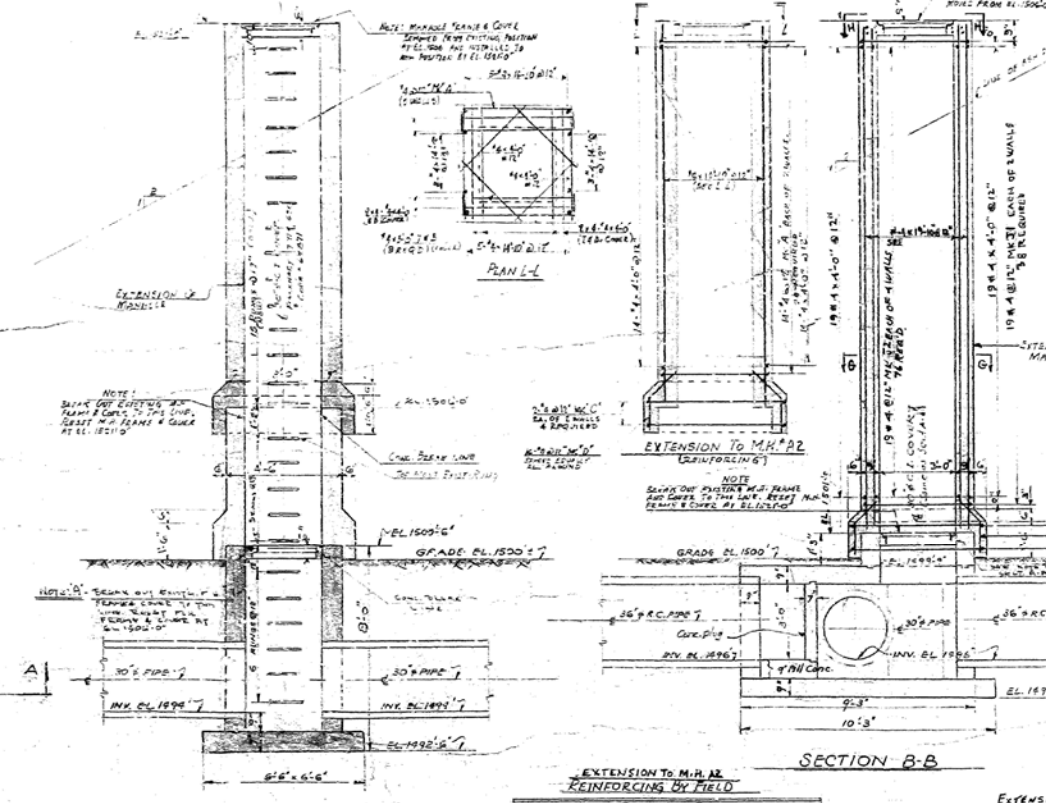




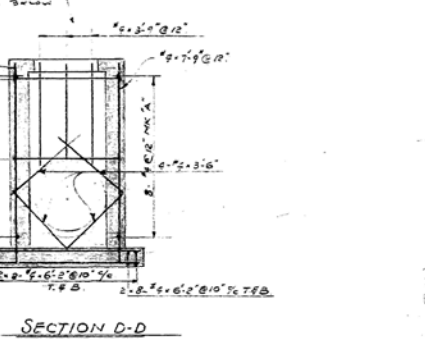
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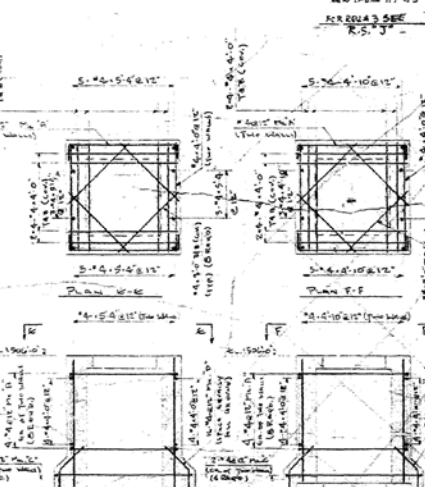
SECTION D-D



SECTION-B-B



SECTION D-D



EXTENSION: MW. 9.2

WT = 4.1060 LB - 0.58T  
CONCENTR = 15.5 Cu Yds  
RINGS (CALC) = 10 RINGS 75.41 LB Tens - GR 10579  
2-10P 2.5 CORER. PLUMMET TYPE 610  
CORE # 48271 BY: SUMMITT: 100Y  
LCK 10579

Run 1  
WT = 4.400 LB - 0.51T  
CONCENTR = 15.5 Cu Yds  
RINGS (CALC) = 10 RINGS 75.41 LB Tens - GR 10581

Run 2  
WT = 4.400 LB - .51T Tens  
CONC = 15.5 Cu Yds  
RINGS (CALC) = 10 RINGS 75.41 LB Tens - GR 10582

[illegible][illegible]

UNITS 1&2-ASH AREA  
MANHOLES-A1&A2  
MASONRY & REINFORCING

APPALACHIAN ELECTRIC POWER CO.  
CLINCH RIVER PLANT

DB No 13-3033 A-1

ARCH.	ELEC.	MECH.	STR.
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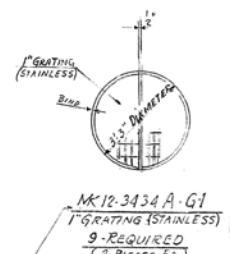
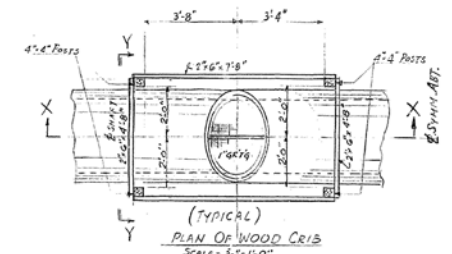
SCALE: AS NOTED APPROVED 5/5

DR. J. G.	
CH. T. S.	

DATE 8-30-57

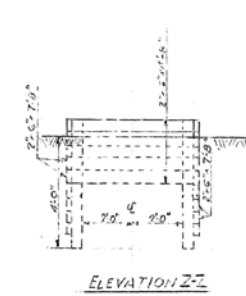
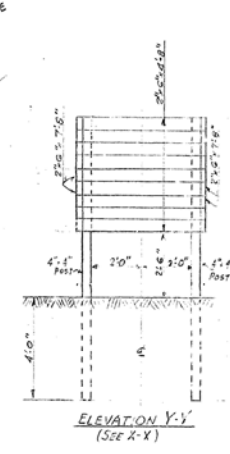
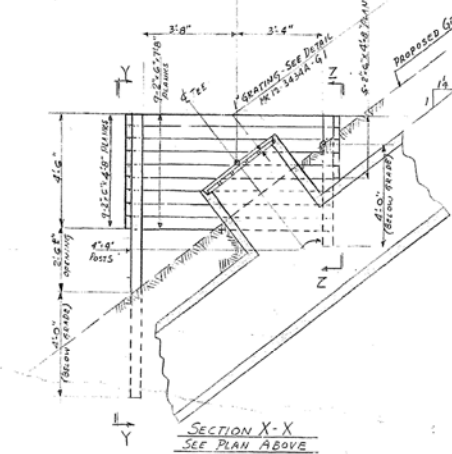
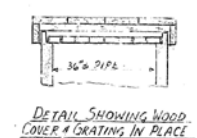
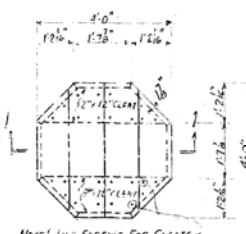
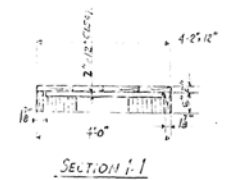
AMERICAN GAS & ELECTRIC SERVICE CORP.  
10 CHURCH STREET

9 0 1 1

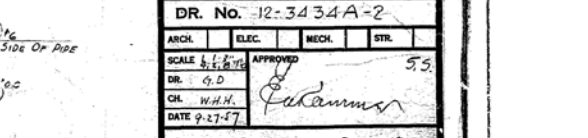
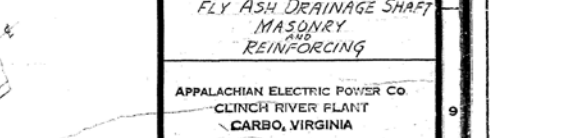
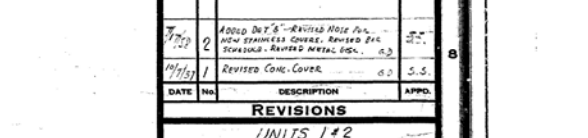
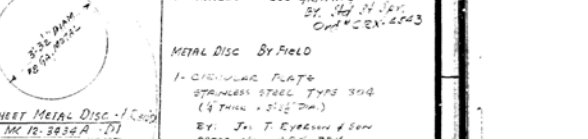
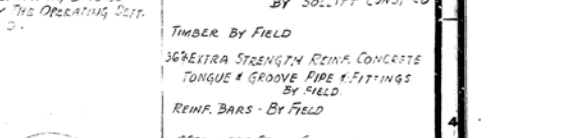
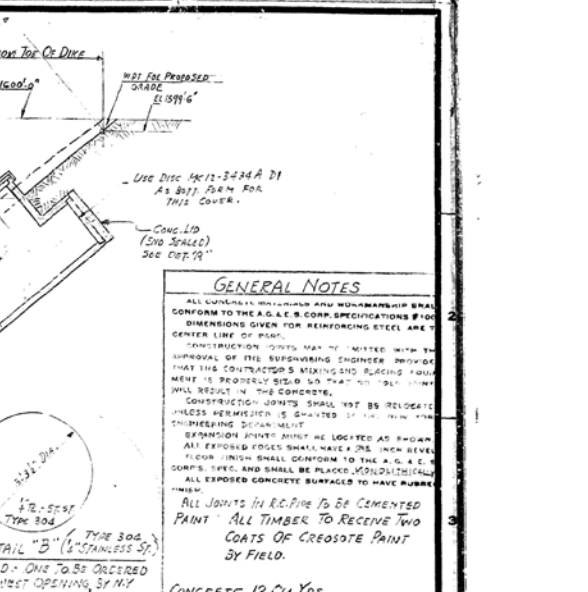
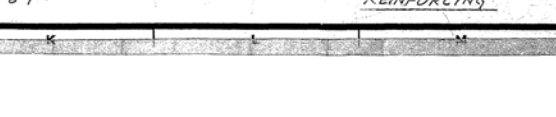
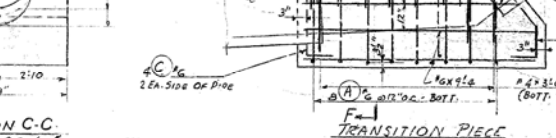
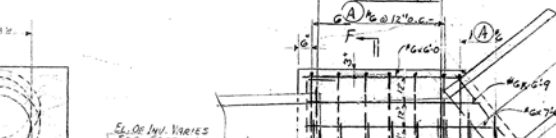
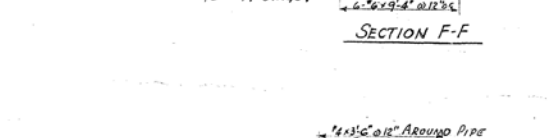
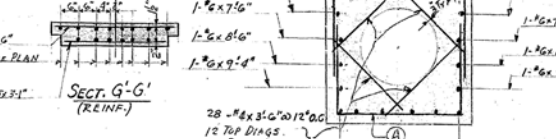
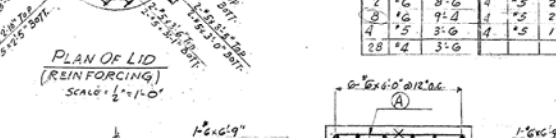
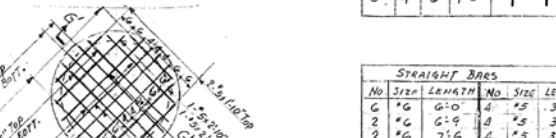
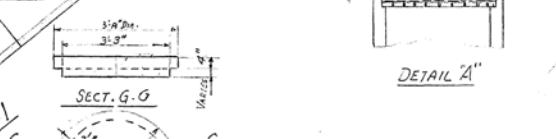
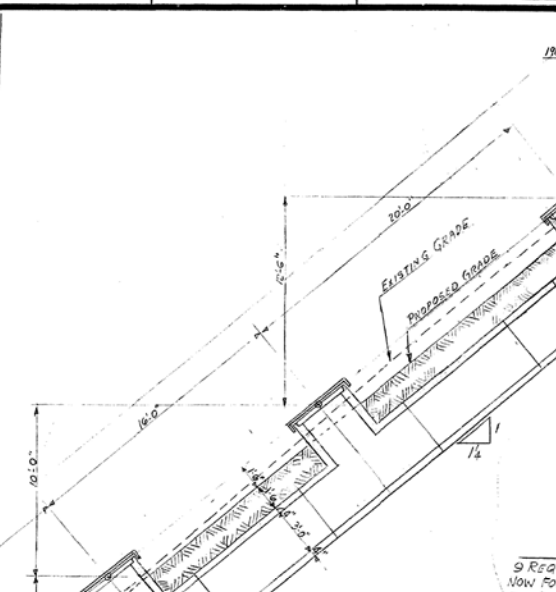
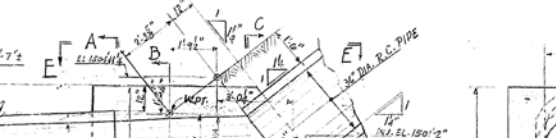
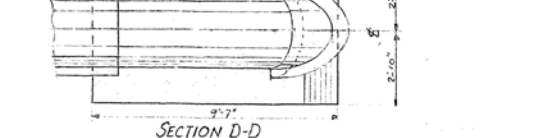
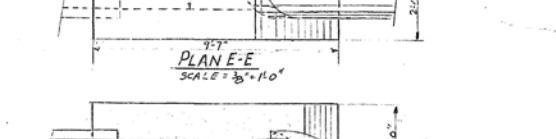
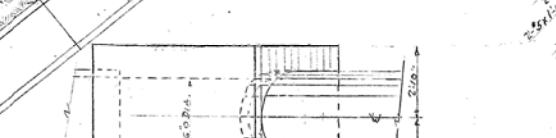
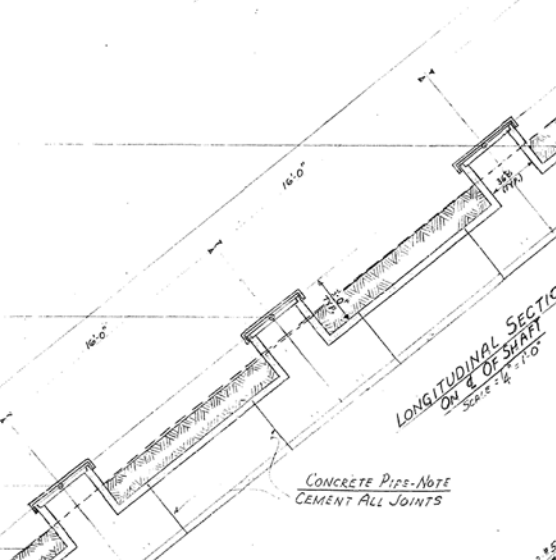
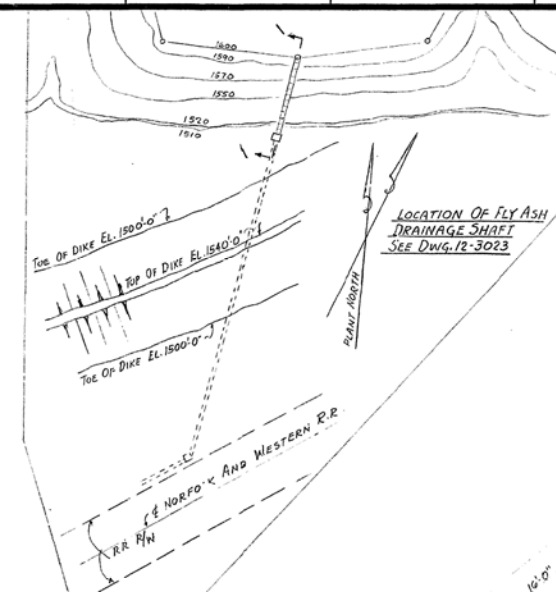
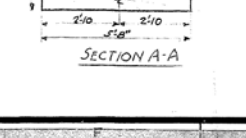
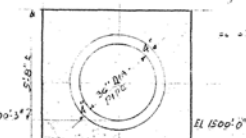
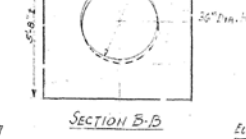
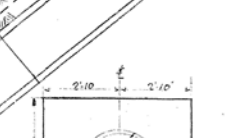
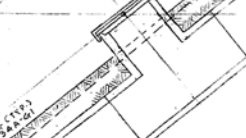
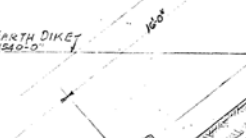
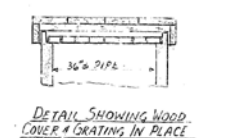
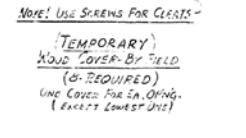
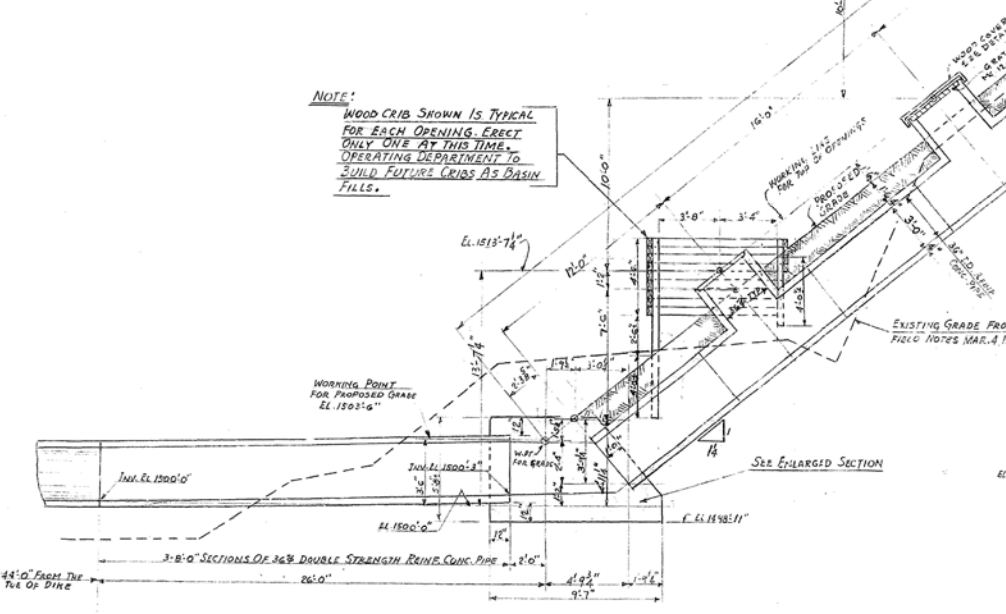


NOTE 1  
EACH OPENING IS TO HAVE A REMOVABLE WOODEN COVER AND A GRATING. ON THE LOWEST OPENING ONLY, TO INSURE DRAINAGE INTO THE SHAFT. AS THE FLY ASH BUILD UP TO THE RIM OF EACH OPENING, REPLACE THE WOOD COVER WITH A STAINLESS STEEL CIRCULAR PLATE OVER THE GRATING. SEE DETAIL B.

SEE NOTE BELOW RE: CRIB



NOTE:  
WOOD CRIB SHOWN IS TYPICAL FOR EACH OPENING. ERRECT ONLY ONE AT THIS TIME. OPERATING DEPARTMENT TO BUILD FUTURE CRIBS AS BASIN FILLS.



GENERAL NOTES  
ALL DIMENSIONS UNLESS OTHERWISE SPECIFIED SHALL CONFORM TO THE A.C.I. 308.1 SPECIFICATIONS FOR DIMENSIONS GIVEN FOR REINFORCING STEEL ARE CENTER LINE OF BAR.  
CONSTRUCTION NOTES MAY BE NOTED WITH THE APPROVAL OF THE ENGINEERING DEPARTMENT. PROVIDE THAT THE CONTRACTOR'S MEASUREMENTS AND PLACING SHALL BE PROPERLY DONE SO THAT THE SHAFT SHALL BE PROPERLY CONSTRUCTED.  
CONSTRUCTION JOINTS SHALL NOT BE LOCATED AT THE TOP OF THE SHAFT.  
ALL EXPOSED REINFORCING SHALL HAVE A ONE INCH BEVEL FLARE WITH SMALL CONFORM TO THE A.C.I. 308.1 SPEC. TYPE, AND SHALL BE PLACED WITHIN THE SHAFT.  
ALL EXPOSED CONCRETE SURFACES TO HAVE RUBBER PAINT.  
ALL JOINTS IN REINFORCING TO BE CEMENTED PAINT. ALL TIMBER TO RECEIVE TWO COATS OF CREOSOTE PAINT BY FIELD.

CONCRETE 12 CY. YOS. BY SOLLITT CONST CO  
TIMBER BY FIELD  
36" EXTRA STRENGTH REINFORCING CONCRETE TONGUE & GROOVE PIPE & FITTINGS BY FIELD  
REINFORCING BARS BY FIELD  
1" STAINLESS STEEL GRATING BY J. T. CYRUS & SON, CHICAGO, ILL. 60643  
METAL DISC BY FIELD  
1" CIRCULAR PLATE STAINLESS STEEL TYPE 304 (1/4" THICK - 3/16" DIA.) BY J. T. CYRUS & SON, CHICAGO, ILL. 60643

REFERENCE DWGS.  
12-3023 RELOCATION DUMPS CREEK

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12-3023 RELOCATION DUMPS CREEK





EXTENSION OF DIKE FROM EL 1525.0  
TO EL 1535.0 TO BE CHARGED TO  
C.I.A #10885.

EXTENSION OF DIKE FROM EL 1535.0'  
TO EL 1545.0' TO BE CHARGED TO  
C.I.A #10852.

EXTENSION OF DIKE FROM EL 1545.0 TO  
EL 1550.0 TO BE CHARGED TO C.I.A #10857.

RAINFALLS & FIRE  
BY: RICHARD SPENCER CORP  
CALL NO. 06090-021-0  
2" (16GA) PREPARED 16 ROWS OF HXRS  
INTERNAL COATED RAVED INVERT  
CORR. PIPE 2000' MIN AT  
MANHOLLS B REQ

\* EXISTING PIEZOMETER LOCATIONS.  
 ⊗ NEW PIEZOMETER LOCATIONS.

NOTE:  
PIEZOMETERS P2 THRU P9 TO BE ADDED  
LATER CONSTRUCTION OF DIKE.

-AP-2666-3108 UNITS 142 ASH DIRT RELOCATION  
OF DUMPS CREEK & STATE HWY. 616

"THIS DRAWING IS THE PROPERTY OF THE AMERICAN ELECTRIC POWER SERVICE CORP. AND IS LOANED UPON CONDITION THAT IT IS NOT TO BE REPRODUCED OR COPIED, IN WHOLE OR IN PART, OR USED FOR FURNISHING INFORMATION TO ANY PERSON WITHOUT THE WRITTEN CONSENT OF THE AEP SERVICE CORP., OR FOR ANY PURPOSE DETRIMENTAL TO THEIR INTEREST, AND IS TO BE RETURNED UPON REQUEST."

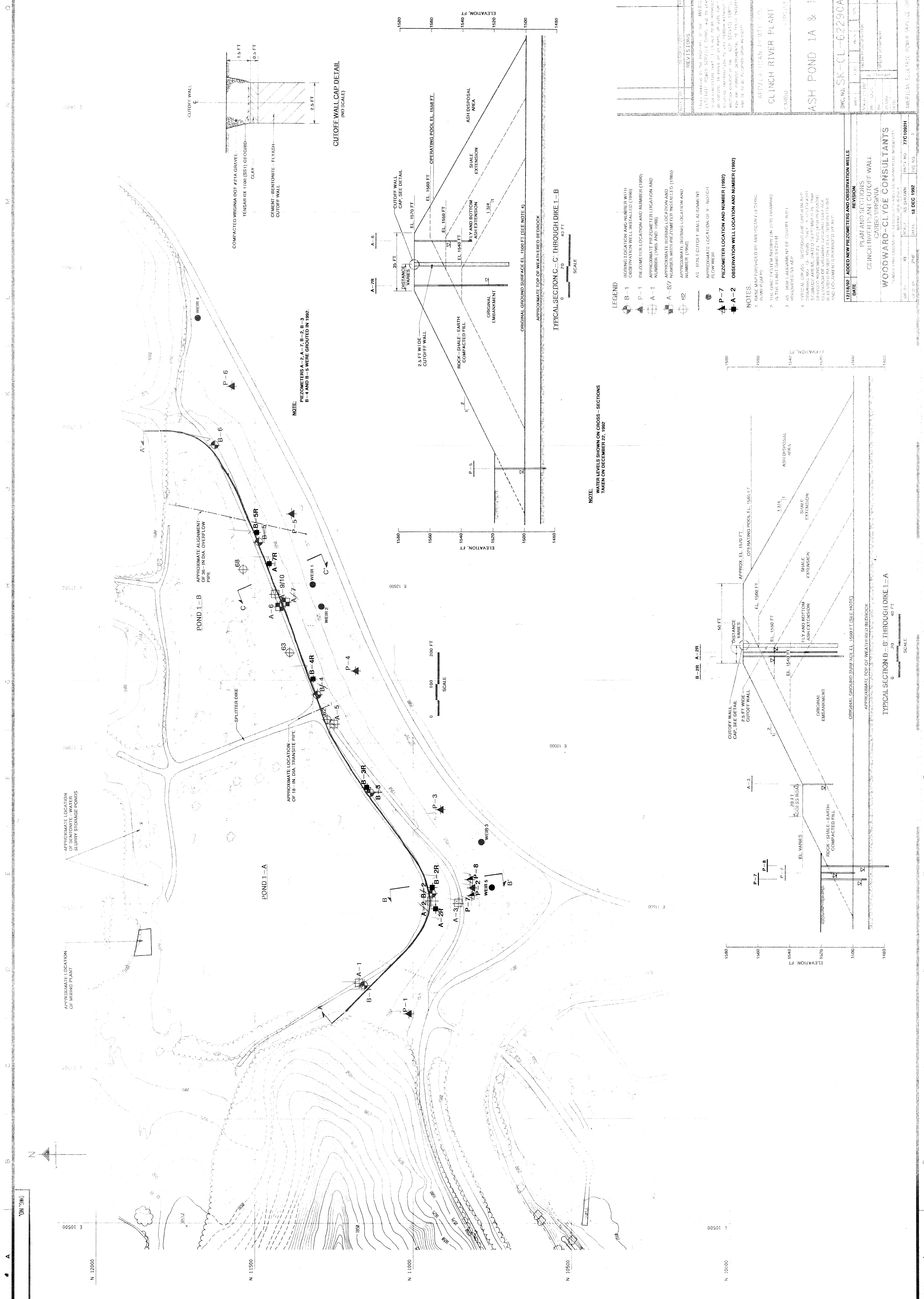
ASH STORAGE AREAS

DR. NO. 13-10585-3107-4

ARCH.	ELEC.	MECH.	STR.
SCALE: 1"=100'- AS NOTED	ENGINEERING DIV HET J. R. P. [Signature]		J S
DR. J. M.	DESIGN DIV [Signature]		
CH. I. B.			
50 LOR. M.E.			
DATE: 9/19/72	APPROVED BY:		

AMERICAN ELECTRIC POWER SERVICE CORP.  
2 BROADWAY NEW YORK

WORK THIS DRAWING WITH DWG. 13-10796-3107A



NOTE:  
WATER LEVELS SHOWN ON CROSS-SECTIONS  
TAKEN ON DECEMBER 22, 1992

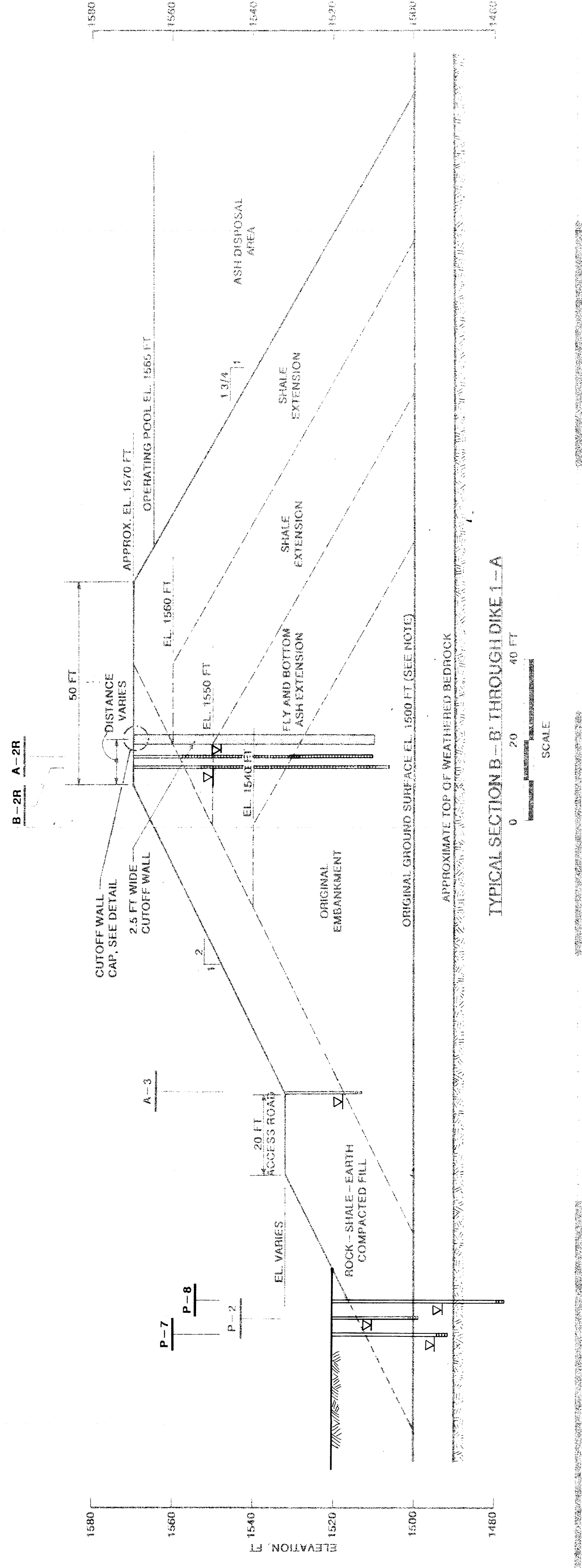
- LEGEND
- B-1 BORING LOCATION AND NUMBER WITH OBSERVATION WELL INSTALLED (1990)
  - P-1 PIEZOMETER LOCATION AND NUMBER (1990)
  - A-1 APPROXIMATE PIEZOMETER LOCATION AND NUMBER (1965 AND 1980)
  - A-67 APPROXIMATE BORING LOCATION AND NUMBER WITH PIEZOMETER INSTALLED (1965)
  - B2 APPROXIMATE BORING LOCATION AND NUMBER (1965)
  - AS BUILT CUTOFF WALL ALIGNMENT FLOWWEIR
  - P-7 PIEZOMETER LOCATION AND NUMBER (1992)
  - A-2 OBSERVATION WELL LOCATION AND NUMBER (1992)

- NOTES:
1. BASE MAP PROVIDED BY AMERICAN ELECTRIC POWER (AEP).
  2. THE GRID SYSTEM SHOWN ON THIS DRAWING IS THE PLANT GRID SYSTEM.
  3. AS BUILT ALIGNMENT OF CUTOFF WALL APPROXIMATE BY AEP.
  4. TYPICAL CROSS-SECTIONS ARE SHOWN IN THIS DRAWING. STORAGE AREAS, DATED SEPTEMBER 19, 1989, REVISED NOVEMBER 21, 1992 (IN PROVISION), IS BASED ON FIELD ASH ANALYSES AND DOCUMENTS PROVIDED BY AEP.

REVISIONS		DATE	BY	CHKD	APP'D

APPALACHIAN POWER CO.	
CLINCH RIVER PLANT	
CASCO	ASH POND 1A & 1B
DWC NO. SK-CL-62290A	
DATE	12/18/92
BY	AS SHOWN
CHKD	77C1029H
APP'D	18 DEC 1992

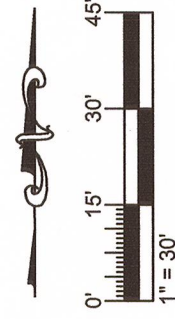
PLAN AND SECTIONS	
CLINCH RIVER PLANT CUTOFF WALL	
CARBON VIRGINIA	
WOODWARD-CLYDE CONSULTANTS	
CONCEPT ENGINEER	GEORGE A. WOODWARD
SCALE	AS SHOWN
DATE	18 DEC 1992



TYPICAL SECTION B-B' THROUGH DIKE 1-A



1. ALL EXCAVATIONS SHALL BE MADE TO GRADES SHOWN. TAKE MEASURES TO PREVENT CAVE-IN OF THE EXCAVATION. THESE MEASURES MAY INCLUDE CUTTING BACK THE SIDE SLORES AND/OR THE USE OF SHIELDING.
2. ANY SOFT AREAS OR AREAS SUSPECTED OF POOR MATERIALS SHALL BE REMOVED TO A DEPTH OF NOT LESS THAN 24 INCHES BELOW THE UNSUITABLE MATERIAL, AND REPLACED WITH SUITABLE BACKFILL MATERIAL IN THE DIRECTION OF THE OWNERS ENGINEER.
3. ALL BACKFILL MATERIAL SHALL BE COMPACTED TO 98% OF THE STANDARD PROCTOR, UNLESS OTHERWISE NOTED.
4. ALL PREZUMPTERS AND MONITORING WILL BE TO BE RE-TESTED (SUCH PRIOR TO) INER CONSTRUCTION.



THIS DRAWING IS CLASSIFIED AS:

REFERENCE KEYS CORPORATE INFORMATION SECURITY POLICY

APPALACHIAN POWER COMPANY  
CLINCH RIVER PLANT

CARBO

VIRGINIA

GRADING PLAN

POND 1A1B DIVERSION CHANNEL

CARBO, RUSSELL COUNTY, VIRGINIA

UNIT:	DRAWING NUMBER:	REV:
	13-30235-02-A	

SCALE GRAPHIC	APPROVED BY		 <b>AEP</b> <b>AMERICAN</b> <b>ELECTRIC</b> <b>POWER</b>	<b>AEP SERVICE CORP.</b> <b>1 RIVERSIDE PLAZA</b> <b>COLUMBIUS, OH 43215</b>
	DR. AT			
	CH. CB			
	SUP. BO			
	ENG. LV			
DATE: 06-07-20013				

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EMEC Project # 3050130220.2



