APPENDIX D
Universal Soil Loss Demonstration

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SUBJECT Dominion – Chesterfield Upper (East) Pond Closure – Final Cover Soil Loss

BY BUTLEKM DATE 07/13/15 PROJ. NO. C150035.00

CHKD. BY KMB DATE 12/10/2015 SHEET NO. 1 OF 3

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OBJECTIVE

The purpose of this calculation is to estimate the rate of soil loss that would take place on the final soil cover of the closed Upper (East) Pond area. The soil loss should be less than 2.0 tons/acre/year (VDEQ Solid Waste Requirement for Post-Closure).

METHODOLOGY

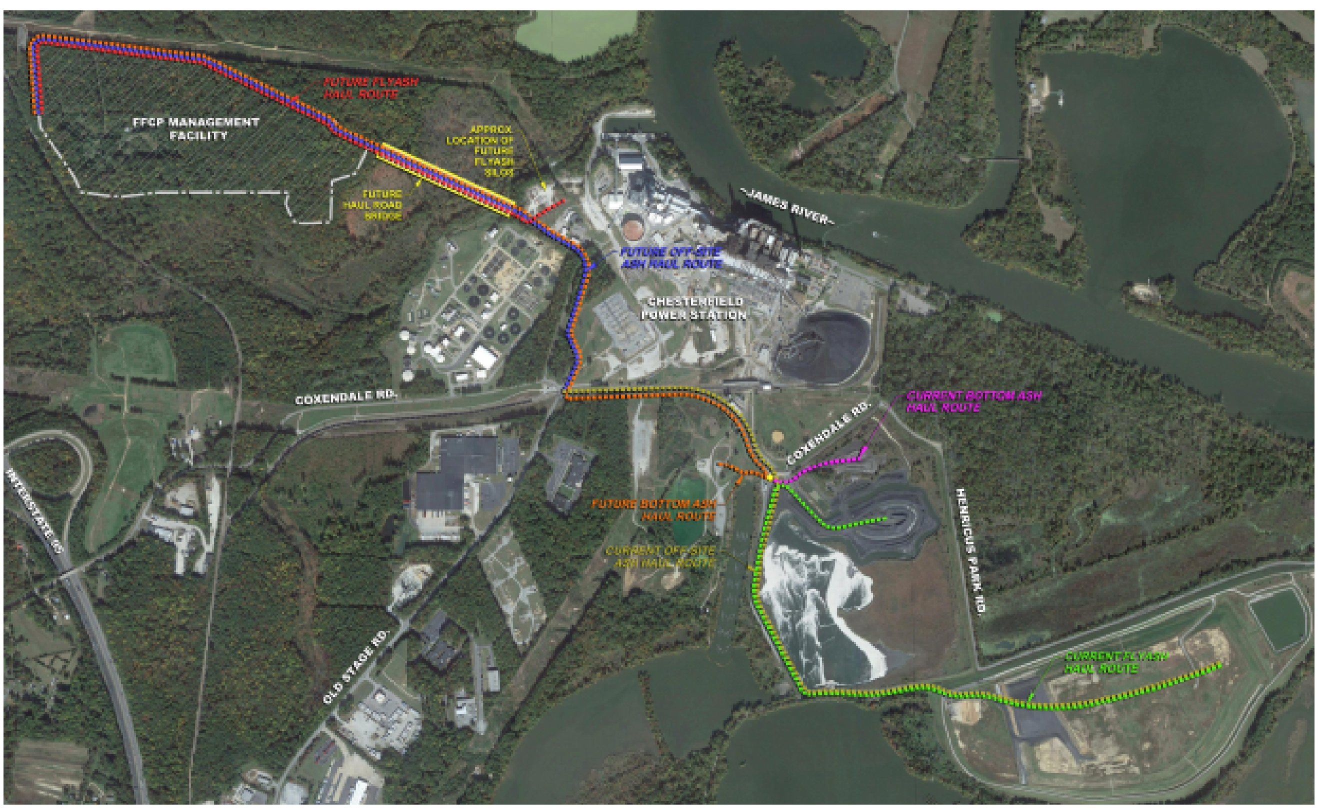
The anticipated soil loss to the final cover system was evaluated using several areas of the final soil cover that were anticipated to yield the greatest soil loss rates. These soil loss evaluations were analyzed using the Revised Universal Soil Loss Equation (RUSLE).

REFERENCES

1. "Web Soil Survey" http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx [Accessed: 7/10/2015].

BACKGROUND

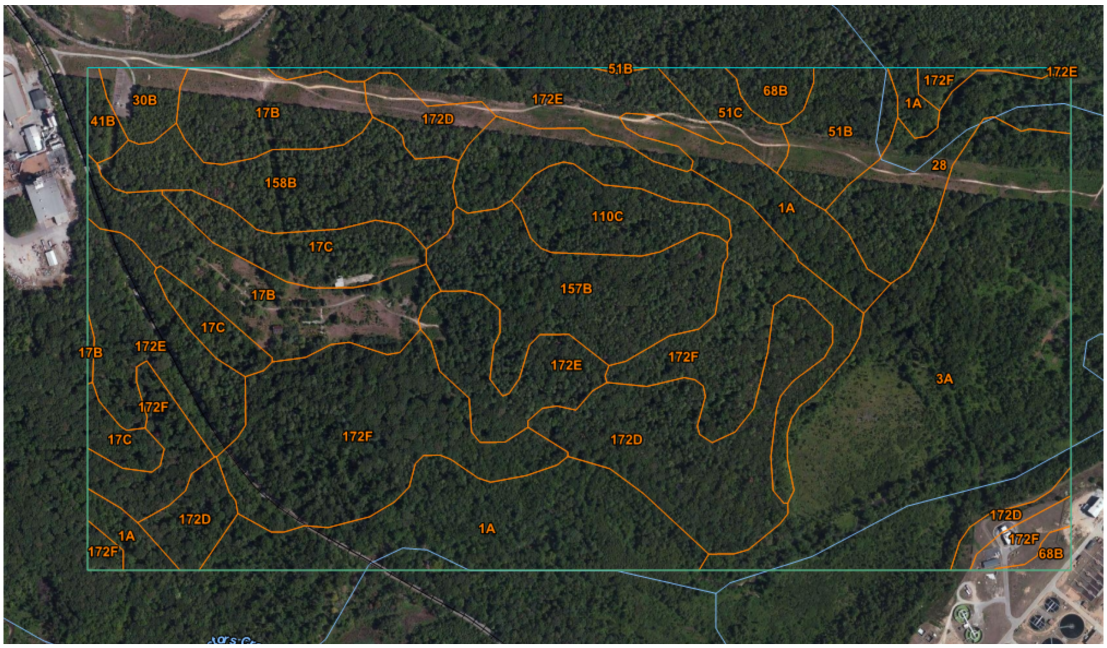
The existing Upper (East) Pond at the Chesterfield Power Station is to be closed using a geosynthetic cap system overlaid with two feet of soil. As part of this project, Dominion has decided that the Power Station will convert to a dry disposal system, and soil from the proposed landfill area was considered as the cover soil material in question for the soil loss calculation.



OVERALL SITE (GOLDER ASSOCIATES, MAY 2015)

REVISED UNIVERSAL SOIL LOSS EQUATION (RUSLE)

The computer program RUSLE2 was used to determine the soil loss from the closed landfill conditions. The USDA Web Soil Survey was used to obtain the site specific soil classifications of the proposed landfill. Based on the numerous soil types in the area of interest, only the types that comprised the majority of the area were considered.



Web Soil Survey Map of Borrow Area with Soil Classes

Chesterfield County, Virginia (VA041)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
1A	Fluvaquents	27.9	10.8%
3A	Fluvaquents, ponded	46.5	18.0%
17B	Gritney fine sandy loam, 2 to 6 percent slopes	19.7	7.6%
17C	Gritney fine sandy loam, 6 to 12 percent slopes	10.1	3.9%
28	Chewacla loam	9.7	3.7%
30B	Lenoir silt loam, 0 to 4 percent slopes	2.5	1.0%
41B	Craven fine sandy loam, 2 to 6 percent slopes	1.2	0.5%
51B	Pamunkey loam, 0 to 6 percent slopes	6.1	2.4%
51C	Pamunkey loam, 6 to 12 percent slopes	2.9	1.1%
68B	Dogue loam, variant, 0 to 4 percent slopes	2.6	1.0%
110C	Faceville-Gritney gravelly fine sandy loams, 6 to 12 percent slopes	6.6	2.5%
157B	Faceville-Gritney fine sandy loams, 2 to 6 percent slopes	14.9	5.8%
158B	Tetotum loam, clayey substratum, 2 to 6 percent slopes	12.7	4.9%
172D	Ochrepts and Udults, sloping	23.1	8.9%
172E	Ochrepts and Udults, strongly sloping	26.0	10.0%
172F	Ochrepts and Udults, steep	46.1	17.8%
Totals for Area of Interest		**258.6**	**100.0%**

Area Distributions based on Soil Group Classification

Two conditions were analyzed for each of the six soils (outlined in red in the table on the previous page) in question, which represent the two extreme cases. These include a longer reach with a shallower slope, found at the top of the proposed pile, and a shorter reach with a steeper slope, found along the proposed benching.

Plan: Chesterfield Upper Ash Pond Closure Loss Page 1

Cons. plan. soil loss, t/ac/yr
0.012 0.0092 0.011 0.0077 0.0077 0.0077 0.092 0.073 0.085 0.061 0.061 0.061
Description

Field
Worksheet Worksheet Worksheet Worksheet Worksheet Worksheet Worksheet Worksheet Worksheet Worksheet
Worksheet Worksheet
Fuel cost, US$/ac
0 0 0 0 0 0 0 0 0 0 0 0
Info
-
Location
Virginia\Henrico county average (Richmond)
Project Name
Owner name
Section or reach
1A Top 3A Top 17B Top 172D Top 172E Top 172F Top 1A Slope 3A Slope 17B Slope
172D Slope 172E Slope 172F Slope
Section or reach
1A Top 3A Top 17B Top 172D Top 172E Top 172F Top 1A Slope 3A Slope 17B Slope
172D Slope 172E Slope 172F Slope
Sed. delivery, t/ac/yr
0.012 0.0092 0.011 0.0077 0.0077 0.0077 0.092 0.073 0.085 0.061 0.061 0.061
Slope T Value, t/ac/yr
5.0 4.0 5.0 5.0 5.0 5.0 5.0 4.0 5.0 5.0 5.0 5.0
Slope length, ft
1000 1000 1000 1000 1000 1000 100 100 100 100 100 100
Slope steepness, %
2.0 2.0 2.0 2.0 2.0 2.0 33 33 33 33 33 33
Soil
soils\Chesterfield County, Virginia\1A Fluvaquents\Fluvaquents Silt loam 85%
soils\Chesterfield County, Virginia\3A Fluvaquents, ponded\Fluvaquents, ponded Fine sandy loam 85%
soils\Chesterfield County, Virginia\17B Gritney fine sandy loam, 2 to 6 percent slopes\Gritney Fine sandy loam 85%
soils\Chesterfield County, Virginia\172D Ochrepts and Udults, sloping\Ochrepts Sandy loam 45%
soils\Chesterfield County, Virginia\172E Ochrepts and Udults, strongly sloping\Ochrepts Sandy loam 45%
soils\Chesterfield County, Virginia\172F Ochrepts and Udults, steep\Ochrepts Sandy loam 50%
soils\Chesterfield County, Virginia\1A Fluvaquents\Fluvaquents Silt loam 85%
soils\Chesterfield County, Virginia\3A Fluvaquents, ponded\Fluvaquents, ponded Fine sandy loam 85%
soils\Chesterfield County, Virginia\17B Gritney fine sandy loam, 2 to 6 percent slopes\Gritney Fine sandy loam 85%
soils\Chesterfield County, Virginia\172D Ochrepts and Udults, sloping\Ochrepts Sandy loam 45%
soils\Chesterfield County, Virginia\172E Ochrepts and Udults, strongly sloping\Ochrepts Sandy loam 45%
soils\Chesterfield County, Virginia\172F Ochrepts and Udults, steep\Ochrepts Sandy loam 50%

Soil Type	Soil Loss (t/ac/yr)	
	Top of Pile	Bench
1A	0.012	0.092
3A	0.0092	0.073
17B	0.011	0.085
172D	0.0077	0.061
172E	0.0077	0.061
172F	0.0077	0.061

The average expected soil loss for the final cover soil on the Upper Pond is less than 2 tons/acre/year. Overall, the final cover is expected to withstand erosion losses.

APPENDIX E
Stability Calculations

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This appendix contains the following geotechnical calculations:

- Veneer Stability Analysis (pages 1-24)
- Deep Seated Stability Analysis (pages 25-32)
- Anchor Trench Design (pages 33-40)
- GDN Flows (pages 41-45)
- GDN Design (pages 46-54)

SUBJECT Closure of Upper (East) Pond - Veneer Stability Analyses

BY TIM DATE 5/5/2015 PROJ. NO. C150035.00

CHKD. BY CAG DATE 9/25/2015 SHEET NO. 1 OF 24

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OBJECTIVE:

Determine the factor of safety against failure for translational failures (veneer stability) of the cap system for the proposed closure of the Upper (East) Pond of the Chesterfield Station located in Chesterfield County, Virginia. The Closure will be evaluated for compliance with the EPA CCR Rule and Virginia Solid Waste Regulations.

METHODOLOGY:

To quantify the risk of translational failures of the proposed cap system as a factor of safety (FS) against sliding, the existing conditions and proposed top-of-liner plans were analyzed using finite slope methods and GSTABL 7 software. Material properties (unit weights and interface shear strength) were acquired from previous submittals, published values, or laboratory test data. The following sections provide a detailed description of the methods used to evaluate veneer stability of the proposed liner system.

REFERENCES:

1. Schnabel Engineering Consultants, Inc. *Geotechnical Engineering Report: Upper Pond Stability Evaluation,* August 2014.

2. Petersen, Mark, et al. Seismic-Hazard Maps for the Conterminous United States. 2014. United States Department of Interior. United States Geological Survey.

BACKGROUND:

This study provides slope stability analyses for the closure of the Upper (East) Pond. The cap will consist of a liner system that will include from bottom to top:

-Subgrade (Existing ground, regraded CCR material, or a cushion (nonwoven) geotextile);
-40 mil low-linear density polyethylene (LLDPE) geomembrane;
- Geocomposite Drainage Net (GDN) consisting of an HDPE geonet core with nonwoven, needle-punched geotextiles heat-bonded to its upper and lower surfaces; and
-24” of soil cover.

For stability and drainage purposes, benches on the 33% slopes of the pond will be constructed at a regular vertical spacing of every 25 feet around the perimeter of the pond. Each bench is 20-feet wide and will be sloped to direct runoff on the sideslopes to the slope drains.

A typical cross-section of the benches and proposed liner system are shown in Figure 1. GAI completed a long-term stability analysis of the cap system shown.

LINER SYSTEM ANALYSIS:

To determine the minimum FS against slope failure of the liner system components, the Upper Ash Pond Final Closure Drawing No. C150035-00-000-00-C-E1-030 was used to determine critical and typical slopes. From this plan, sideslopes are 3:1 (H:V) with the benches having a slope of 2%. A “typical” construction cross-section including the geosynthetic liner components was modeled.

SUBJECT Closure of Upper (East) Pond - Veneer Stability Analyses

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Sections were analyzed with typical conditions which assume average conditions (static-dry), as well as a design maximum rainfall (static-wet) and earthquake (seismic-dry). The peak shear strength was used for interface strength. As a check on the stability runs, the residual shear strength for the liner interface was used. The target factor of safety for all conditions with residual strength is 1.0. Target factors of safety for each analysis condition are enumerated as follows:

Condition Analyzed	Minimum FS
Static-Dry (typical)	1.5 (CCR rule)
Static-Wet (design storm)	1.1
Seismic-Dry (earthquake)	1.0 (CCR rule)
Residual shear strength	1.0

The critical interfaces considered in the liner system analyses are listed below:

1) Textured-LLDPE against Geocomposite Drainage Net (GDN) consisting of HPDE geonet core with needle-punched nonwoven geotextiles heat-bonded to its upper and lower surfaces;
2) Geotextile to soil.

Phreatic Surfaces – Based on the subsurface investigation performed by Schnabel, groundwater should not impact slope stability. Based on a review of the boring logs in Schnabel's report, groundwater elevation is approximately at sea level. The lowest elevations for the capped portions of the Upper (East) Pond will occur at the dike (elev. 40'). To help convey runoff water off the side slopes, benches will be sloped toward the slope drains and then perimeter channels will carry the water to VPDES Outfall 005.

Liner System Properties – Based on previous GAI experience and data from Agru America, it was determined that the critical interface of the liner system would be the soil to the non-woven geotextile.

Since interface friction is primarily derived from the normal loads placed on the materials comprising the liner system, it was necessary to determine the phi angles and cohesion for anticipated site conditions after construction of the liner system. Assuming protective cover material is placed using low ground pressure equipment, a normal load of 1,000 psf is representative of the final conditions. GAI received typical interface friction data from Agru America for interface friction strength between the soil and the non-woven geotextile. Assuming a cohesion (c) of zero psf, the interface friction angle (ϕ) for each interface was defined by the secant angle bounded by a line connecting the origin (0,0; x,y) and the shear strength value (τ) at a normal load of 1,000 psf (σ_n).

Table 1: Liner System Properties

Interface	c=c' (psf)	ϕ=ϕ' (Degrees)
Soil to non-woven geotextile	0	26.0 (peak), 17.0 (residual)

Based on the interface shear strength determined through laboratory tests, the interface most likely to fail under anticipated conditions is that of the soil to the non-woven geotextile. To simplify the model, the interface with the lowest shear strength (soil to non-woven geotextile) was used to model the geosynthetics as a composite layer (liner). To be conservative, the shear strength of this interface was used to represent the liner layer in this analysis.

Other Material Properties – Material properties for the CCR material were obtained from Schnabel's report from 2014. A summary of the shear strength parameters used in this analysis are provided in Table 2.

Table 2: Slope Stability Material Properties

Soil Type	Soil No.	γ_T (pcf)	γ_{Sat} (pcf)	c=c' (psf)	ϕ=ϕ' (Degrees)
Soil Cover	1	120	125	0	25
Liner	2	90	95	0	26
CCRs	3	93	98	0	28
Imperm	3(1)	150	155	0	50

(1)-In selected stability runs, "Imperm" layer is used instead of CCRs layer to present an accurate model.

To evaluate stability of the liner system, unsaturated, saturated, and seismic (0.075g) loading conditions were analyzed. Saturated conditions were developed using the capacity of the GDN. The calculation shows that with the current geometry (3H:1V slope, 75' long between benches) the proposed GDN will be able to pass the flow from the ½ PMF (Probable Maximum Flood) see Upper East Pond – PMP Precipitation Distributions. Thus, the phreatic surface on the slopes was modeled as if there was no water in the liner system. The same calculation was used to determine if the GDN on the benches would be able to pass the full PMF. While the calculation shows that the GDN on the benches should be able to pass the flow, the phreatic surface on the benches was modeled to the top of the soil cover to be conservative. Also, for the stability runs modeling the phreatic surface, the soil properties were edited. When a phreatic surface is entered into GSTABL, the program then assumes that everything under the phreatic surface is saturated, which is not the case for the cap system. Since the program showed that the slopes were stable in a static-dry condition, the liner system and everything below it were modeled as an impermeable layer. The unit weights and friction angles were raised to force the failure surface through the cover soil.

SUBJECT Closure of Upper (East) Pond - Veneer Stability Analyses

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Seismic Conditions – The existing facility is located in Chesterfield County, Virginia, which is an area of low seismic activity and risk. The peak horizontal ground acceleration at the proposed site (using a 2 percent probability of exceedance in 50 years) is approximately 0.075g. This acceleration was estimated using USGS mapping and prior stability runs performed by GAI and Schnabel and was used in GSTABL to perform a pseudo-static force evaluation of seismic stability.

RESULTS & SUMMARY:

Results of the liner system veneer stability analyses are included as Attachment 1 and are summarized below:

Table 3: 25-Foot Vertical Spacing Between Benches

File Name	Analysis Conditions	Failure Plane Analyzed	FSmin	Target FS
static-dry-peak	dry-static	Critical Liner Interface	1.6	1.5
static-dry-residual			1.3	1.0
bottom-dry-static-peak			1.5	1.5
bottom-dry-static-residual			1.3	1.0

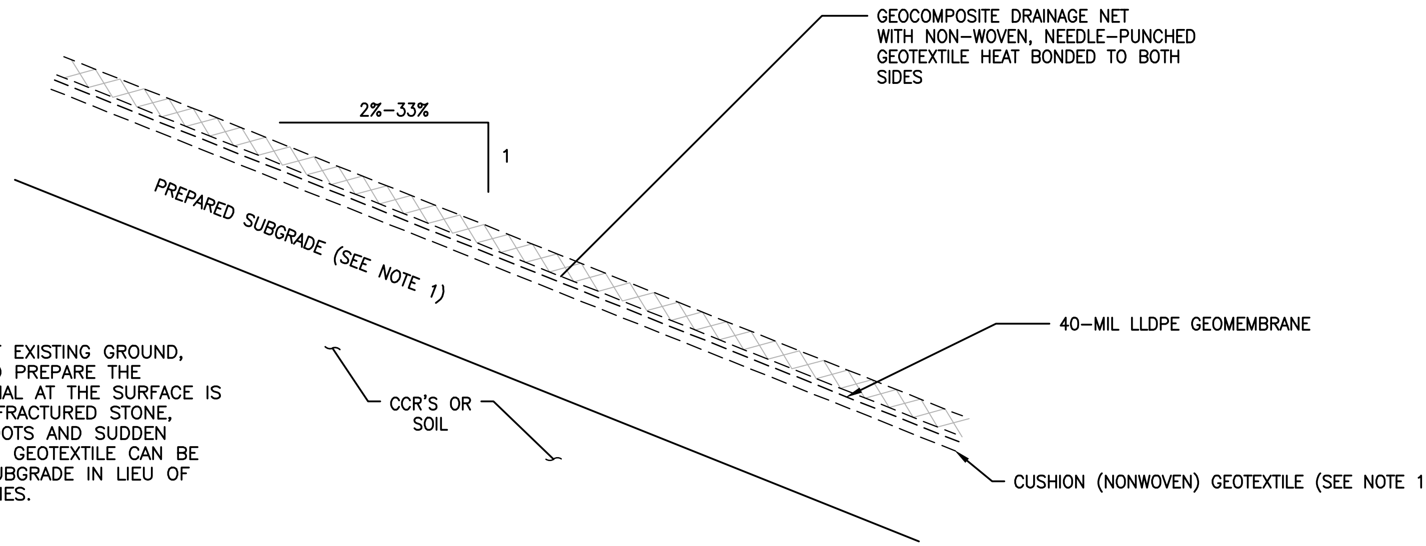
File Name	Analysis Conditions	Failure Plane Analyzed	FSmin	Target FS
Seismic-dry-peak	dry-seismic	Critical Liner Interface	1.3	1.0
seismic-dry-residual			1.0	1.0
bottom-seismic-dry-peak			1.2	1.0
bottom-seismic-dry-residual			1.0	1.0

File Name	Analysis Conditions	Failure Plane Analyzed	FSmin	Target FS
Wet-static-peak	Wet-static	Critical Liner Interface	1.5	1.1
Wet-static-residual			1.2	1.0
bottom-wet-static-peak			1.4	1.1
bottom-wet-static-residual			1.3	1.0

Based on these results, the cap system for the closure of the Upper (East) Pond cap system is stable as designed provided benches are included as designed at vertical spacing of 25 feet or less and the Protective Cover materials are placed as soon as practical after liner construction.

FIGURE 1

PROPOSED LINER CROSS SECTION AND BENCH GEOMETRY



NOTES:

1. WHERE SUBGRADE CONSISTS OF EXISTING GROUND, STRIP EXISTING VEGETATION AND PREPARE THE SUBGRADE SO THAT THE MATERIAL AT THE SURFACE IS FREE OF PROTRUDING ROCKS, FRACTURED STONE, DEBRIS, COBBLES, RUBBISH, ROOTS AND SUDDEN CHANGES IN SLOPE. A CUSHION GEOTEXTILE CAN BE PLACED OVER THE STRIPPED SUBGRADE IN LIEU OF SUBGRADE PREPARATION ACTIVITIES.

CAP SYSTEM DETAIL 1
N.T.S. 5,7,9,11,16,18,20,25,26,27,28,29

ATTACHMENT 1

SLOPE STABILITY OUTPUT FILES

8/24

Dominion Upper Pond static-dry-peak

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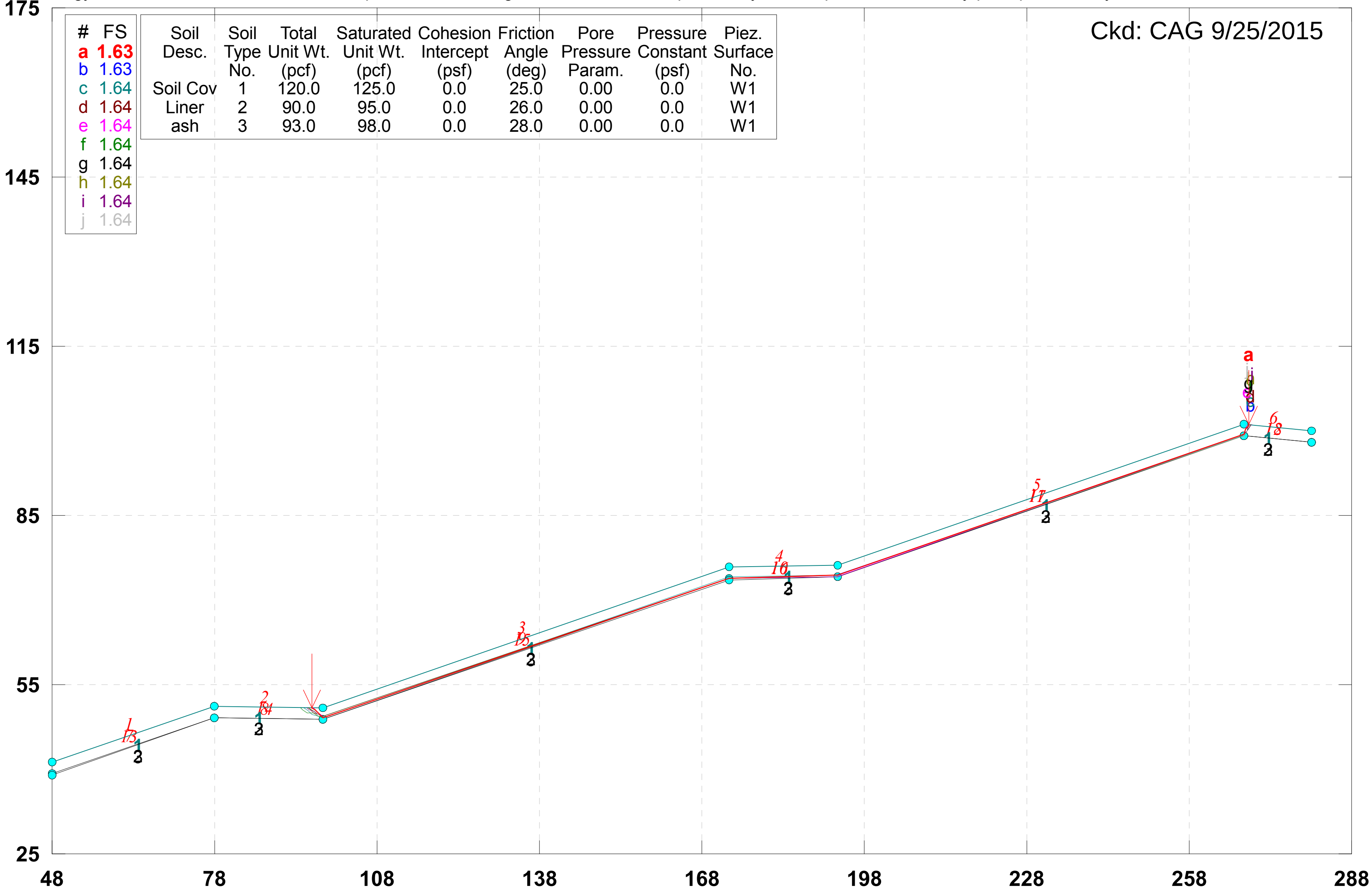

#	FS
a	1.63
b	1.63
c	1.64
d	1.64
e	1.64
f	1.64
g	1.64
h	1.64
i	1.64
j	1.64

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Soil Cov	1	120.0	125.0	0.0	25.0	0.00	0.0	W1
Liner	2	90.0	95.0	0.0	26.0	0.00	0.0	W1
ash	3	93.0	98.0	0.0	28.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=1.63

Safety Factors Are Calculated By The Simplified Janbu Method

Dominion Upper Pond static-dry-residual

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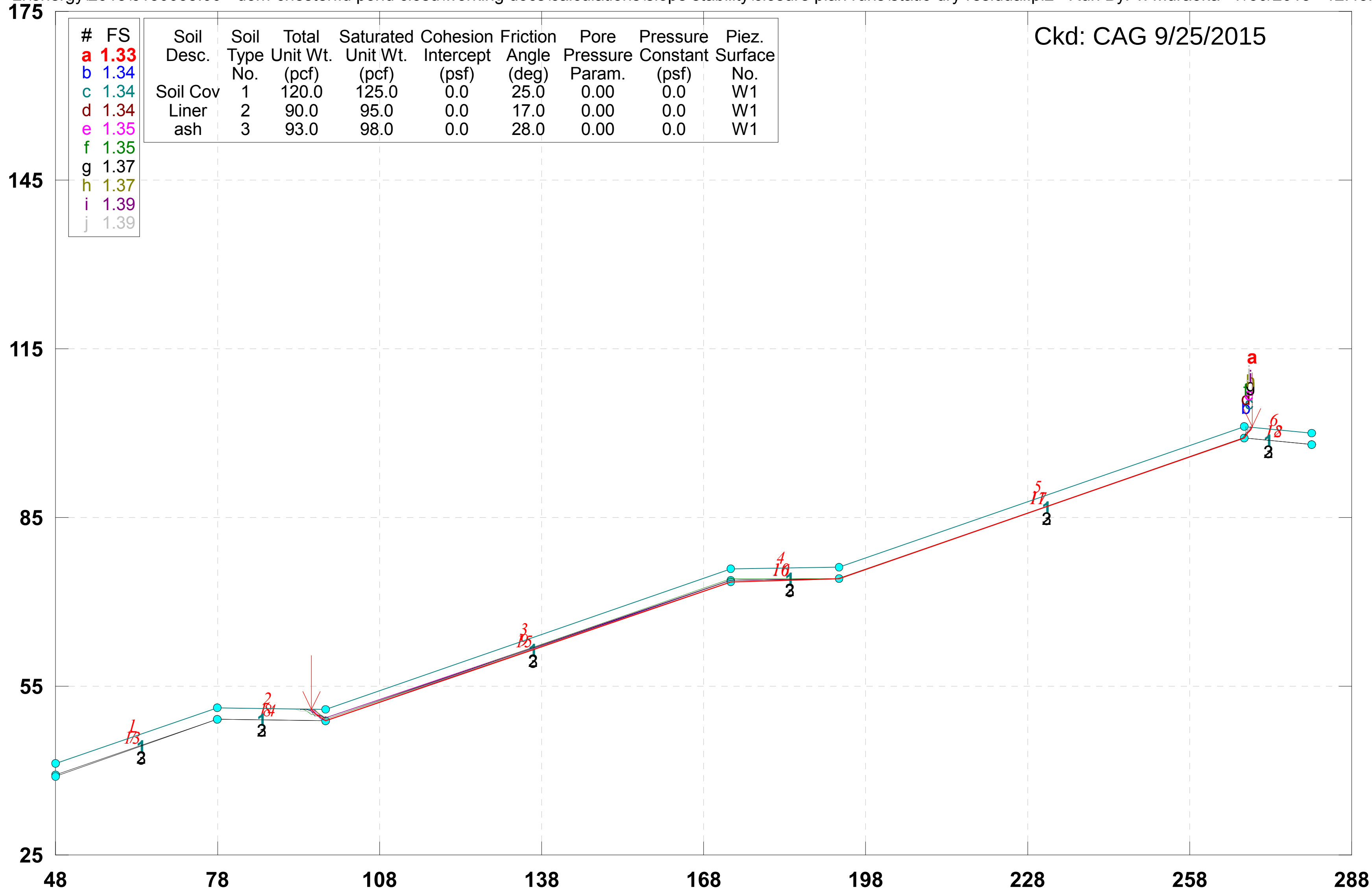

GSTABL7 v.2 FSmin=1.33
Safety Factors Are Calculated By The Simplified Janbu Method

Dominion Upper Pond seismic-dry-peak

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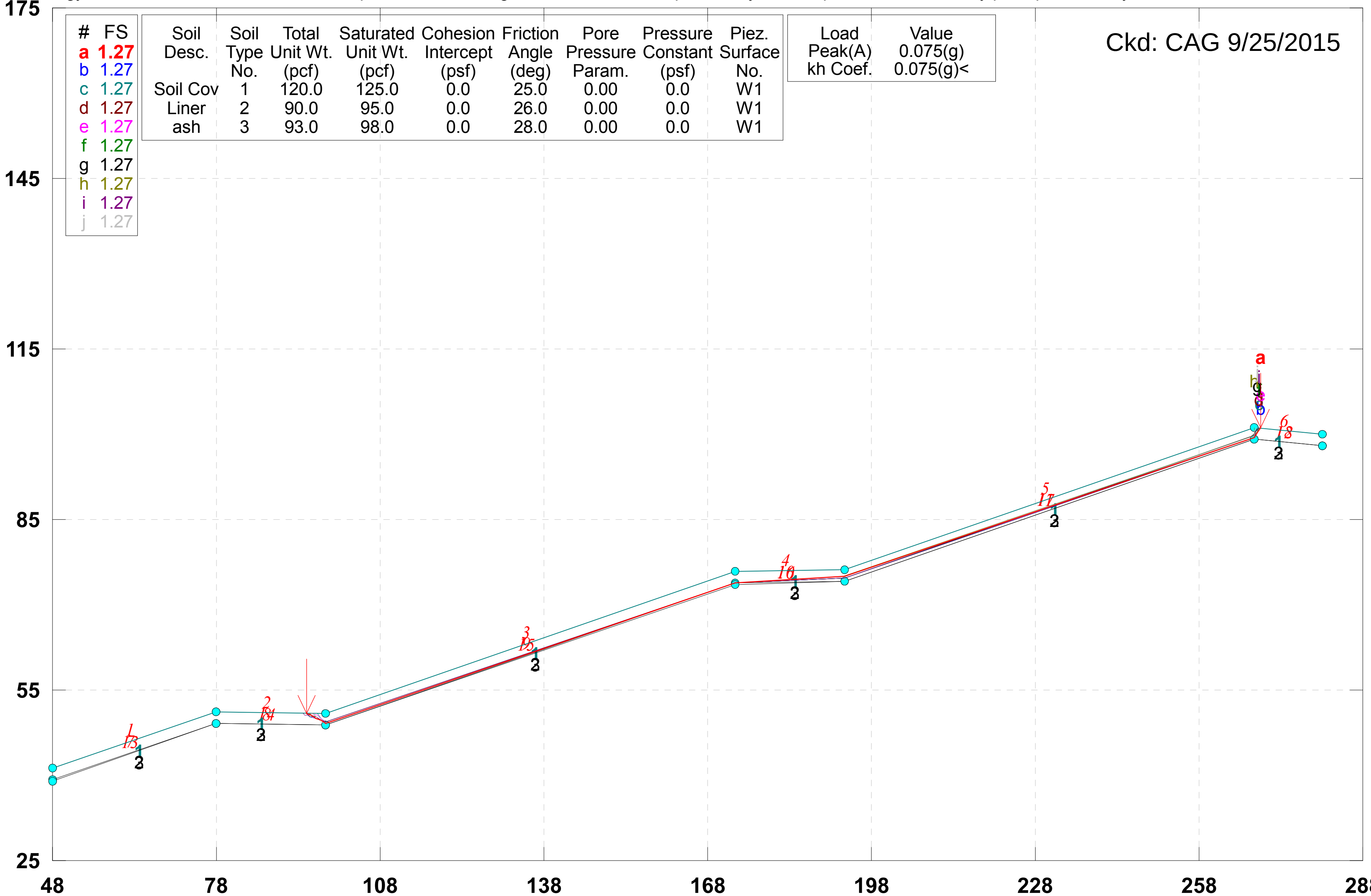

GSTABL7 v.2 FSmin=1.27

Safety Factors Are Calculated By The Simplified Janbu Method

Dominion Upper Pond seismic-dry-residual

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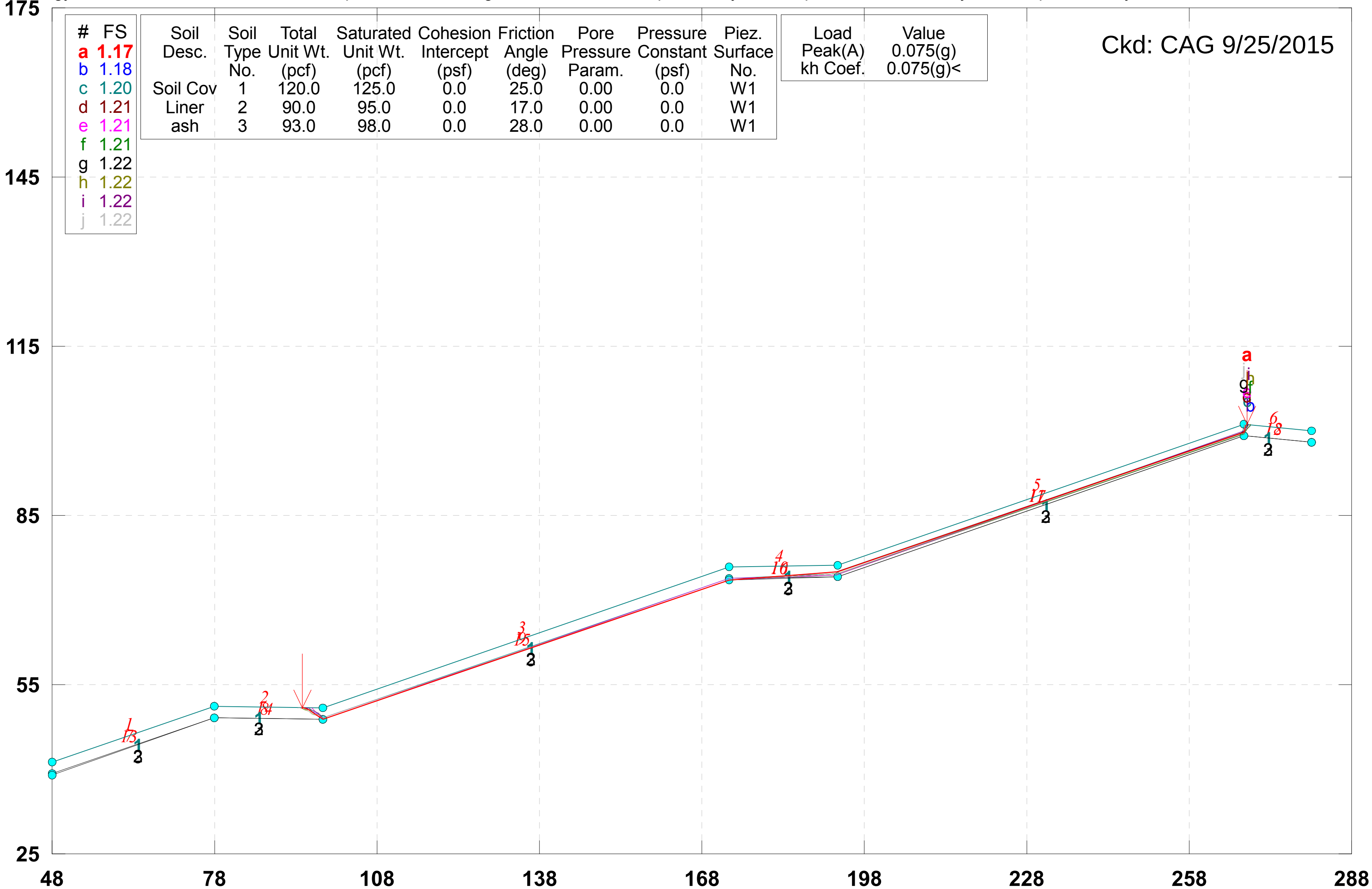

GSTABL7 v.2 FSmin=1.17

Safety Factors Are Calculated By The Simplified Janbu Method

Dominion Upper Pond bottom-dry-static-peak

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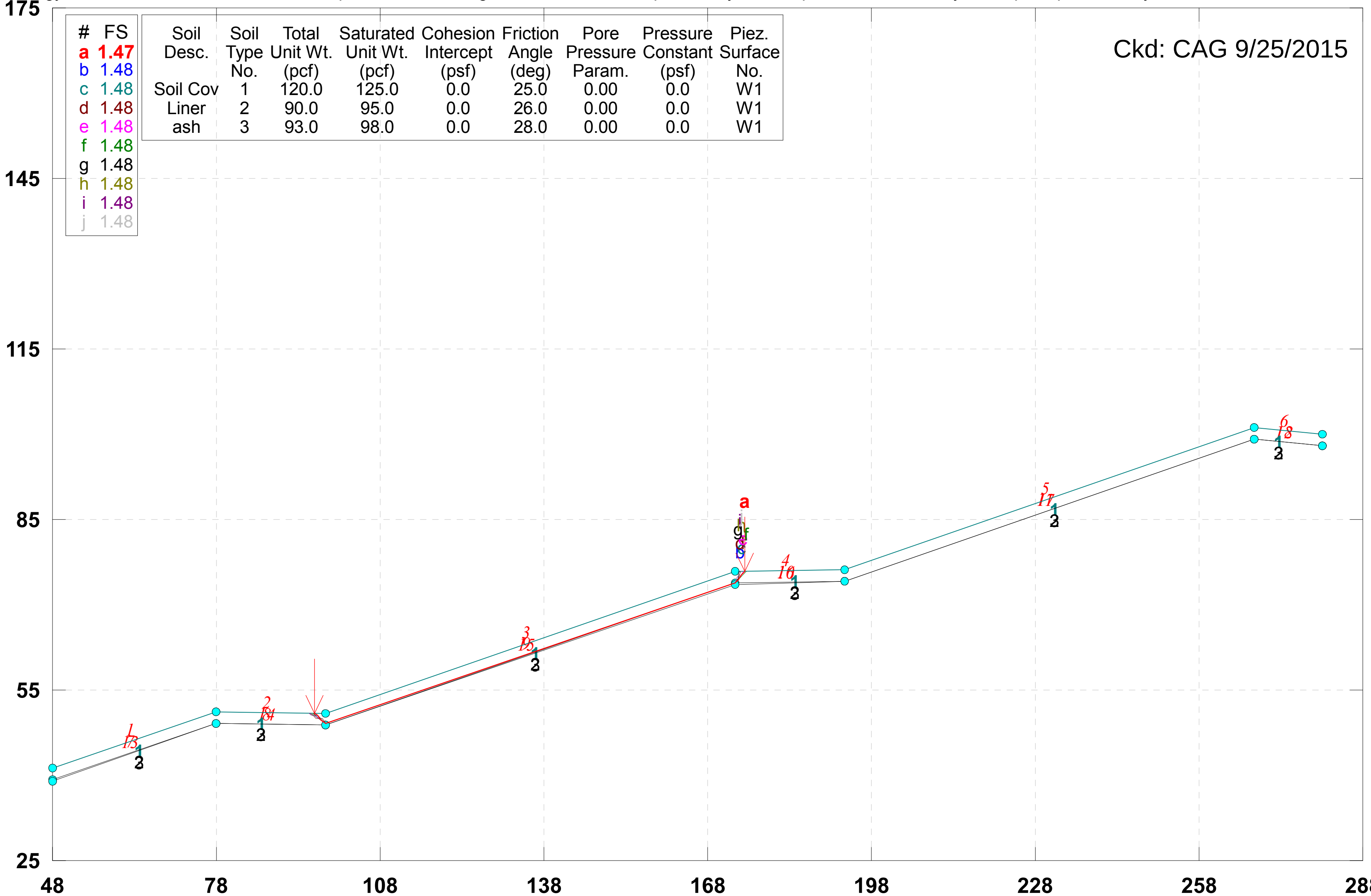

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Soil Cov	1	120.0	125.0	0.0	25.0	0.00	0.0	W1
Liner	2	90.0	95.0	0.0	26.0	0.00	0.0	W1
ash	3	93.0	98.0	0.0	28.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=1.47

Safety Factors Are Calculated By The Simplified Janbu Method

Chesterfield Upper Pond bottom-dry-static-residual

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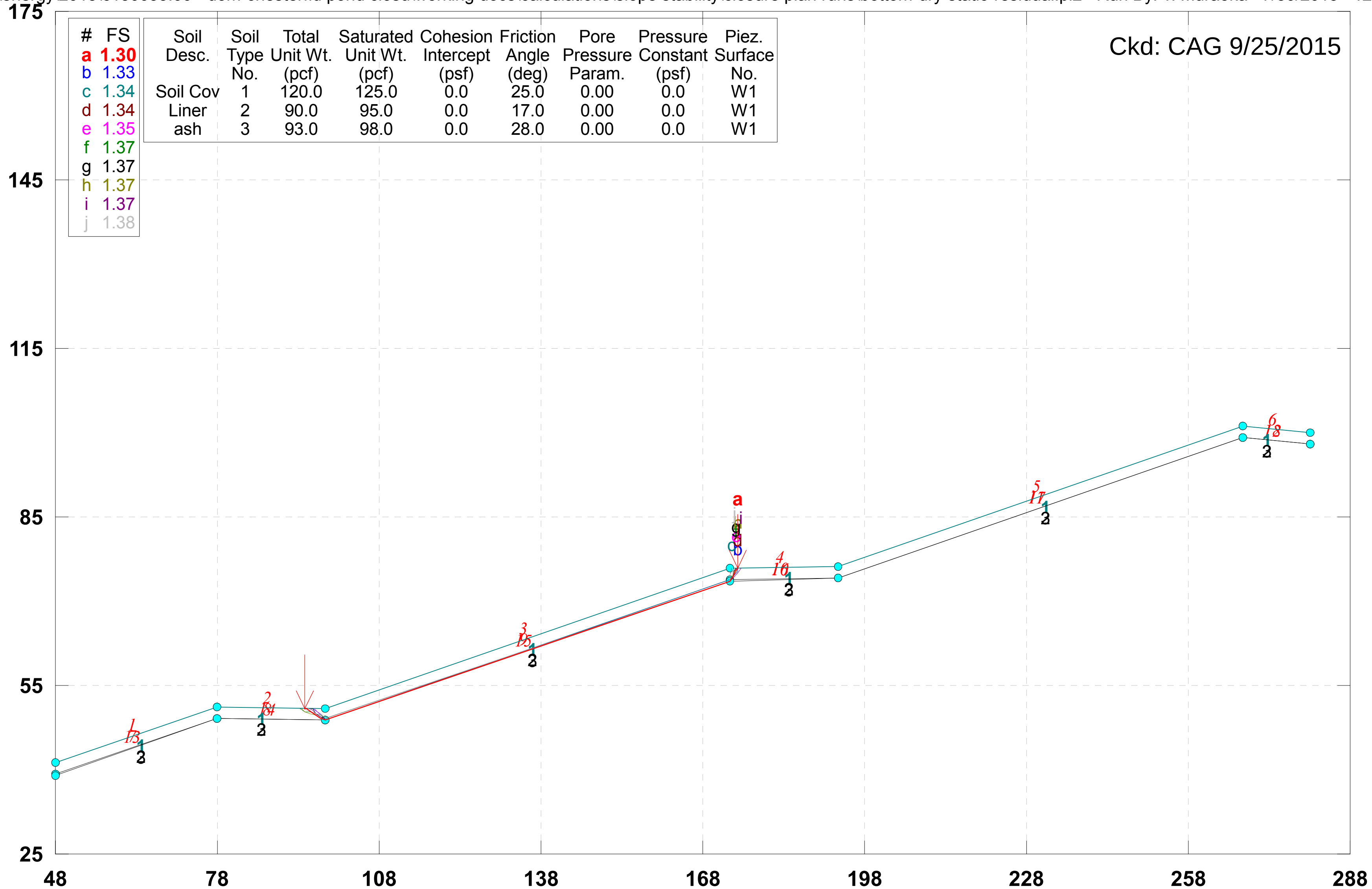

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
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Liner	2	90.0	95.0	0.0	17.0	0.00	0.0	W1
ash	3	93.0	98.0	0.0	28.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=1.30
Safety Factors Are Calculated By The Simplified Janbu Method

Dominion Upper Pond bottom-seismic-dry-peak

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#	FS
a	1.17
b	1.17
c	1.17
d	1.17
e	1.17
f	1.17
g	1.17
h	1.18
i	1.18
j	1.18

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
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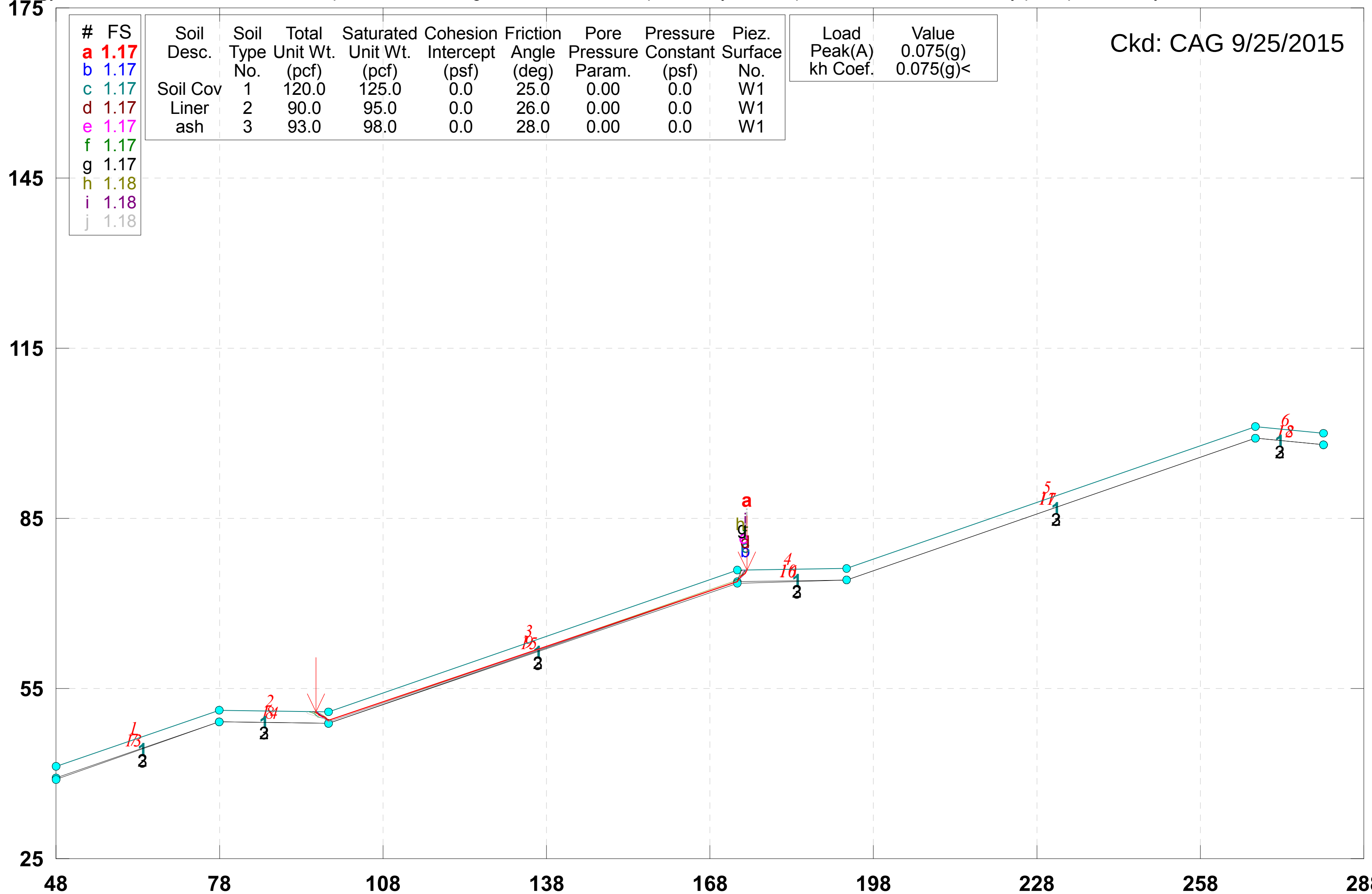
Liner	2	90.0	95.0	0.0	26.0	0.00	0.0	W1
ash	3	93.0	98.0	0.0	28.0	0.00	0.0	W1

Load	Value
Peak(A)	0.075(g)
kh Coef.	0.075(g)<

Ckd: CAG 9/25/2015



GSTABL7 v.2 FSmin=1.17

Safety Factors Are Calculated By The Simplified Janbu Method

Dominion Upper Pond bottom-seismic-dry-residual

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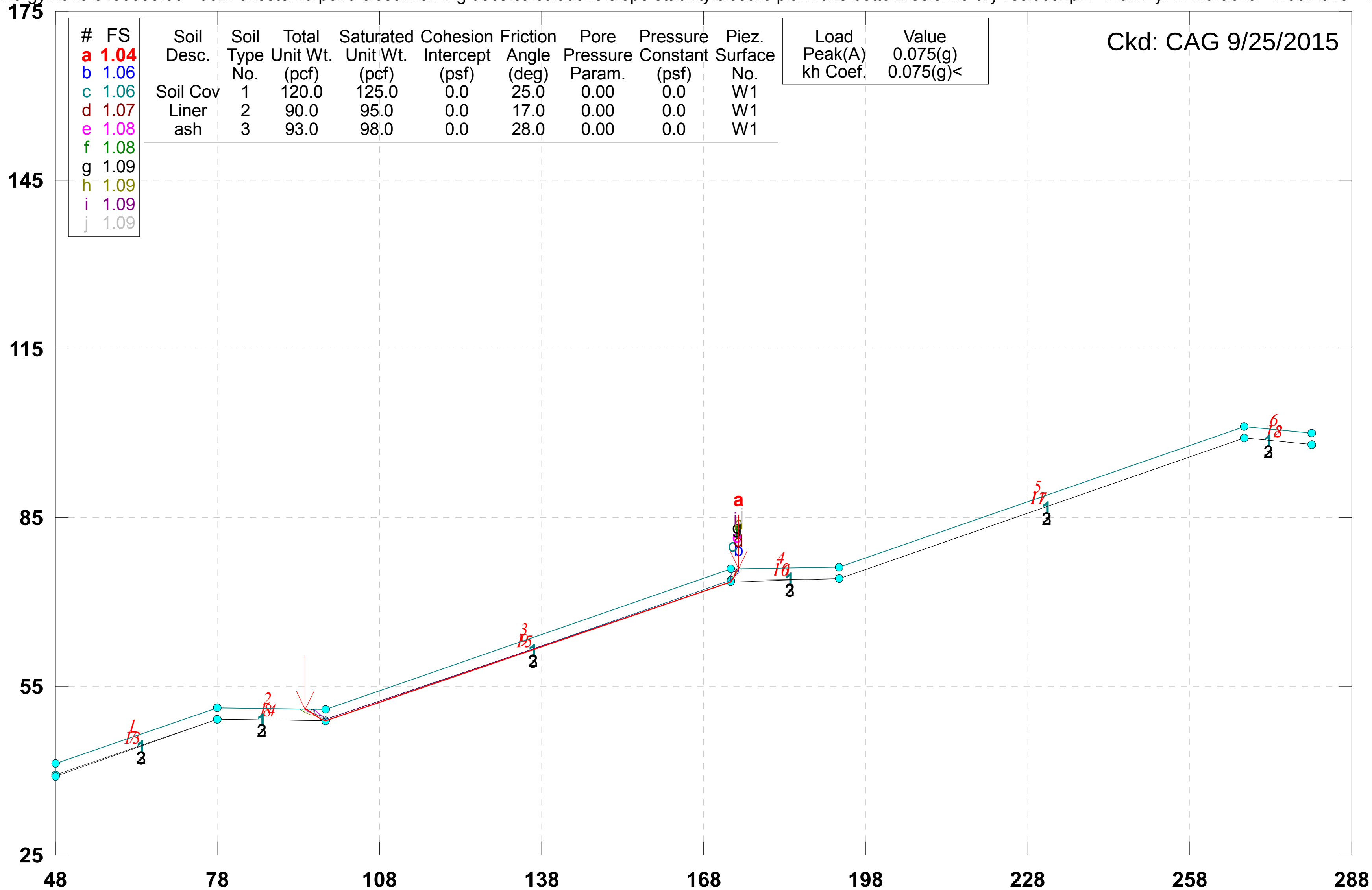

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Safety Factors Are Calculated By The Simplified Janbu Method

Chesterfield Upper Pond wet-static-peak

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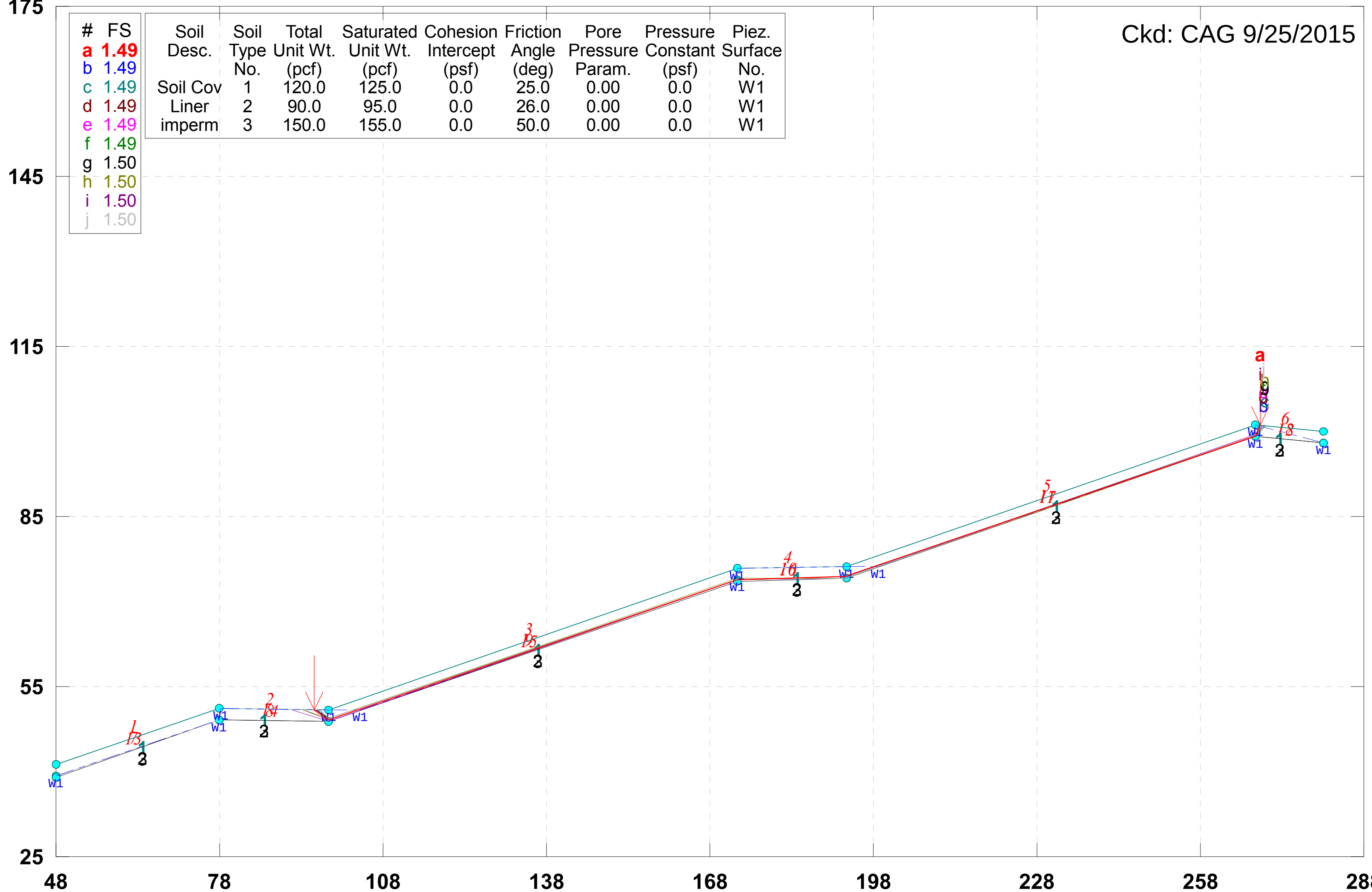


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Soil Cov	1	120.0	125.0	0.0	25.0	0.00	0.0	W1
Liner	2	90.0	95.0	0.0	26.0	0.00	0.0	W1
imperm	3	150.0	155.0	0.0	50.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=1.49

Safety Factors Are Calculated By The Simplified Janbu Method

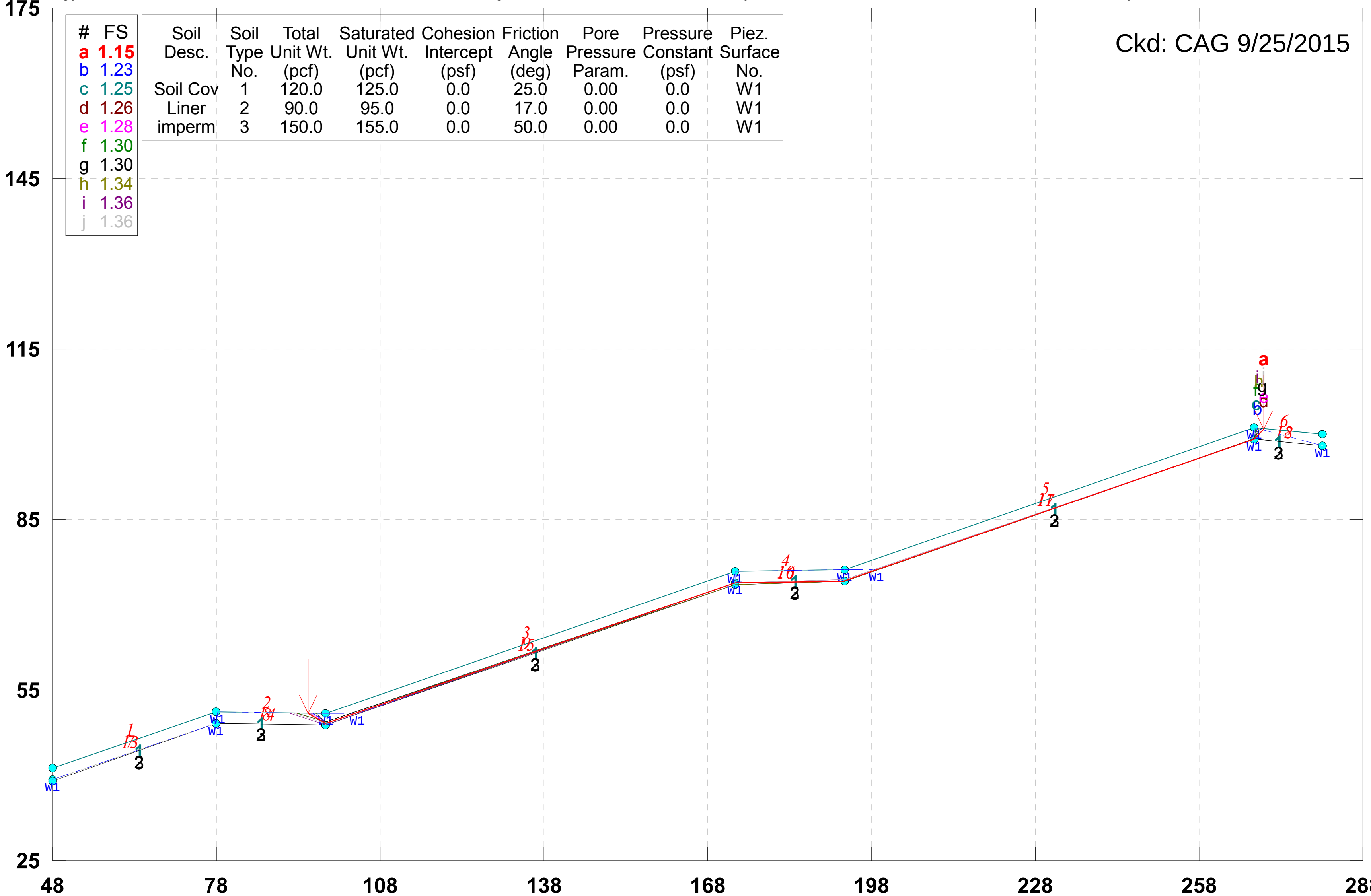
Chesterfield Upper Pond wet-static residual

z:\energy\2015\c150035.00 - dom-chesterfld pond closu\working docs\calculations\slope stability\closure plan runs\wet-static-residual.pl2 Run By: T. Muraoka 9/28/2015 09:05AM

#	FS
a	1.15
b	1.23
c	1.25
d	1.26
e	1.28
f	1.30
g	1.30
h	1.34
i	1.36
j	1.36

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Soil Cov	1	120.0	125.0	0.0	25.0	0.00	0.0	W1
Liner	2	90.0	95.0	0.0	17.0	0.00	0.0	W1
imperm	3	150.0	155.0	0.0	50.0	0.00	0.0	W1

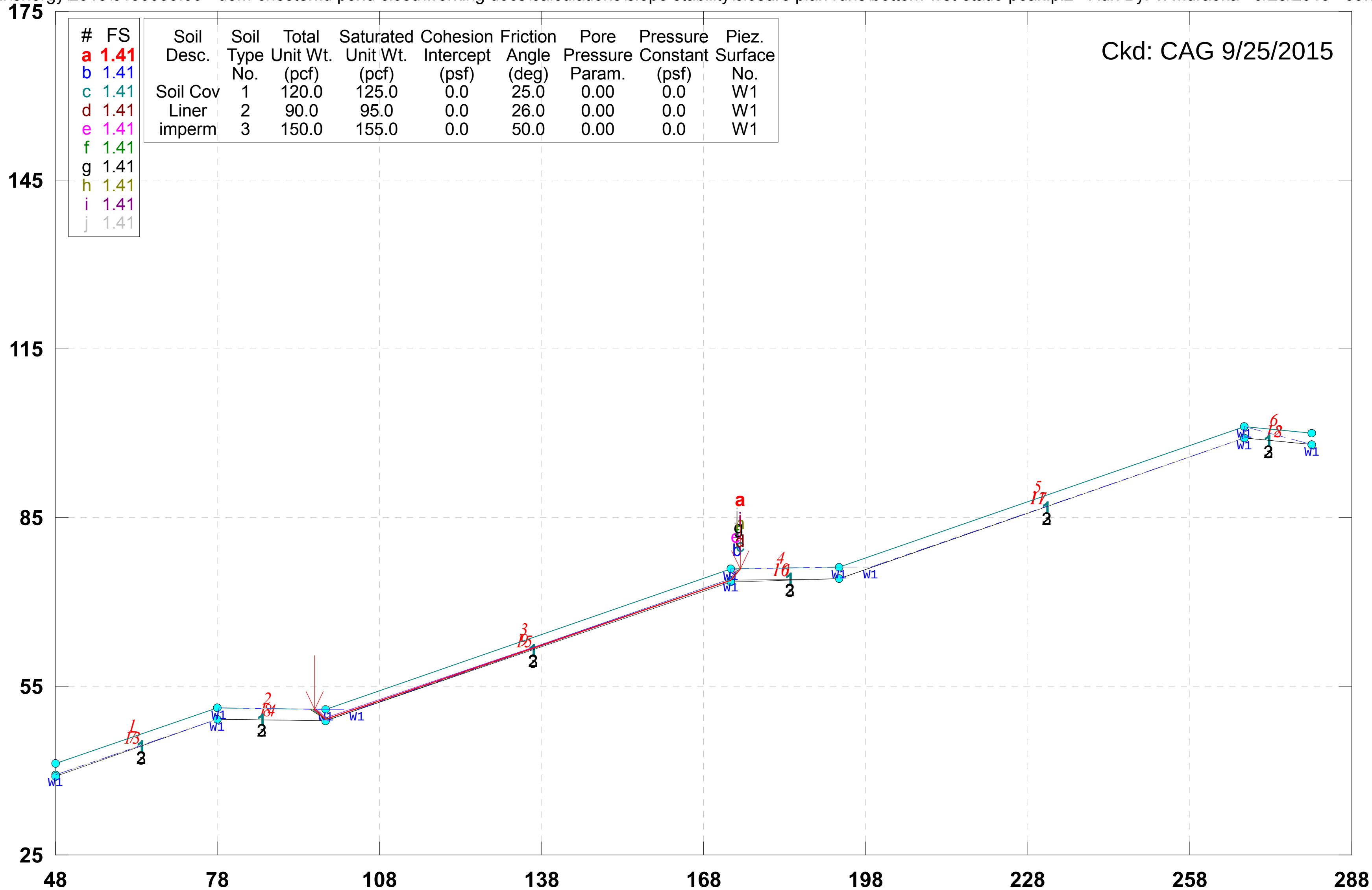
Ckd: CAG 9/25/2015



GSTABL7 v.2 FSmin=1.15
Safety Factors Are Calculated By The Simplified Janbu Method

Chesterfield Upper Pond bottom-wet-static-peak

z:\energy\2015\c150035.00 - dom-chesterfld pond closu\working docs\calculations\slope stability\closure plan runs\bottom-wet-static-peak.pl2 Run By: T. Muraoka 9/28/2015 09:06AM

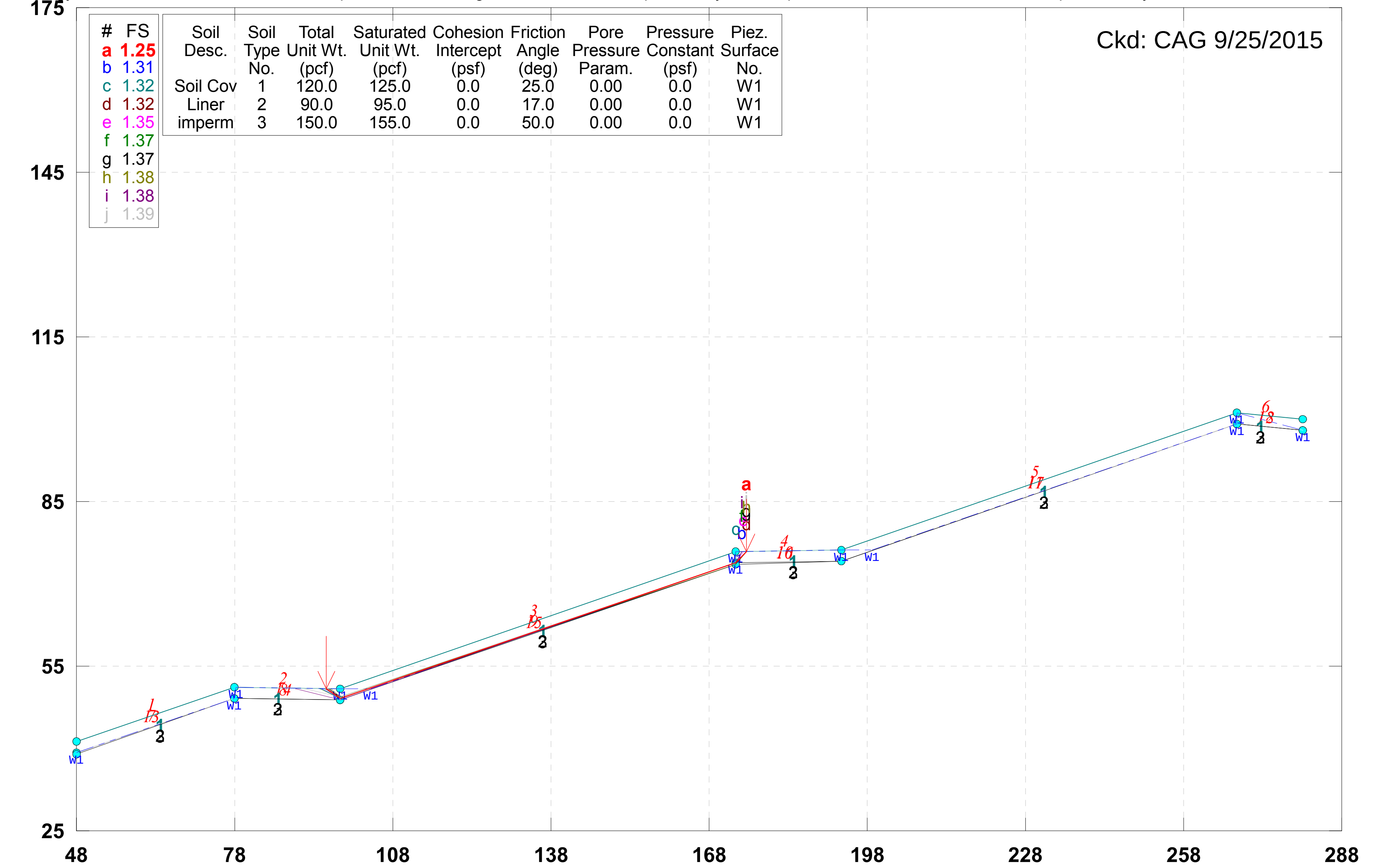


GSTABL7 v.2 FSmin=1.41

Safety Factors Are Calculated By The Simplified Janbu Method

Chesterfield Upper Pond bottom-wet-static-residual

z:\energy\2015\c150035.00 - dom-chesterfld pond closu\working docs\calculations\slope stability\closure plan runs\bottom-wet-static-residual.pl2 Run By: T. Muraoka 9/28/2015 09:07AM



GSTABL7 v.2 FSmin=1.25

Safety Factors Are Calculated By The Simplified Janbu Method

20/24
Ckd: CAG 9/25/2015

```
                        ***  GSTABL7  ***
          ** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
     ** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **
              (All Rights Reserved-Unauthorized Use Prohibited)
*********************************************************************************
                    SLOPE STABILITY ANALYSIS SYSTEM
       Modified Bishop, Simplified Janbu, or GLE Method of Slices.
       (Includes Spencer & Morgenstern-Price Type Analysis)
       Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
       Nonlinear Undrained Shear Strength, Curved Phi Envelope,
       Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
       Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
*********************************************************************************
    Analysis Run Date:        8/20/2015
    Time of Run:              03:07PM
    Run By:                   T. Muraoka
    Input Data Filename:      Z:\Energy\2015\C150035.00 - DOM-Chesterfld Pond Closu\Workin
g Docs\Calculations\slope stability\closure plan runs\static-dry-peak.in
    Output Filename:          Z:\Energy\2015\C150035.00 - DOM-Chesterfld Pond Closu\Workin
g Docs\Calculations\slope stability\closure plan runs\static-dry-peak.OUT
    Unit System:              English
    Plotted Output Filename:  Z:\Energy\2015\C150035.00 - DOM-Chesterfld Pond Closu\Workin
g Docs\Calculations\slope stability\closure plan runs\static-dry-peak.PLT
    PROBLEM DESCRIPTION:  Chesterfield Upper Pond
                          static-dry-peak
    BOUNDARY COORDINATES
        6 Top   Boundaries
```

Ckd: CAG 9/25/2015

18 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	48.00	41.20	78.00	51.20	1
2	78.00	51.20	98.00	50.80	1
3	98.00	50.80	173.00	75.80	1
4	173.00	75.80	193.00	76.20	1
5	193.00	76.20	268.00	101.20	1
6	268.00	101.20	280.60	100.00	1
7	48.00	39.20	78.00	49.20	2
8	78.00	49.20	98.00	48.80	2
9	98.00	48.80	173.00	73.80	2
10	173.00	73.80	193.00	74.20	2
11	193.00	74.20	268.00	99.20	2
12	268.00	99.20	280.60	98.00	2
13	48.00	39.10	78.00	49.10	3
14	78.00	49.10	98.00	48.70	3
15	98.00	48.70	173.00	73.70	3
16	173.00	73.70	193.00	74.10	3
17	193.00	74.10	268.00	99.10	3
18	268.00	99.10	280.60	97.90	3

User Specified Y-Origin = 25.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	125.0	0.0	25.0	0.00	0.0	1
2	90.0	95.0	0.0	26.0	0.00	0.0	1
3	93.0	98.0	0.0	28.0	0.00	0.0	1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

100 Trial Surfaces Have Been Generated.

4 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 1.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	97.90	49.15	98.10	49.15	0.50
2	172.90	73.75	173.10	73.75	0.50
3	192.90	74.15	193.10	74.15	0.50
4	267.90	99.15	268.10	99.15	0.50

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Simplified Janbu Method * *

Total Number of Trial Surfaces Attempted = 100
Number of Trial Surfaces With Valid FS = 100
Statistical Data On All Valid FS Values:
FS Max = 1.816 FS Min = 1.632 FS Ave = 1.709
Standard Deviation = 0.046 Coefficient of Variation = 2.72 %

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	96.065	50.839
2	96.474	50.540
3	97.207	49.860
4	97.917	49.155
5	172.908	73.921
6	193.067	74.383
7	267.962	99.327
8	268.493	100.174
9	269.026	101.020
10	269.100	101.095

Factor of Safety
*** 1.632 ***

Individual data on the 13 slices

Ckd: CAG 9/25/2015

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.4	7.1	0.0	0.0	0.	0.	0.0	0.0	0.0
2	0.7	54.8	0.0	0.0	0.	0.	0.0	0.0	0.0
3	0.7	110.8	0.0	0.0	0.	0.	0.0	0.0	0.0
4	0.1	16.3	0.0	0.0	0.	0.	0.0	0.0	0.0
5	74.9	15573.6	0.0	0.0	0.	0.	0.0	0.0	0.0
6	0.1	20.6	0.0	0.0	0.	0.	0.0	0.0	0.0
7	20.0	4433.6	0.0	0.0	0.	0.	0.0	0.0	0.0
8	0.1	14.6	0.0	0.0	0.	0.	0.0	0.0	0.0
9	74.9	16624.5	0.0	0.0	0.	0.	0.0	0.0	0.0
10	0.0	8.3	0.0	0.0	0.	0.	0.0	0.0	0.0
11	0.5	82.6	0.0	0.0	0.	0.	0.0	0.0	0.0
12	0.5	33.9	0.0	0.0	0.	0.	0.0	0.0	0.0
13	0.1	0.4	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 10 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	96.268	50.835
2	96.514	50.643
3	97.224	49.938
4	98.055	49.381
5	172.912	73.912
6	192.935	74.287
7	267.966	99.229
8	268.601	100.002
9	269.161	100.830
10	269.296	101.077

Factor of Safety
*** 1.632 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	94.627	50.867
2	95.397	50.408
3	96.106	49.702
4	97.089	49.521
5	98.036	49.199
6	173.096	73.993
7	193.064	74.304
8	267.935	99.228
9	268.607	99.969
10	269.124	100.825
11	269.331	101.073

Factor of Safety
*** 1.635 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	95.337	50.853
2	95.572	50.711
3	96.317	50.044
4	97.278	49.766
5	97.985	49.059
6	172.912	73.824
7	192.953	74.309
8	267.953	99.235
9	268.534	100.049
10	269.241	100.756
11	269.327	101.074

Factor of Safety
*** 1.635 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	96.251	50.835
2	96.494	50.689
3	97.212	49.993

Ckd: CAG 9/25/2015

4	97.925	49.292
5	173.095	73.978
6	192.930	74.260
7	268.074	99.358
8	268.426	100.294
9	268.758	101.128

Factor of Safety
*** 1.636 ***

Failure Surface Specified By 12 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	94.048	50.879
2	94.369	50.562
3	95.211	50.022
4	96.204	49.909
5	97.069	49.406
6	98.033	49.141
7	172.959	73.818
8	193.018	74.324
9	268.005	99.256
10	268.613	100.050
11	269.285	100.790
12	269.359	101.071

Factor of Safety
*** 1.640 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	95.293	50.854
2	95.380	50.804
3	96.151	50.167
4	97.027	49.683
5	97.970	49.352
6	173.029	73.765
7	193.025	74.358
8	268.076	99.291
9	268.634	100.121
10	269.021	101.043
11	269.038	101.101

Factor of Safety
*** 1.640 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	95.479	50.850
2	95.504	50.829
3	96.251	50.164
4	97.200	49.847
5	98.052	49.324
6	172.975	73.737
7	193.088	74.308
8	268.035	99.258
9	268.682	100.020
10	269.266	100.832
11	269.297	101.076

Factor of Safety
*** 1.641 ***

Failure Surface Specified By 11 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	95.055	50.859
2	95.492	50.731
3	96.380	50.271
4	97.217	49.724
5	98.010	49.114
6	172.969	73.865
7	193.047	74.186
8	268.011	99.364
9	268.663	100.123
10	269.370	100.830

11	269.561	101.051

Factor of Safety
*** 1.643 ***

Failure Surface Specified By 9 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	95.906	50.842
2	96.468	50.324
3	97.182	49.623
4	97.956	48.991
5	173.096	73.956
6	193.041	74.336
7	268.060	99.166
8	268.562	100.030
9	268.566	101.146

Factor of Safety
*** 1.644 ***

**** END OF GSTABL7 OUTPUT ****

SUBJECT Closure of Upper (East) Pond –Deep Seated Stability Analyses

BY TIM DATE 5/5/2015 PROJ. NO. C150035.00

CHKD. BY CAG DATE 9/25/2015 SHEET NO. 1 OF 8

gai consultants

Engineers • Geologists • Planners
Environmental Specialists

OBJECTIVE:

Evaluate deep-seated rotational failure surfaces under static and seismic conditions for the proposed closure of the Upper (East) Pond at the Chesterfield Power Station, located in Chesterfield County, Virginia.

METHODOLOGY:

Stability will be evaluated under both static and seismic conditions using two-dimensional limit equilibrium analysis with the software GSTABL 7.

REFERENCES:

1. Schnabel Engineering Consultants, Inc. *Geotechnical Engineering Report: Upper Pond Stability Evaluation,* August 2014.

2. GAI Consultants, Inc. *Revised Closure Plan Upper (East Pond) Chesterfield Power Station, Chesterfield County, Virginia.* September 2003.

3. Geotechnical Engineering and Groundwater Hydrology Services, Ash Disposal Pond, Chesterfield Power Station, dated 12/20/1982. Prepared by Schnabel Engineering Associates, Inc.

4. Geotechnical Engineering Study, Long Term Ash Storage Pond Dike, Chesterfield County, Virginia, dated April 22, 1996. Prepared by Schnabel Associates, Inc.

5. Petersen, Mark, et al. Seismic-Hazard Maps for the Conterminous United States. 2014. United States Department of Interior. United States Geological Survey.

BACKGROUND:

In the 2003 closure plan for the Upper (East) Pond, GAI identified sections that could be susceptible to slope failures in the future. GAI will re-evaluate the stability analyses with the new geometry of the impoundment, site conditions, and material properties.

ANALYSIS:

Sections were cut in locations similar to where they were cut for the 2003 closure plan. These areas were in the southeast area of the Pond near VPDES Outfall 005 and along Henricus Access Road in the northeast portion of the Pond. After reviewing the 2003 closure plan (Reference 2) and the Schnabel geotechnical reports conducted in the area in 1982 and 1996 (Reference 3 and 4), soil parameters used in this analysis were equal to the parameters referenced in the Schnabel reports. A table of the parameters is listed below.

Table 1: Slope Stability Material Properties

Soil Type	Soil Type #	γ_T (pcf)	γ_{Sat} (pcf)	c=c' (psf)	$\phi=\phi'$ (Degrees)
Embankment/Road Fill	1	120	125	0	32
SM-SP	2	125	130	0	35
Low Blow Count	3	120	125	0	27
SM,SC	4	135	140	0	40
Old Marsh	5	90	95	40	9

The “Low Blow Count “ layer of soil referenced in the table above was added to the stability analysis after reviewing the relevant boring logs. The 2003 closure plan identified a 40’ thick section of “marsh soil”. The strength parameters for the soil were a contributing factor to the calculated instability of the road. The “Low Blow Count” layer was included to model the softer soil. A phi angle was developed based on N values corrected for field procedures and phi angle/blow count correlation. The weaker soil layer thickness was adjusted based on the boring logs.

Information for the phreatic surface was taken from the Schnabel boring logs. The 2003 closure plan had the phreatic surface elevation of approximately 35’. The water level should not be that high as wet disposal of ash has been discontinued in the Pond.

Seismic Conditions – The existing facility is located in Chesterfield County, Virginia, which is an area of low seismic activity and risk. The peak horizontal ground acceleration at the proposed site (using a 2 percent probability of exceedance in 50 years) is approximately 0.075g. This acceleration was estimated using USGS mapping and prior stability runs performed by GAI and Schnabel and was used in PCSTABL to perform a pseudo-static force evaluation of seismic stability.

The factors of safety for the southern section under static and seismic conditions were 2.5 and 2.0, respectively. The static and seismic factors of safety for the northern section was equal to 1.4 and 1.1. The factor of safety for the static condition is under the desired outcome of 1.5. After evaluating the slope stability runs, the critical failure surface occurs near the top of the slope of the road adjacent to the impoundment. The failure surface is a considerable distance (over 30’) from the toe of the dike. If a slope failure were to occur in this area, integrity of the dike should not be affected. The failure surface is far away from the dike that there should be enough time to address the failure. The stability runs are included in this report as Attachment 1 (Sheets 5 to 8).

RESULTS & SUMMARY:

Stability analyses were performed on a section in the southeast area of the Pond (near VPDES Outfall 005) and in the northeast area (along Henricus Access Road). Multiple surfaces were generated and the most critical failure surface for each analysis was isolated to determine the minimum factor of safety. The factor of safety for the dry condition of the northern section was under the desired outcome of 1.5. If a failure were to occur, GAI recommends the failure be addressed quickly to protect the stability of the dike.

ATTACHMENT 1

SLOPE STABILITY OUTPUT FILES

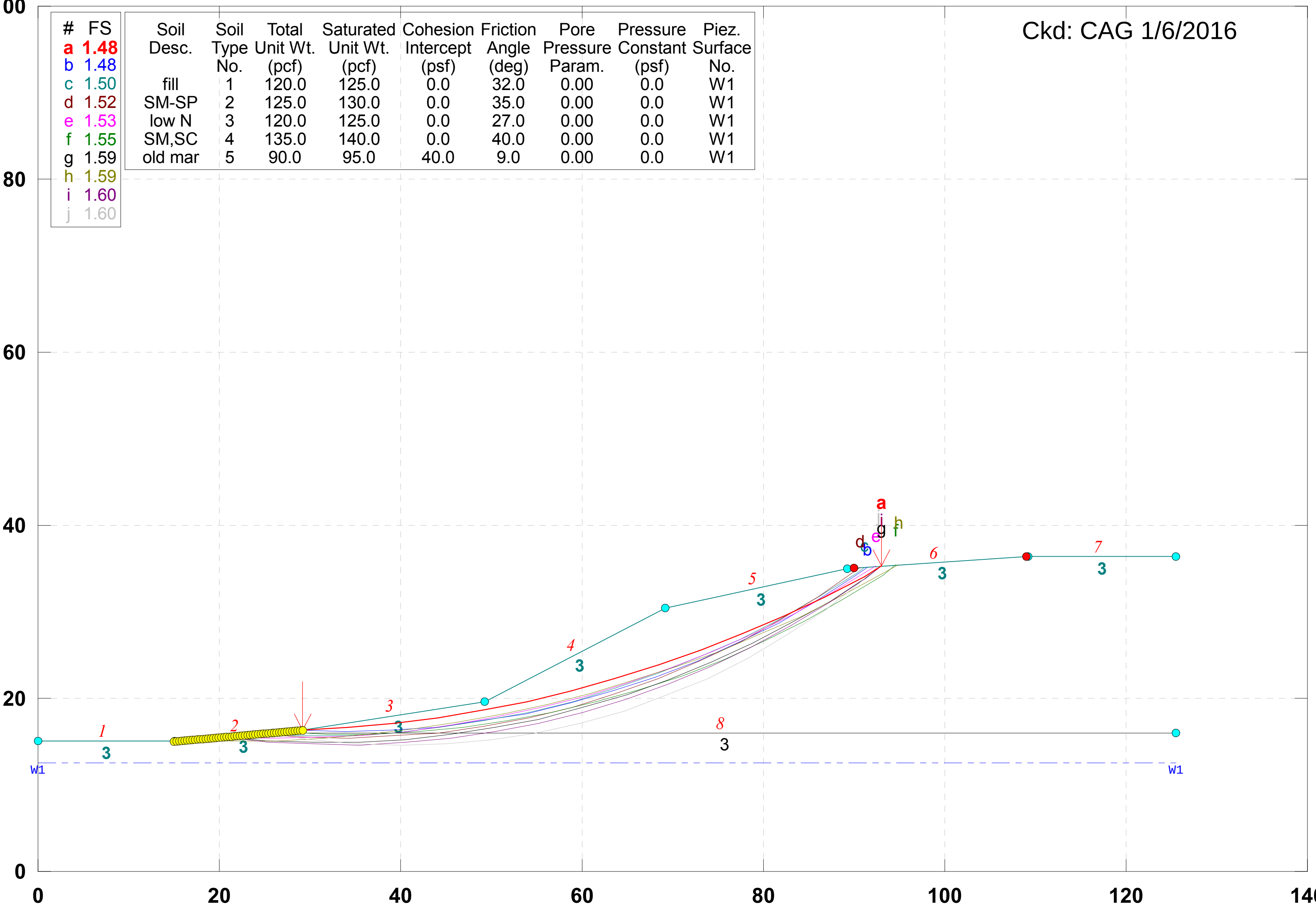
Chesterfield Station north area-dry static

z:\energy\2015\c150035.00 - dom-chesterfld pond closu\working docs\calculations\slope stability\closure plan sections\north area-application run.pl2   Run By: T. Muraoka   1/6/2016   09:44AM

#	FS
a	1.48
b	1.48
c	1.50
d	1.52
e	1.53
f	1.55
g	1.59
h	1.59
i	1.60
j	1.60

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
fill	1	120.0	125.0	0.0	32.0	0.00	0.0	W1
SM-SP	2	125.0	130.0	0.0	35.0	0.00	0.0	W1
low N	3	120.0	125.0	0.0	27.0	0.00	0.0	W1
SM,SC	4	135.0	140.0	0.0	40.0	0.00	0.0	W1
old mar	5	90.0	95.0	40.0	9.0	0.00	0.0	W1

Ckd: CAG 1/6/2016

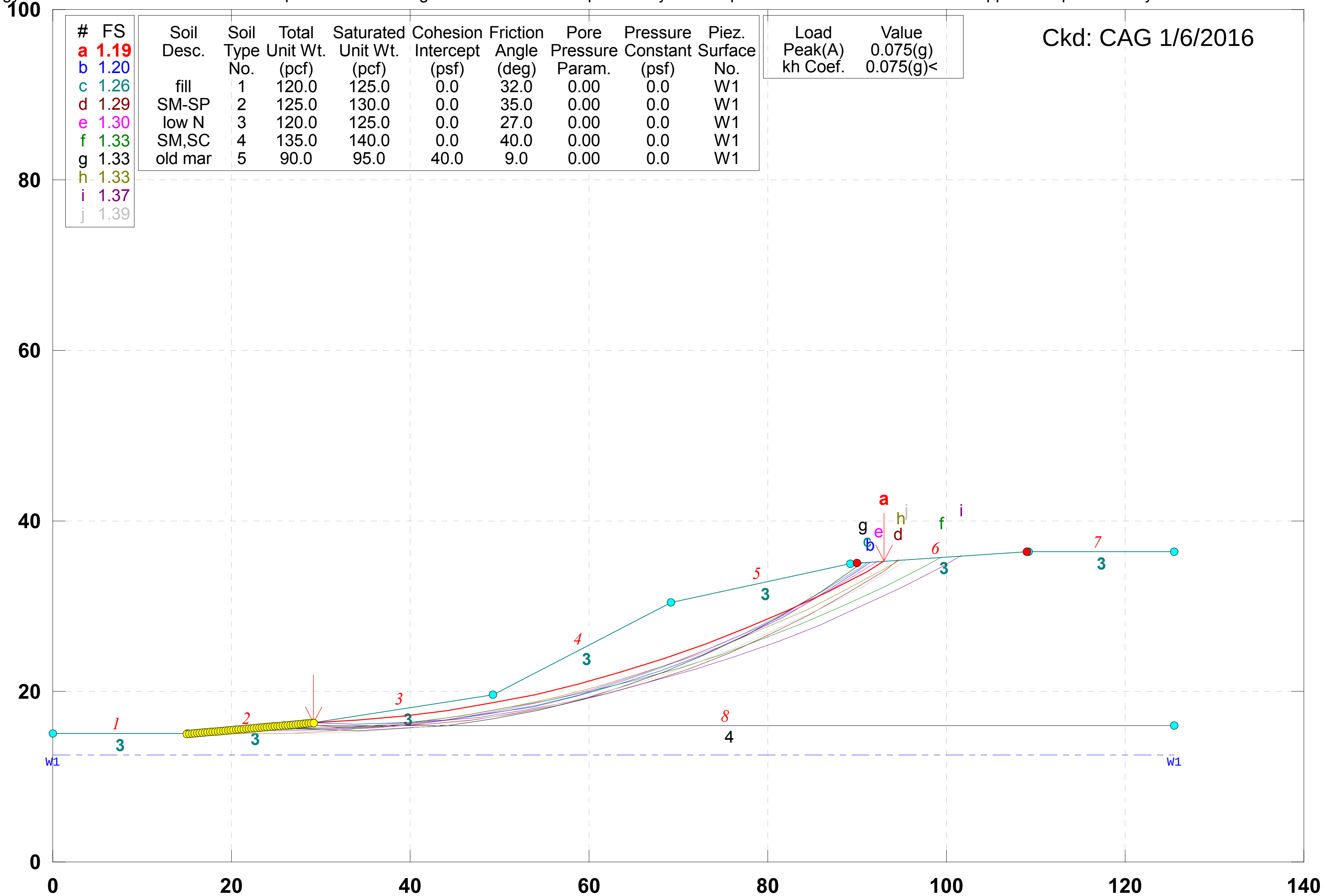


GSTABL7 v.2 FSmin=1.48

Safety Factors Are Calculated By The Modified Bishop Method

Chesterfield Station north area-dry seismic

z:\energy\2015\c150035.00 - dom-chesterfld pond closu\working docs\calculations\slope stability\closure plan sections\north area-seismic-application.pl2 Run By: T. Muraoka 1/6/2016 09:46AM

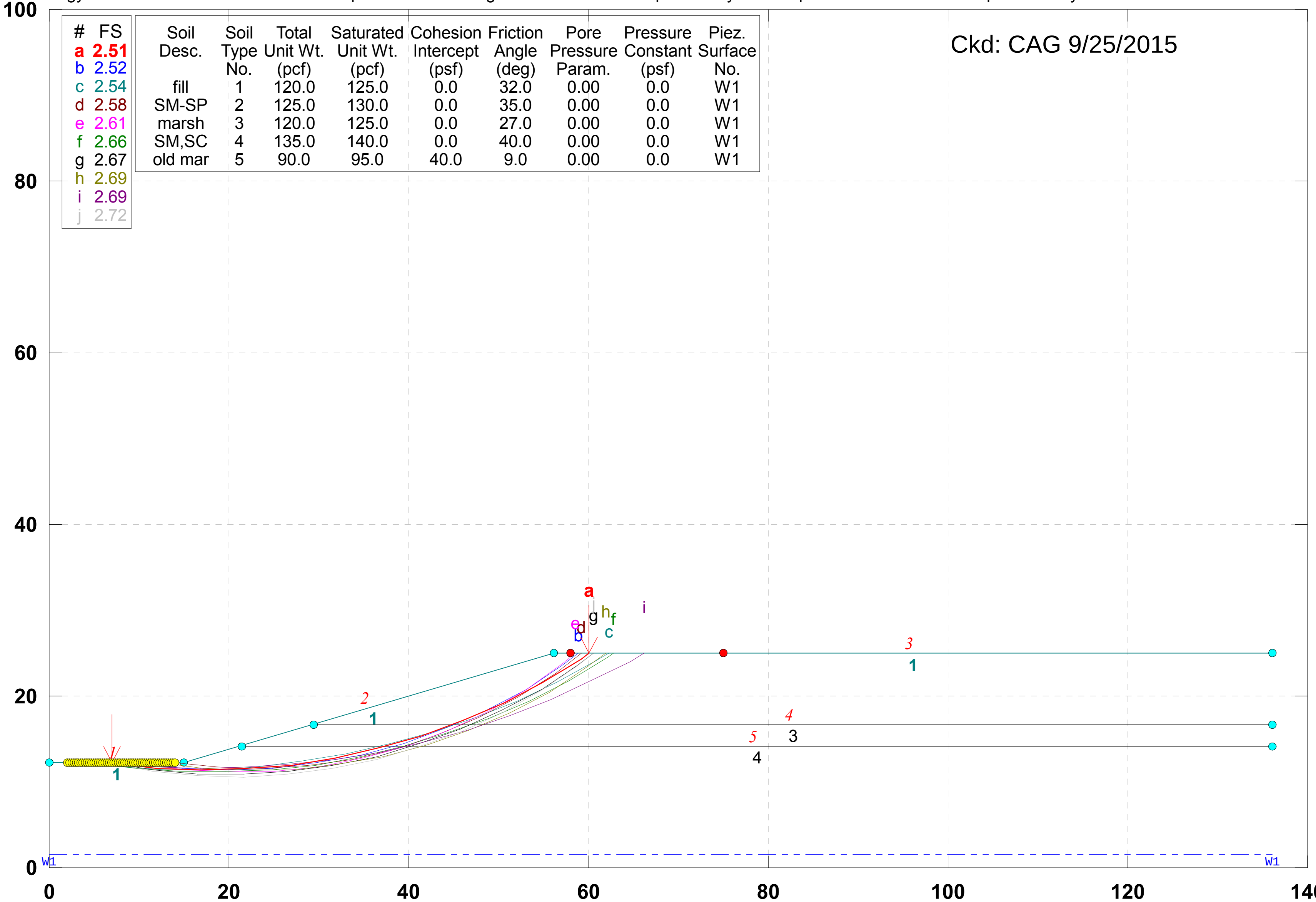


GSTABL7 v.2 FSmin=1.19

Safety Factors Are Calculated By The Modified Bishop Method

Chesterfield Station south area-dry static

z:\energy\2015\c150035.00 - dom-chesterfld pond closu\working docs\calculations\slope stability\closure plan sections\south area.pl2 Run By: T. Muraoka 7/10/2015 09:28AM



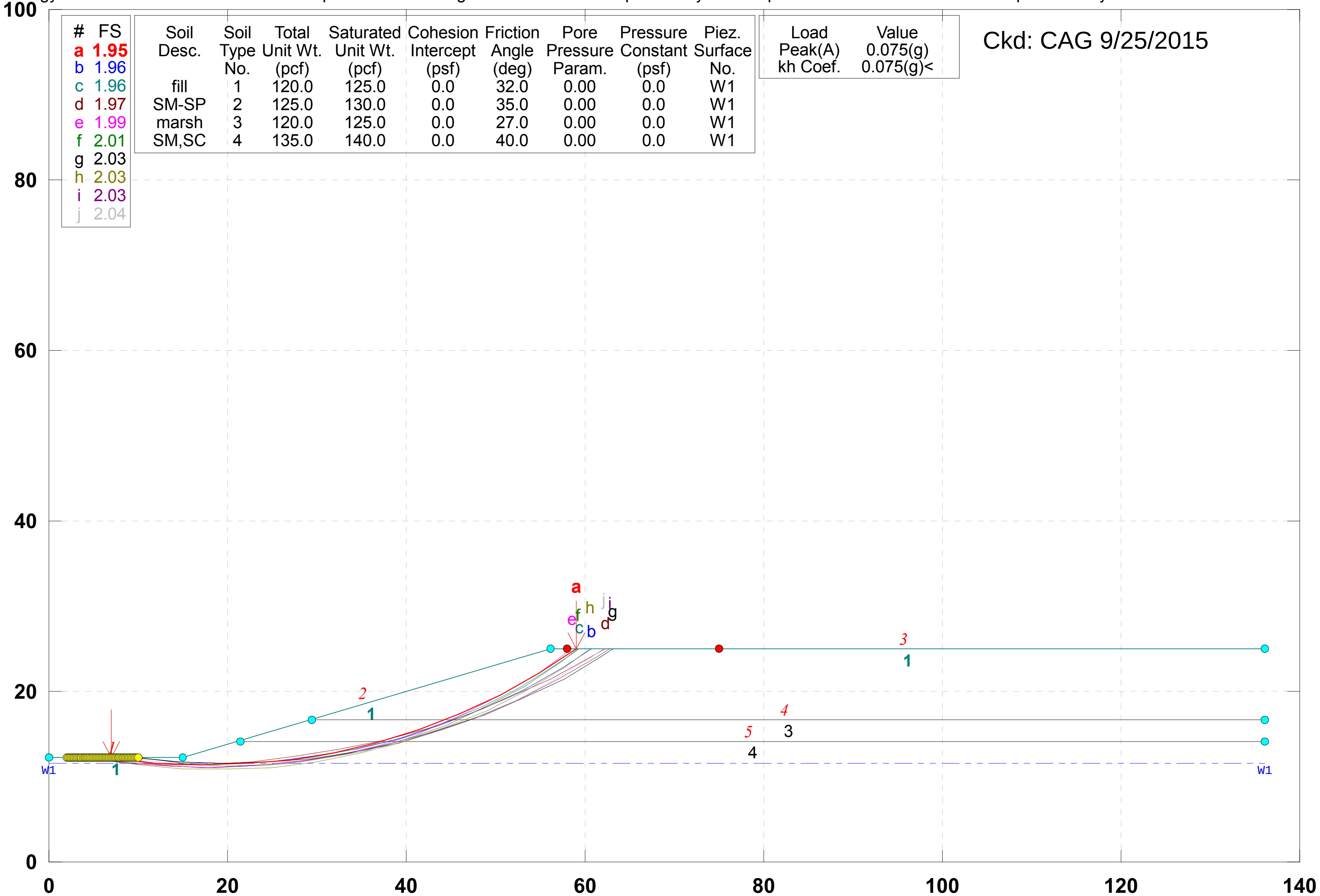
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
fill	1	120.0	125.0	0.0	32.0	0.00	0.0	W1
SM-SP	2	125.0	130.0	0.0	35.0	0.00	0.0	W1
marsh	3	120.0	125.0	0.0	27.0	0.00	0.0	W1
SM,SC	4	135.0	140.0	0.0	40.0	0.00	0.0	W1
old mar	5	90.0	95.0	40.0	9.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=2.51

Safety Factors Are Calculated By The Modified Bishop Method

Chesterfield Station south area-dry seismic

z:\energy\2015\c150035.00 - dom-chesterfld pond closu\working docs\calculations\slope stability\closure plan sections\south area-seismic.pl2   Run By: T. Muraoka   7/9/2015   01:56PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
fill	1	120.0	125.0	0.0	32.0	0.00	0.0	W1
SM-SP	2	125.0	130.0	0.0	35.0	0.00	0.0	W1
marsh	3	120.0	125.0	0.0	27.0	0.00	0.0	W1
SM,SC	4	135.0	140.0	0.0	40.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=1.95

Safety Factors Are Calculated By The Modified Bishop Method

SUBJECT CLOSURE OF UPPER (EAST) POND

ANCHOR TRENCH DESIGN CALCULATIONS

BY TIM DATE 10/5/2015 PROJ. NO. C150035.00

CHKD. BY JLM DATE 10/6/2015 SHEET NO. 1 OF 8

gai consultants

OBJECTIVE:

Determine the adequacy of the proposed anchor trench design for the closure of the Upper (East) Pond.

METHODOLOGY:

Use force equilibrium to estimate the factor of safety against liner pullout from the anchor trench.

REFERENCES:

1. "Geotechnical Aspects of Landfill Design and Construction", Qian et al., 2002, pp. 117.

2. *GSE Ultraflex Textured Geomembranes*, GSE Lining Technology, Inc. June 14, 2011, http://www.gseworld.com/content/documents/datasheets/membranes/North_America/U1UltraFlex_textured_geomem_english.pdf

3. "Designing With Geosynthetics Fifth Edition", Koerner, 2005, pp. 500.

BACKGROUND:

The Upper (East) Pond will be capped with a liner system that conforms to Virginia regulations and U.S. Environmental Protection Agency's Coal Combustion Residual (CCR) Rule requirements. The proposed liner system will include a 40-mil textured linear low-density polyethylene (LLDPE) geomembrane as the liner. The liner will serve as a barrier to prevent surface water from contacting the underlying CCR material. This geomembrane will be placed directly atop the subgrade (existing ground, regraded CCR material, or a cushion (nonwoven) geotextile) and covered with a geocomposite drainage net (GDN). A 24" soil cover layer will cover the GDN.

ANALYSIS:

Anchor Trench Design – An anchor trench with a frictional capacity less than the liner peak tensile strength was developed to resist lateral movement and prohibit surface water from migrating under the geomembrane. A one-sided V-shaped anchor trench was considered due to the high flexibility of LLDPE liner material. Interface friction angles for the LLDPE geomembrane to nonwoven geotextile came from data provided by Agru America. From Reference 1, the following equation was used for design:

$$\boxed{T = \frac{\gamma_s d_{CS} L_{RO} \tan\delta_C + \gamma_s\left[(d_{CS} + 0.5 d_{AT}) d_{AT} (\tan\delta_C + \tan\delta_F) / \tan a_L\right]}{\cos\beta - \sin\beta\tan\delta_C}}$$

Where

	T	= geomembrane tensile force per unit width, lb/ft
	γ_S	= Unit weight of cover and backfill soil, pcf
	d_{cs}	= depth of cover soil, ft
	d_{AT}	= depth of anchor trench, ft
	L_{RO}	= runout length, ft
	δ_c	= interface between geomembrane and underlying soil, deg
	δ_F	= interface between geomembrane and backfill soil, deg
	a_L	= left bottom angle of V-shaped anchor trench, measured from horizontal;

Φ = friction angle between geomembrane and soil, deg
β = side slope angle, deg

From the calculations included in Attachment 2 (Sheet 6 of 8), the anchor trench dimensions listed below provide an allowable geomembrane tensile force (T_{allow}) of 42.9 lbs/linear inch of liner width. The ultimate geomembrane tensile strength (T_{ult}), based on manufacturer's cut sheets, Attachment 3 (Sheet 8 of 8), is 60.0 lb/in. The factor of safety against pullout is calculated as follows:

$$FS = \frac{\mathrm{T_{ult}}}{\mathrm{T}_{allow}} = \frac{60.0\ lb/in}{42.9\ lb/in} = 1.4$$

Dimensions of the anchor trench are listed below:

1. Liner Runout Length = 1.0 ft
2. Depth = 1.0 ft
3. Bottom Width = 1.0 ft
4. Cover Soil Depth = 2.0 ft
5. Trench Side Slope angle = 45.0 degrees

SUMMARY:

Conditions were evaluated using conservative interface shear strength values for the proposed construction materials to determine the loading conditions on the proposed liner system. The anchor trench was designed to pull out of the anchor trench before the liner tears. A range of interface friction angles were used to analyze the anchor trench. Using an interface friction angle of 26.0 degrees, the factor of safety is equal to 1.4.

SUBJECT CLOSURE OF UPPER (EAST) POND

ANCHOR TRENCH DESIGN CALCULATIONS

BY TIM DATE 7/20/2015 PROJ. NO. C150035.00

CHKD. BY JLM DATE 10/6/2015 SHEET NO. 3 OF 8

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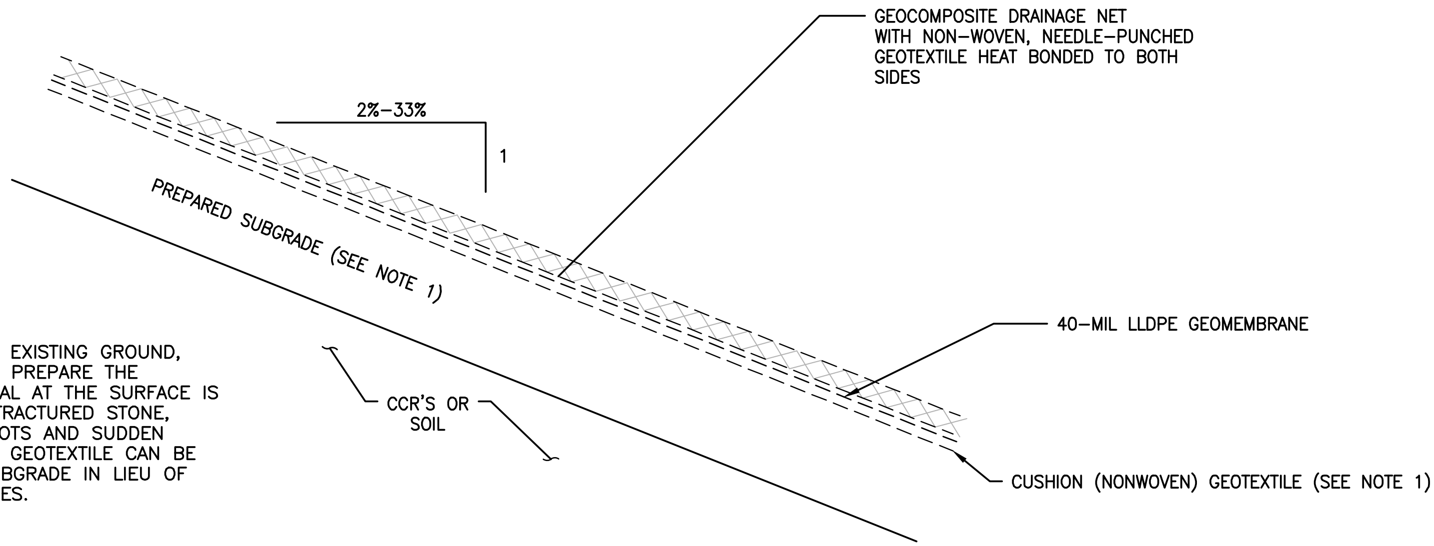
ATTACHMENT 1

DESIGN DETAILS

SHEET NO. 4 OF 8



NOTES:

1. WHERE SUBGRADE CONSISTS OF EXISTING GROUND, STRIP EXISTING VEGETATION AND PREPARE THE SUBGRADE SO THAT THE MATERIAL AT THE SURFACE IS FREE OF PROTRUDING ROCKS, FRACTURED STONE, DEBRIS, COBBLES, RUBBISH, ROOTS AND SUDDEN CHANGES IN SLOPE. A CUSHION GEOTEXTILE CAN BE PLACED OVER THE STRIPPED SUBGRADE IN LIEU OF SUBGRADE PREPARATION ACTIVITIES.

CAP SYSTEM DETAIL 1
N.T.S. 5,7,9,11,16,18,20,25,26,27,28,29

SUBJECT CLOSURE OF UPPER (EAST) POND

ANCHOR TRENCH DESIGN CALCULATIONS

BY TIM DATE 7/20/2015 PROJ. NO. C150035.00

CHKD. BY JLM DATE 10/6/2015 SHEET NO. 5 OF 8

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ATTACHMENT 2

CALCULATIONS

LANDFILL LINER Triangular-Shaped Anchor Trench

Ref: Qian et al. "Geotechnical Aspects of Landfill Design and Construction", 2002, pp. 117

Unit weight of cover and backfill soil, γ_s =	120	lb/ft^3
Depth of cover soil, d_{CS} =	2	ft
Anchor trench depth, d_{AT} =	1	ft
Runout length, L_{RO} =	1	ft
Anchor trench length, L_{AT} =	1.0	ft

					tan	sin	cos
Interface between geomembrane and underlying soil, δ_C =	0.454	rad =	26.0	degrees	0.488	0.438	0.899
Interface between geomembrane and backfill soil, δ_F =	0.454	rad =	26.0	degrees	0.488	0.438	0.899
Trench side slope angle, α_L =	0.785	rad =	45.0	degrees	1.000	0.707	0.707
Side slope angle, β =	0.321	rad =	18.4	degrees	0.333	0.316	0.949
Friction angle between geomembrane and soil, φ =	0.454	rad =	26.0	degrees	0.488	0.438	0.899

$$\boxed{T = \frac{\gamma_s d_{CS} L_{RO} \tan\delta_C + \gamma_s \left[\left(d_{CS} + 0.5 d_{AT}\right) d_{AT} \left(\tan\delta_C + \tan\delta_F\right) / \tan a_L\right]}{\cos\beta - \sin\beta \tan\delta_C}}$$

Allowable Geomembrane tensile force per unit width, T = 515.4 lb/ft = 42.9 lb/in

Anchor Trench Geometry: 

ATTACHMENT 3

MANUFACTURER'S CUT SHEET

GSE UltraFlex Textured Geomembrane

GSE UltraFlex Textured is a co-extruded textured linear low density polythylene (LLDPE) geomembrane available on one or both sides. It is manufactured from the highest quality resin specifically formulated for flexible geomembranes. This product is used in applications that require increased frictional resistance, flexibility and elongation properties where differential or localized subgrade settlements may occur such as in a landfill closure application.

[*]

AT THE CORE:
An LLDPE geomembrane that is used in applications requiring increased frictional resistance, flexibility and elongation properties, such as landfill closures and mining applications.

Product Specifications

These product specifications meet GRI GM17

Tested Property	Test Method	Frequency	Minimum Average Value			
			40 mil	60 mil	80 mil	100 mil
Thickness, mil Lowest individual reading	ASTM D 5994	every roll	40 36	60 54	80 72	100 90
Density, g/cm³ (max.)	ASTM D 1505	200,000 lb	0.939	0.939	0.939	0.939
Tensile Properties (each direction) Strength at Break, lb/in-width Elongation at Break, %	ASTM D 6693, Type IV Dumbbell, 2 ipm G.L. 2.0 in	20,000 lb	60 250	90 250	120 250	150 250
Tear Resistance, lb	ASTM D 1004	45,000 lb	22	33	44	55
Puncture Resistance, lb	ASTM D 4833	45,000 lb	44	66	88	110
Carbon Black Content, % (Range)	ASTM D 1603*/4218	20,000 lb	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0
Carbon Black Dispersion	ASTM D 5596	45,000 lb	Note(1)	Note(1)	Note(1)	Note(1)
Asperity Height, mil	ASTM D 7466	second roll	18	18	18	18
Oxidative Induction Time, mins	ASTM D 3895, 200°C; O_2, 1 atm	200,000 lb	> 100	> 100	> 100	> 100
TYPICAL ROLL DIMENSIONS						
Roll Length(2), ft	Double-Sided Textured Single-Sided Textured		700 780	520 540	400 410	330 330
Roll Width(2), ft			22.5	22.5	22.5	22.5
Roll Area, ft²	Double-Sided Textured Single-Sided Textured		15,750 17,550	11,700 12,150	9,000 9,225	7,425 7,425

NOTES:
- (1)Dispersion only applies to near spherical agglomerates. 9 of 10 views shall be Category 1 or 2. No more than 1 view from Category 3.
- (2)Roll lengths and widths have a tolerance of ±1%.
- GSE UltraFlex Textured is available in rolls weighing approximately 4,000 lb.
- All GSE geomembranes have dimensional stability of ±2% when tested according to ASTM D 1204 and LTB of <-77°C when tested according to ASTM D 746.
- *Modified.

GSE is a leading manufacturer and marketer of geosynthetic lining products and services. We've built a reputation of reliability through our dedication to providing consistency of product, price and protection to our global customers.

Our commitment to innovation, our focus on quality and our industry expertise allow us the flexibility to collaborate with our clients to develop a custom, purpose-fit solution.

GSE ENVIRONMENTAL™

[DURABILITY RUNS DEEP] For more information on this product and others, please visit us at GSEworld.com, call 800.435.2008 or contact your local sales office.

INTRODUCTION

The GDN beneath the cover layer for the Upper (East) Pond will convey flow that infiltrates through the cover soil. Estimate the amount of water that will need to be conveyed.

INFILTRATION QUANTITY

A HELP model analysis for a site in southern Virginia will be used to aid in determining infiltration volume:

Layer 1 = 24-inch vegetated soil layer.

```
                    LAYER  1
                    --------

      TYPE 1 - VERTICAL PERCOLATION LAYER
          MATERIAL TEXTURE NUMBER  10
THICKNESS                  =     24.00   INCHES
POROSITY                   =      0.3980 VOL/VOL
FIELD CAPACITY             =      0.2440 VOL/VOL
WILTING POINT              =      0.1360 VOL/VOL
INITIAL SOIL WATER CONTENT =      0.3116 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.119999997000E-03 CM/SEC
NOTE:  SATURATED HYDRAULIC CONDUCTIVITY IS MULTIPLIED BY  4.63
         FOR ROOT CHANNELS IN TOP HALF OF EVAPORATIVE ZONE.
```

Layer 2 = conveyance layer (GDN)

```
                    LAYER  2
                    --------

      TYPE 2 - LATERAL DRAINAGE LAYER
          MATERIAL TEXTURE NUMBER   0
THICKNESS                  =      0.25   INCHES
POROSITY                   =      0.8500 VOL/VOL
FIELD CAPACITY             =      0.0100 VOL/VOL
WILTING POINT              =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT =      0.0129 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =   1.00000000000      CM/SEC
SLOPE                      =     33.30   PERCENT
DRAINAGE LENGTH            =     78.0    FEET
```

The results show:

ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	45.82	166326.609	100.00
RUNOFF	1.285	4665.616	2.81
EVAPOTRANSPIRATION	30.427	110448.844	66.40
DRAINAGE COLLECTED FROM LAYER 2	14.1627	51410.457	30.91

Infiltration = 46.5% of the volume that did not run off.

Estimate the runoff quantity at the Upper (East) Pond under a final cover condition on a unit area basis.

Curve Number = 74 for final reclaimed areas (from 2003 closure package)

From TR-55,

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \text{[eq. 2-3]}$$

$$S = \frac{1000}{CN} - 10 \qquad \text{[eq. 2-4]}$$

Q = runoff (in)
P = rainfall (in)

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - GDN FLOWS

BY KMB DATE 07/01/2015 PROJ. NO. C150035.00

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For a 25-year 24-hour storm, precipitation = 6.31 inches:

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
24-hr	2.78 (2.54-3.06)	3.36 (3.08-3.71)	4.31 (3.93-4.76)	5.11 (4.65-5.64)	6.31 (5.69-6.94)	7.33 (6.57-8.05)	8.45 (7.52-9.27)	9.70 (8.55-10.6)	11.6 (10.0-12.6)	13.1 (11.3-14.4)

CN = 74
S = 3.51

P = 6.31 inches
Q = 3.45 inches

Volume not run off =
2.86 inches

Infiltration =
1.33 inches

INFILTRATED FLOW

Estimate the peak flow rate generated in the underdrain layer by using TR-55 methods. Surface runoff will be extrapolated to underdrain flow by:

- Modeling a storm event that produces the same runoff quantity (1.33 inches) as the assumed infiltration amount
- Using soil permeability to generate a time of concentration

Using a CN = 100, the full precipitation value is considered as runoff:

CN = 100
S = 0.00

P = 1.33 inches
Q = 1.33 inches

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - GDN FLOWS

BY KMB DATE 07/01/2015 PROJ. NO. C150035.00

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If 2 feet of cover soil are used, estimate the time to infiltrate:

Assumed soil permeability	1.00E-03	cm/s
Soil depth	2	ft
	60.96	cm
Infiltration duration	60,960	seconds
	1016	minutes
	16.9	hours

Use 16.9 hours as a time of concentration. With a 1-acre watershed, the results are:

Hyd. No. 1

1 acre trial watershed

Hydrograph type	= SCS Runoff	Peak discharge	= 0.08 cfs
Storm frequency	= 25 yrs	Time interval	= 6 min
Drainage area	= 1.000 ac	Curve number	= 100
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= USER	Time of conc. (Tc)	= 1016.00 min
Total precip.	= 1.33 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

Hydrograph Volume = 4,834 cuft

This works out to

0.08	cfs	1	acre	equals	0.0000018	ft	5.60E-05	cm
1	acre	43,560	sf			sec		sec

For a 25-year 24-hour storm, peak infiltrated flow is 5.60 x 10^{-5} cm/sec

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - GDN FLOWS

BY KMB DATE 07/01/2015 PROJ. NO. C150035.00

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PMF EVENT

Estimate infiltrated flow due to a PMF event.

The PMP (probable maximum precipitation) is 39 inches for 24-hours.

CN =	74	
S =	3.51	
P =	39	inches
Q =	35.08	inches
Volume not run off =		
	3.92	inches
Assumed infiltration =		
	1.82	inches

Using a 1.82-inch precipitation on a 1-acre watershed with CN = 100:

Hyd. No. 1

1 acre trial watershed

Hydrograph type	= SCS Runoff	Peak discharge	= 0.11 cfs
Storm frequency	= 2 yrs	Time interval	= 6 min
Drainage area	= 1.000 ac	Curve number	= 100
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= USER	Time of conc. (Tc)	= 1016.00 min
Total precip.	= 1.82 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

Hydrograph Volume = 6,615 cuft

0.11 cfs		1 acre	equals	0.0000025	ft	7.70E-05	cm
1 acre		43,560	sf		sec		sec

For a PMP event, peak infiltrated flow is 7.70 x 10^{-5} cm/sec

SUBJECT CLOSURE OF UPPER (EAST) POND

GEOCOMPOSITE DRAINAGE NET (GDN) DESIGN

BY TIM DATE 5/14/2015 PROJ. NO. C150035.00

CHKD. BY JRK DATE 12/10/2015 SHEET NO. 1 OF 9

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OBJECTIVE:

Determine the capacity of the proposed Geocomposite Drainage Net (GDN) for the proposed closure of the Upper (East) Pond of the Chesterfield Station located in Chesterfield County, Virginia.

REFERENCES:

1. Geotechnical Aspects of Landfill Design and Construction. Qian, X., Koerner, R. M., and Gray, D. H., Prentice Hall, Upper Saddle River, New Jersey, 2002.

2. "Designer's Forum: Landfill Drainage Layers Part 3 of 4," Thiel, Richard, Narejo, Dhani, and Richardson, Gregory N.

3. Lessons Learned from Failure: Developing Better Drainage Systems for Cover Side Slopes by Studying Failed Ones, Gregory. N. Richardson and K.L. Pavlik, October/November 2004,

4. Geosynthetic Institute, GSI White Paper #4, Reduction Factors (RFs) Used in Geosynthetic Design

BACKGROUND:

Dominion is closing the Upper (East) Pond at the Chesterfield Station in Chesterfield County, Virginia and will install a cap and cover. The cap will consist of a liner system and will include from bottom to top:

-Subgrade (existing ground, regraded CCR material, or a cushion (nonwoven) geotextile);
-40 mil low-linear density polyethylene (LLDPE) geomembrane; and
-Geocomposite drainage net (GDN) with non-woven, needle punched geotextile heat bonded to both sides.
-Final cover above the cap will consists of 24" of a soil cover layer.

The GDN will collect and convey any infiltrated stormwater to cap drains. This calculation will determine if the proposed GDN has the capacity to handle the full PMF stormwater quantity and prevent saturation of the cover soils located on the closed sideslopes and benches.

The critical areas for the GDN are located in the 33% sideslopes of the impoundment (highest potential for failure of the cap and/or cover). The side slopes will have a geometry of 3H:1V and a bench every 25' in vertical height. Stability of the closed sideslopes are covered in a separate calculation.

SUBJECT CLOSURE OF UPPER (EAST) POND

GEOCOMPOSITE DRAINAGE NET (GDN) DESIGN

BY TIM DATE 5/14/2015 PROJ. NO. C150035.00

CHKD. BY JRK DATE 12/10/2015 SHEET NO. 2 OF 9

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ANALYSIS:

To determine if the proposed GDN has adequate capacity, GAI determined the infiltration rate through the cover soil to the GDN. GAI used prior Hydrologic Evaluation of Landfill Performance (HELP) models to develop the percolation rate. The rate could be limited by the permeability of the cover soil. The permeability of the cover soil was equal to a value of 1.0×10^{-3} cm/s. From a separate calculation, the infiltration rate into the GDN was estimated to be 7.7×10^{-5} cm/sec (2.77 mm/hr).

The next step is to determine the required transmissivity of the GDN. The formula to determine the required transmissivity is:

$$\theta_{required} = PERC * L / \sin(B)$$

Where,

$\theta_{required}$	=	transmissivity (m^2/sec)
PERC	=	percolation rate (mm/sec/unit width);
L	=	slope length (maximum undrained horizontal, m); and
B	=	slope angle.

The critical geometry of the side slopes will be a 3H:1V slope with a maximum undrained length of 200 feet (61.0 m). A length of 215' is used in the analysis as this represents the longest slope to the nearest cap drain, which will transmit any water on the bench to the closest slope drain. The benches will have a slope of 2% and have a width of 20'.

The allowable transmissivity of the GDN is dependent on the 100-hour transmissivity test and reduction factors (RF) for clogging in the field. Reduction factors are in accordance with Reference 3. RF is calculated on the formula below:

$$RF=1/(RF_{CR}*RF_{CC}*RF_{IN}*RF_{BC})$$

Where,

RF_{CR}	= reduction factor for creep (1.4);
RF_{CC}	= reduction factor for chemical clogging (1.5);
RF_{IN}	= reduction factor for intrusion of geotextile (1.5); and
RF_{BC}	= reduction factor for biological clogging (1.2).

According to Reference 3, the product of the relevant reduction factors will be equal to 7.56 (rounded up to 8.0), which includes a safety factor of 2.0. GAI contacted a GDN manufacturer to obtain 100 hour transmissivities under a normal load of ~230 psf and a slope of 3H:1V. Applying the reduction factors to the 100 hour transmissivity provides the $\theta_{allowable}$.

SUBJECT CLOSURE OF UPPER (EAST) POND

GEOCOMPOSITE DRAINAGE NET (GDN) DESIGN

BY TIM DATE 5/14/2015 PROJ. NO. C150035.00

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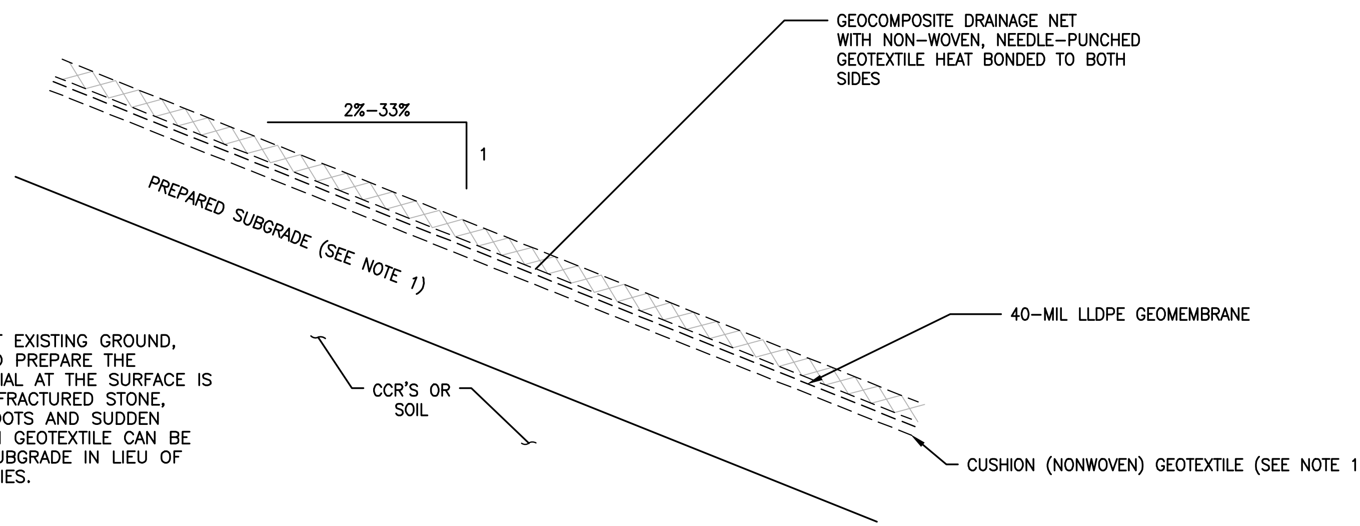
Since a factor of safety is included in the calculation of the reduction factors, the design is considered acceptable if the $\theta_{allowable}$ is greater than the $\theta_{required}$. The attached calculation shows that the $\theta_{allowable}$ is greater than the $\theta_{required}$.

SUMMARY:

The GDN for the proposed cap system for the closure of the Upper (East) Pond will transmit infiltrated stormwater that hits the surface to the surrounding perimeter channel. Based on the proposed geometry of the pond and the soil and material properties listed in the above calculation, the proposed GDN will have the capacity to handle the full PMF rainfall and prevent a condition of saturated cover soils.

ATTACHMENT 1

LINER SYSTEM DETAILS



NOTES:

1. WHERE SUBGRADE CONSISTS OF EXISTING GROUND, STRIP EXISTING VEGETATION AND PREPARE THE SUBGRADE SO THAT THE MATERIAL AT THE SURFACE IS FREE OF PROTRUDING ROCKS, FRACTURED STONE, DEBRIS, COBBLES, RUBBISH, ROOTS AND SUDDEN CHANGES IN SLOPE. A CUSHION GEOTEXTILE CAN BE PLACED OVER THE STRIPPED SUBGRADE IN LIEU OF SUBGRADE PREPARATION ACTIVITIES.

CAP SYSTEM DETAIL 1

N.T.S. 5,7,9,11,16,18,20,25,26,27,28,29

ATTACHMENT 2

GDN MANUFACTURERS' PRODUCT DATA

SKAPS Industries
571 Industrial Parkway
Commerce, GA 30529 (U.S.A.)
Phone (706) 336-7000 Fax (706) 336-7007
e-mail: info@skaps.com

SKAPS TRANSNET™ (TN)
HDPE GEOCOMPOSITE 250

SKAPS TRANSNET™ geocomposite consists of SKAPS GeoNet made from HDPE resin with non-woven polypropylene geotextile fabric heat bonded on both sides of the the geonet.

Property	Test Method	Unit	Required Value		Qualifier
			With 6 oz.	With 8 oz.	
Geonet					
Thickness	ASTM D 5199	mil.	250±15	250±15	Range
Carbon Black	ASTM D 4218	%	2 to 3	2 to 3	Range
Tensile Strength	ASTM D 7179	lb/in	50	50	Minimum
Melt Flow	ASTM D 1238[3]	g/10 min.	1	1	Minimum
Density	ASTM D 1505	g/cm^3	0.94	0.94	Minimum
Transmissivity[1]	ASTM D 4716	m^2/sec.	2.5x10^{-3}	2.5x10^{-3}	MARV[2]
Composite					
Ply Adhesion (Minimum)	ASTM D7005	lb/in	0.5	0.5	MARV
Ply Adhesion (Average)	ASTM D7005	lb/in	1	1	MARV
Transmissivity[1]	ASTM D 4716	m^2/sec	2x10^{-4}	2x10^{-4}	MARV
Geotextile					
Fabric Weight	ASTM D 5261	oz/yd^2	6	8	MARV
Grab Strength	ASTM D 4632	lbs	160	225	MARV
Grab Elongation	ASTM D 4632	%	50	50	MARV
Tear Strength	ASTM D 4533	lbs	65	90	MARV
Puncture Resistance	ASTM D 4833	lbs	95	130	MARV
CBR Puncture	ASTM D 6241	lbs	475	650	MARV
Water Flow Rate	ASTM D 4491	gpm/ft^2	125	100	MARV
Permittivity	ASTM D 4491	sec^{-1}	1.63	1.26	MARV
Permeability	ASTM D 4491	cm/sec	0.3	0.3	MARV
AOS	ASTM D 4751	US Sieve	70	80	MARV

Notes:
1. Transmissivity measured using water at 21 ± 2°C (70 ± 4°F) with a gradient of 0.1 and a confining pressure of 10000 psf between stainless steel plates after 15 minutes. Values may vary between individual labs.
2. MARV is statistically defined as mean minus two standard deviations and it is the value which is exceeded by 97.5% of all the test data.
3. Condition 190/2.16

ATTACHMENT 3

GDN CAPACITY CALCULATIONS

DLC ANALYSIS CASE

Project: Dominion Chesterfield Station

Purpose: Check capacity of Geocomposite Drainage Net (GDN) and find the Drainage Layer Capacity (DLC).

Reference #1: The Design of Drainage Systems over Geosynthetically Lined Slopes, Te-Yang Soong & Robert M. Koerner, June 1997.

Reference #2: Lessons Learned From Failure: Developing Better Drainage Systems for Cover Side Slo|

Find percolation rate through cover to GDN:

PERC = P(1-RC), for P(1-RC) < k_{cs}
PERC = k_{cs}, for P(1-RC) > k_{cs}

K_{cs} =	permeability of cover soil
RC =	Runoff Coefficient
P(1-RC) =	Precipitation (PMF from KMB calc. it is 2.77 mm/hr)

K_{cs} =	1.0E-03	cm/sec	=	0.0100	mm/sec
P =	2.77	mm/hr			
RC =	0				
P(1-RC) =	7.7E-04	mm/sec			
PERC =	7.7E-04	mm/sec			

Find Θ required

$\Theta_{required}$ = q required / i = PERC * .001 m/mm * L/Sinβ

i =	hydraulic gradient (assume continuous slope for entire length)
L =	Slope Length (maximum undrained horizontal)
w=	unit width (1 m)
β=	slope angle

L=	65.532	m			
β=	18.4		=	0.3211	rad
Θ required =	1.6E-04	m^2/sec			

Find Θ allowable

$\Theta_{allowable} = \Theta_{100} * RF$

Θ_{100}=	100 hour transmissivity of GDN at a hydraulic gradient of 0.33 and a load of 230 psf
RF =	Reduction Factor for Clogging = RF = $(1/(RF_{CR}*RF_{CC}*RF_{BC}))$

The combined reduction factor for creep, chemical and biological clogging, intrusion of geotextile, with a FS of 2.0, is equal to 8.0, accoridng to Reference 3

Θ_{100}=	1.44E-03	m^2/sec
RF =	0.125	

$\Theta_{allowable}$ = 1.8E-04 m^2/sec
The Θ of the selected GDN is adequate.

APPENDIX F
Hydrologic and Hydraulic Calculations

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transforming ideas into reality

This appendix contains the following hydrologic and hydraulic calculations for the Upper (East) Pond:

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - HYDROLOGIC AND HYDRAULIC CALCULATIONS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

CHKD. BY CRM DATE 12/10/2105 SHEET NO. 1 OF 1

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HYDROLOGIC AND HYDRAULIC CALCULATIONS

The Upper (East) Pond at the Chesterfield Power Station is to be closed in accordance with the U.S. Environmental Protection Agency's CCR Rules and the Virginia Department of Environmental Quality's Solid Waste Regulations. In addition, the pond is a dam regulated by the Virginia Department of Conservation and Recreation.

The Upper (East) Pond is an impoundment that has been modified to dispose of CCR material in a dry method rather than via slurry. The existing pond embankment/dikes have been maintained and define the dam structure.

This calculation set will design and evaluate the hydrology and hydraulics for the Upper (East) Pond closure. Specifically, this set of calculations includes:

- Precipitation determination, for various storm events and for the Probable Maximum Precipitation (PMP)
- Hydrologic and hydraulic design of perimeter channels and site slope drains

The design event for any channel along the pond perimeter will be the Probable Maximum Flood (PMF) event, so that the PMF event can be contained within the channel without overtopping the pond embankment.

All calculations in this set are based on UEP closure grading using the projected CCR placement volume under current Chesterfield Power Station operating conditions. If the projected CCR placement volume changes due to variable Station operating conditions, the hydrologic and hydraulic calculations will be revised and submitted to the Virginia Department of Environmental Quality.

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - STORM EVENT PRECIPITATION

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

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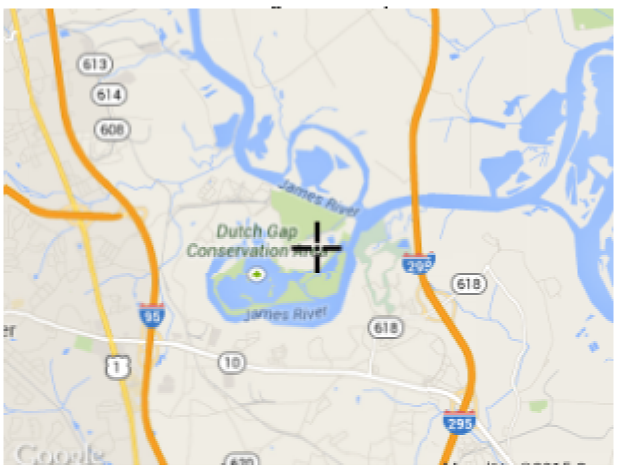
INTRODUCTION

Tabulate the precipitation quantities at the Chesterfield Power Station for various return intervals.

The design storm event for closure based on Virginia Department of Environmental Quality criteria will be the 25-year 24-hour storm. The permitted closure plan used a precipitation of 6.2 inches for this event.

Evaluate more recent rainfall data. From NOAA's Atlas 14, at the marked location:



PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
24-hr	2.78 (2.54-3.06)	3.36 (3.08-3.71)	4.31 (3.93-4.76)	5.11 (4.65-5.64)	6.31 (5.69-6.94)	7.33 (6.57-8.05)	8.45 (7.52-9.27)	9.70 (8.55-10.6)	11.6 (10.0-12.6)	13.1 (11.3-14.4)

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - STORM EVENT PRECIPITATION

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The Atlas 14 values for storm events are:

1-year precipitation = 2.78 inches
2-year precipitation = 3.36 inches
5-year precipitation = 4.31 inches
10-year precipitation = 5.11 inches
25-year precipitation = 6.31 inches
50-year precipitation = 7.33 inches
100-year precipitation = 8.45 inches
200-year precipitation = 9.70 inches
500-year precipitation = 11.6 inches
1000-year precipitation = 13.1 inches

INTRODUCTION

The Upper (East) Pond at the Chesterfield Power Station is classified as a dam. Using National Oceanic and Atmospheric Administration/National Weather Service documents, this calculation will determine the Probable Maximum Precipitation (PMP) at the site for a variety of storm durations.

PRECIPITATION

The National Weather Service's *Hydrometeorological Report No. 51, "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian"* contains charts that show the PMP for watersheds of various sizes and for various durations. The charts for the PMP for a 10-square mile watershed are attached on the next 3 pages, and show the PMP for a 6-hour, 12-hour, 24-hour, 48-hour, and 72-hour storm.

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - PMP PRECIPITATION DISTRIBUTIONS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

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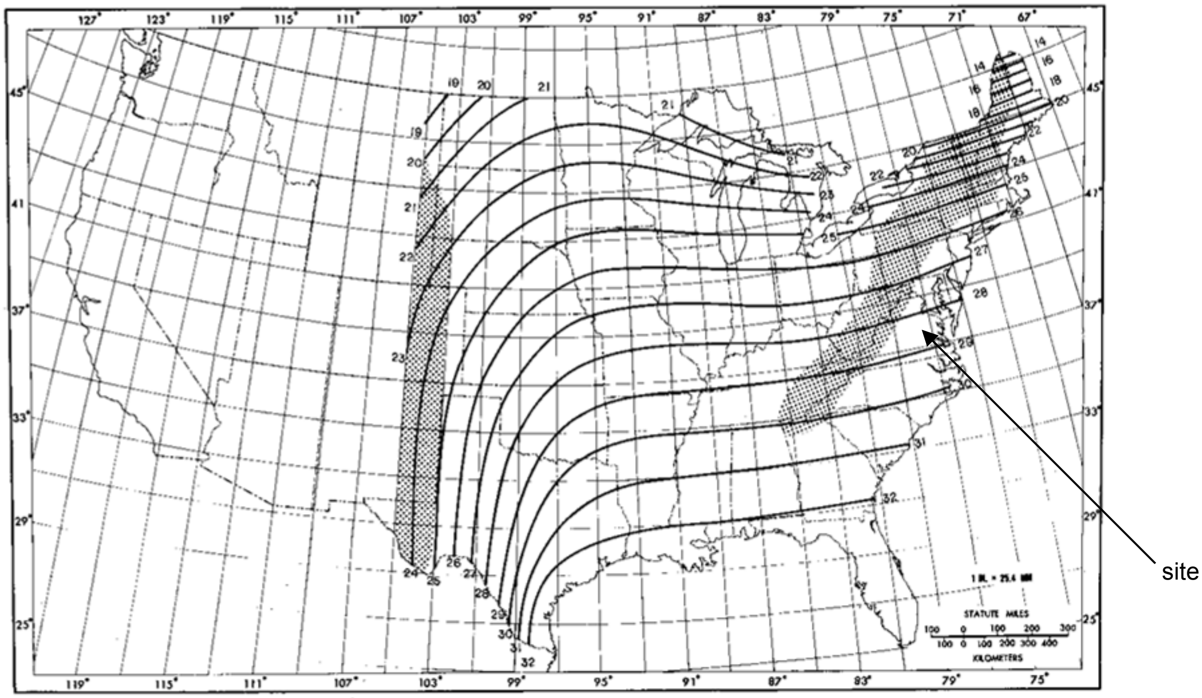

Figure 18.--All-season PMP (in.) for 6 hr 10 mi^2 (26 km^2).

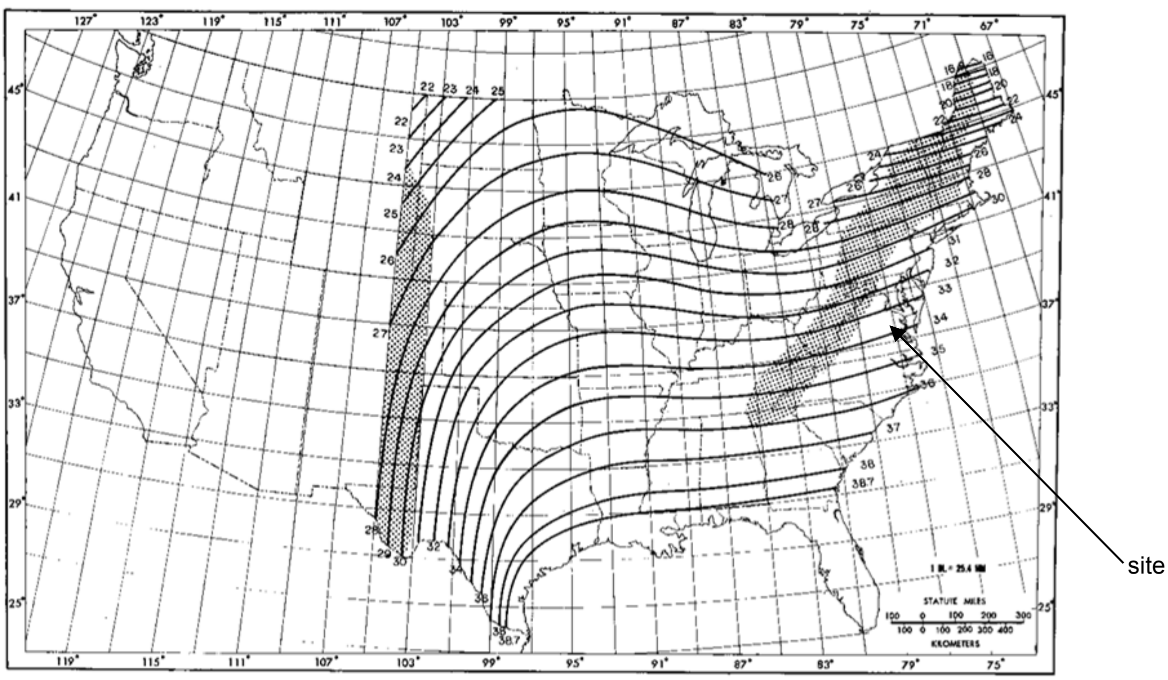

Figure 19.--All-season PMP (in.) for 12 hr 10 mi^2 (26 km^2).

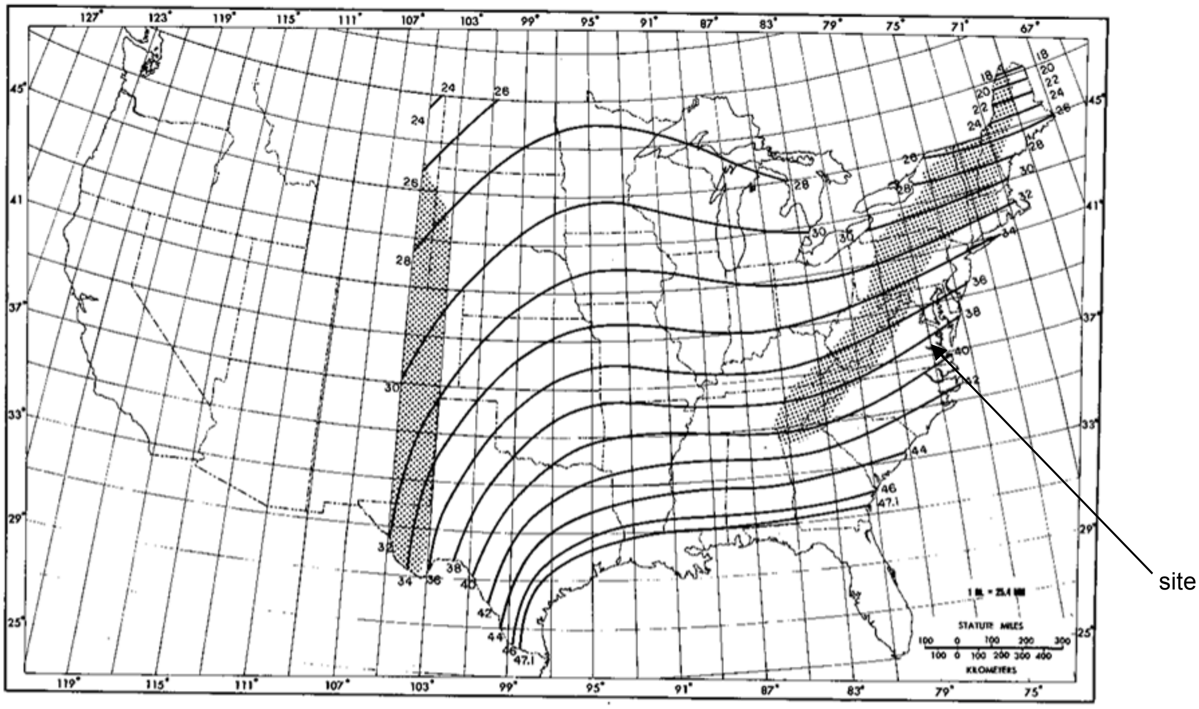

Figure 20.--All-season PMP (in.) for 24 hr 10 mi² (26 km²).

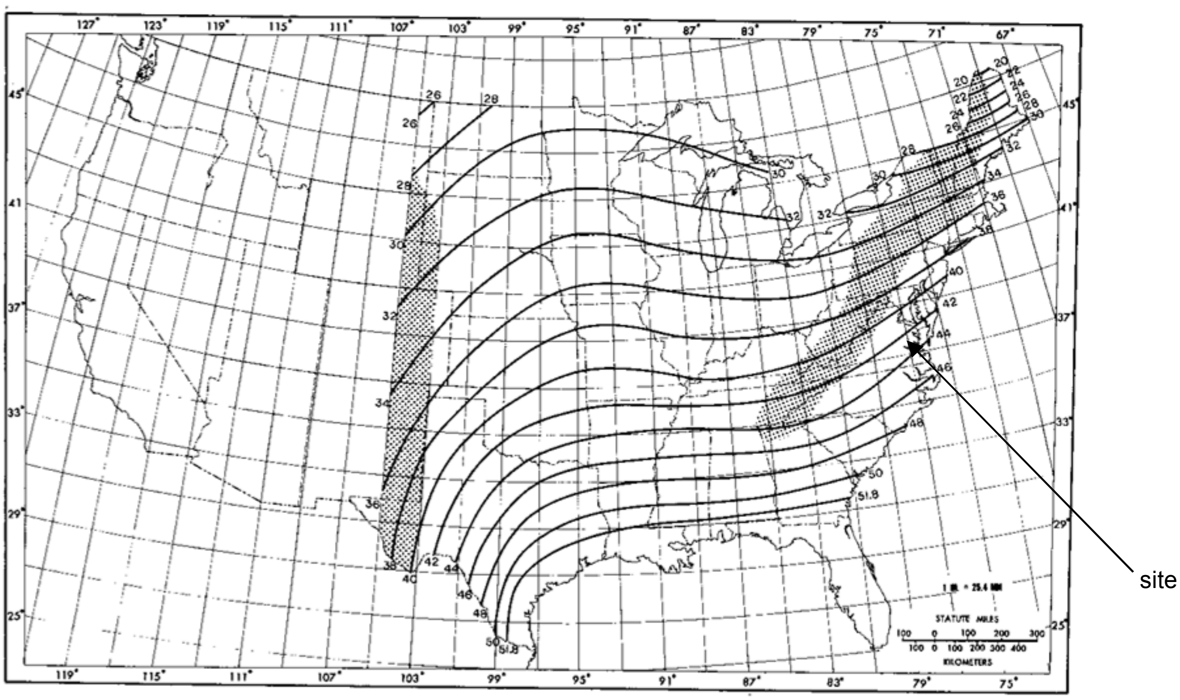

Figure 21.--All-season PMP (in.) for 48 hr 10 mi² (26 km²).

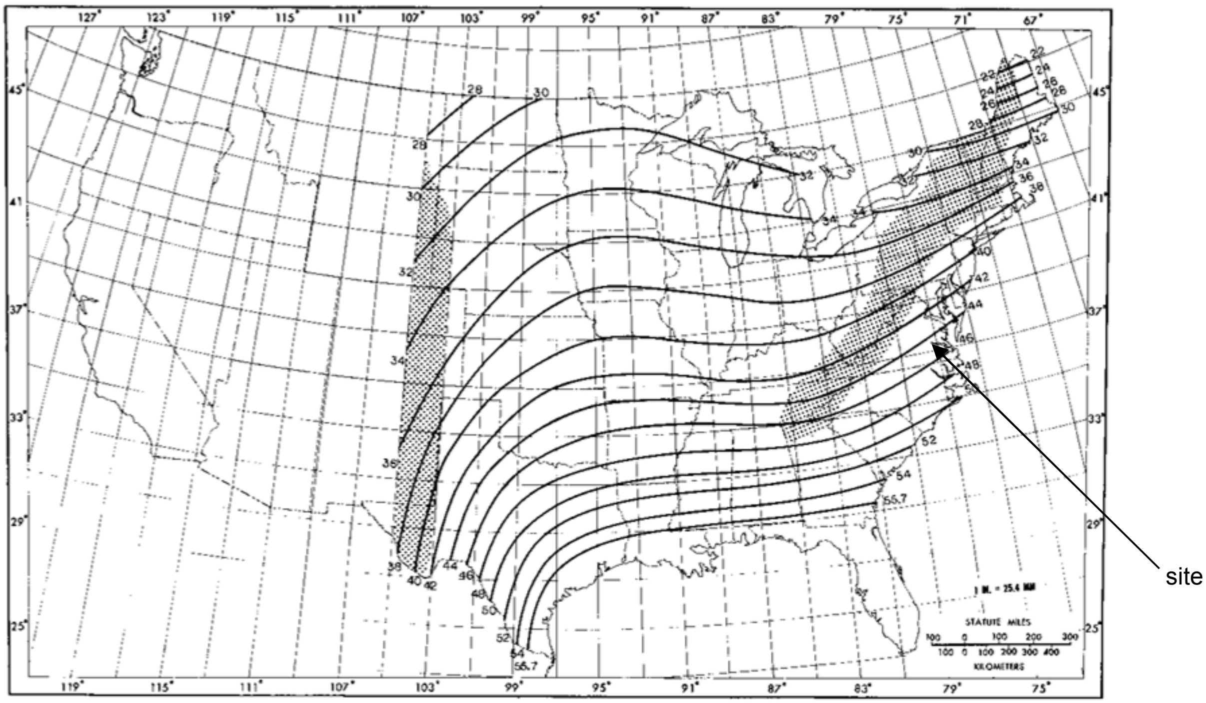

Figure 22.--All-season PMP (in.) for 72 hr 10 mi² (26 km²).

From the above charts, the PMP for a 10 square mile watershed can be summarized as:

6-hour PMP	28.5 inches
12-hour PMP	33.7 inches
24-hour PMP	39.0 inches
48-hour PMP	43.0 inches
72-hour PMP	45.0 inches

PRECIPITATION continued

To perform hydrologic assessments of the PMP event, it is necessary to develop a rainfall mass curve for time intervals less than the 6-hour storm documented previously.

The 1973 edition of *"Design of Small Dams"* (U.S. Bureau of Reclamation) contains the following chart to distribute precipitation for a 6-hour event. As noted, Zone C is appropriate for areas east of the 105° meridian, which is the Mountain Time Zone longitude near Pike's Peak in the Rocky Mountains. The Chesterfield Power Station is east of this location, so the use of Zone C is appropriate.

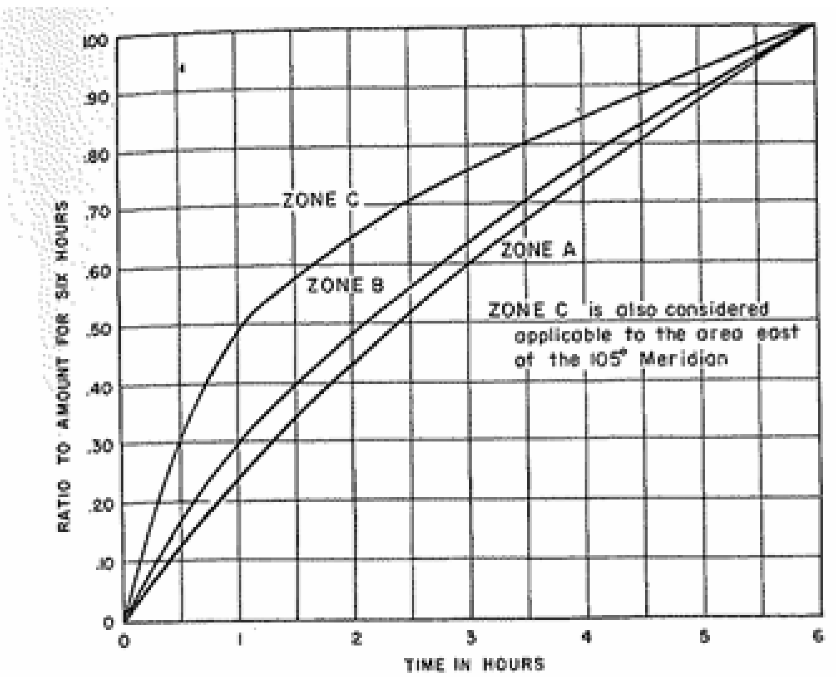

Figure 18. Distribution of 6-hour rainfall for area west of 105° meridian. See figure 17 for area included within each zone. 288-D-2758.

Time	Ratio to 6-hour amount	PMP (inches)
15 minutes	0.15	4.28
1 hour	0.48	13.7
2 hours	0.65	18.5
3 hours	0.75	21.4
6 hours	1.00	28.5

The 12-hour and 24-hour rainfall amounts will be included as documented on sheet 4.

DESIGN EVENT

The Virginia Dam Safety Regulations provide the following requirements for spillway design flows for dams:

TABLE 1 Impounding Structure Regulations			
Applicable to all impounding structures that are 25 feet or greater in height and that create a maximum impounding capacity of 15 acre-feet or greater, and to all impounding structures that are six feet or greater in height and that create a maximum impounding capacity of 50 acre-feet or greater and is not otherwise exempt from regulation by the Code of Virginia.			
Hazard Potential Class of Dam	Spillway Design Flood (SDF)^{B} for New Construction^{F}	Spillway Design Flood (SDF)^{B} for Existing Impounding Structures$^{F, G}$	Minimum Threshold for Incremental Damage Analysis
High	PMF^{C}	0.9 PMP^{H}	100-YR^{D}
Significant	.50 PMF	.50 PMF	100-YR^{D}
Low	100-YR^{D}	100-YR^{D}	50-YR^{E}

B. The spillway design flood (SDF) represents the largest flood that need be considered in the evaluation of the performance for a given project. The impounding structure shall perform so as to safely pass the appropriate SDF.

C. PMF: Probable Maximum Flood is the flood that might be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

D. 100-Yr: 100-year flood represents the flood magnitude expected to be equaled or exceeded on the average of once in 100 years.

E. 50-Yr: 50-year flood represents the flood magnitude expected to be equaled or exceeded on the average of once in 50 years.

F. For the purposes of Table 1 "Existing impounding structure" and "New construction" are defined in 4VAC50-20-30.

G. An existing impounding structure as defined in 4VAC50-20-30, that is currently classified as high hazard, or is subsequently found to be high hazard through reclassification, shall only be required to pass the flood resulting from 0.6 PMP instead of the flood resulting from the 0.9 PMP SDF if the dam owner meets the requirements set out in 4VAC50-20-53.

H. PMP: Probable maximum precipitation means the theoretically greatest depth of precipitation for a given duration that is meteorologically possible over a given size storm area at a particular geographical location at a particular time of year with no allowance made for future long-term climatic trends.

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - PMP PRECIPITATION DISTRIBUTIONS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

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The Upper (East) Pond is currently classified as low hazard. The dam is projected to be considered as a high hazard structure, with a PMF design event.

The PMP precipitation distribution is:

Rainfall Duration	PMP (inches)
15 minutes	4.28
1 hour	13.7
2 hours	18.5
3 hours	21.4
6 hours	28.5
12 hours	33.7
24 hours	39.0

STORM DESIGN DURATION

The Virginia Dam Safety regulations state that the 6-, 12-, and 24-hour design storms must be evaluated and the largest peak outflow used for dam design.

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - UNIFORM SECTION MAT DESIGN PARAMETERS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

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UNIFORM SECTION MAT FABRIC FORMED CONCRETE

Uniform Section Mat (USM) will be proposed as a channel lining at the Upper (East) Pond. Tabulate USM design parameters from manufacturer's literature.

Manning's roughness coefficient

From Texicon, the following is a Manning's roughness range:

Uniform Section Linings are smooth-faced, highly impermeable concrete linings. They reduce the leakage of water, waste products, or other fluids into or out of open channels, landfills, ponds, basins, and containment areas. Uniform Section concrete linings are resistant to most chemicals. The double-layer fabric is vertically connected at closely spaced centers by interwoven drop cords of specified length to form a concrete lining of the desired thickness and weight. With a comparatively smooth and uniform cross section, Uniform Section concrete linings exhibit a relatively low coefficient of hydraulic friction (n = 0.015 to 0.02). Specially designed weep tubes may be inserted through the fabric form, prior to filling, to relieve hydrostatic pressure.

From Armorform,

Table 1. Fabric Formed Concrete Revetment Selection Considerations

FABRIC FORM TYPE	FLOW VELOCITY	BEDLOAD & ICE FORMATIONS	SUB-GRADE SUPPORT	ROUGHNESS COEFF. "n"	WAVE ACTION	UNDERWATER PLACEMENT	ALLOWS SEEPAGE & DRAINAGE
FPM	Low	Light	Well Compacted	.025-.030	Light	Yes	Yes
USM	Low	Light	Well Compacted	.015	Light	Yes	If weep tubes added
ABM	Moderate to High	Light to Heavy	Can tolerate moderate deformation	.045-.050	Light to Heavy	Yes	Yes

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - UNIFORM SECTION MAT DESIGN PARAMETERS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

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Allowable shear stress

From Hydrotex, the following shear stress design limits are provided:

Table 1.0 Typical Dimensions and Weights

Uniform Section	US300	US400	US600	US800	US1000
Average Thickness, in (mm)	3.0 (76)	4.0 (102)	6.0 (152)	8.0 (203)	10.0 (254)
Mass Per Unit Area, lb/ft^2 (kg/m^2)	34 (165)	45 (220)	68 (330)	90 (440)	113 (550)
Drop Point Spacing, in (mm)	3 x 3 (76 x 76)	3 x 4 (76 x 102)	3 x 6 (76 x 152)	4.5 x 7.5 (114 x 191)	4.5 x 9 (114 x 229)
Concrete Coverage, ft^2/yd^3 (m^2/m^3)	100 (12.1)	75 (9.1)	50 (6.1)	38 (4.6)	30 (3.6)
Shear Resistance, lb/ft^2 (kg/m^2)	14 (68)	18 (88)	28 (137)	37 (181)	46 (224)

Note: Values shown are typical and will vary with weight of concrete and field conditions.

And from Texicon:

Select a TEXICON fabric-formed concrete lining or mat style and determine the Permissible Shear Stress, τ_p, from Table 1.0. *The permissible shear stress values presented in Table 1.0 represent stability limits derived from full scale testing of TEXICON fabric-formed concrete linings and mats placed on the beds of steeply-sloped spillways. These values can be used to establish conservative limits for style selection and design.*

Table 1.0 Permissible Shear Stress, τ_p, lb/ft^2

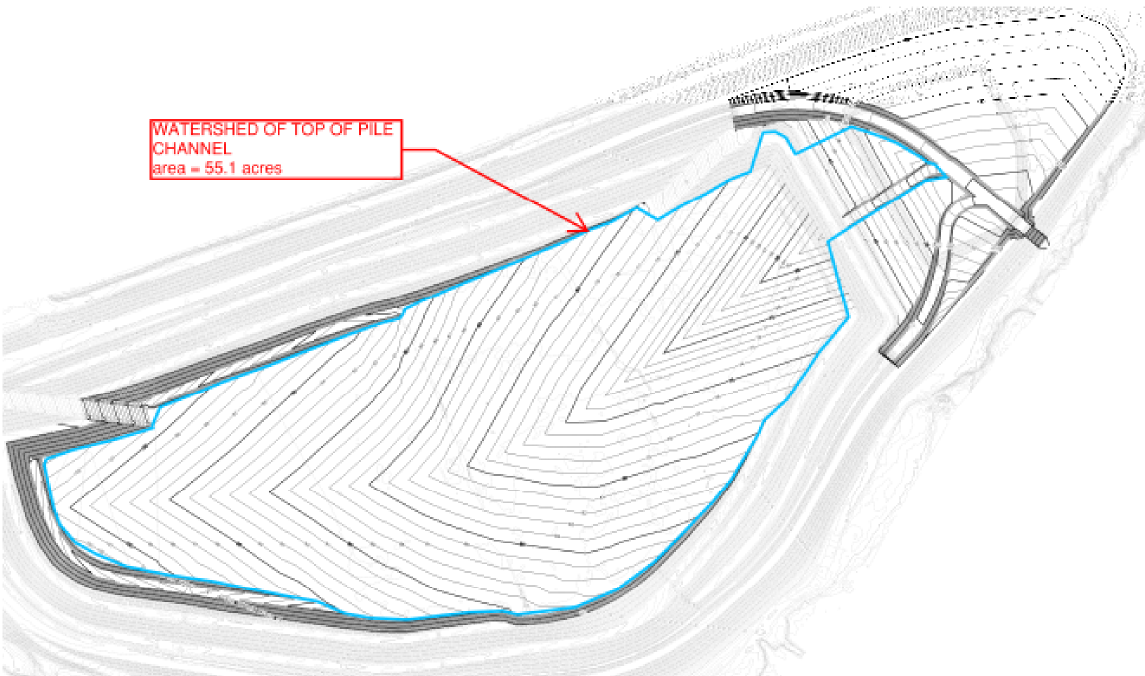
	Average Thickness of Lining or Mat, in						
TEXICON Style	**2.2**	**3.0**	**4.0**	**6.0**	**8.0**	**10.0**	**12.0**
Filter Point	10.9	-	19.9	29.9	39.8	49.8	59.7
Filter Band	-	-	21.0	-	42.0	-	63.0
Uniform Section	-	13.8	18.4	27.6	36.8	46.0	-
Articulating Block	-	-	25.9	38.9	51.8	64.8	77.7

INTRODUCTION

Assess runoff and swale design along the top surface of the closed Upper (East) Pond. Since the Upper (East) Pond is a regulated dam, design the channel to convey flow from a potential dam design event of the Probable Maximum Flood.

CONDITIONS

The proposed top of the Upper (East) Pond will drain to the east. A channel will be formed along the drainage path, and the channel will travel down the east interior face until combining with the north perimeter channel:



Design for post-closure conditions for the dam design event.

- Area = 55.1 acres = 0.0861 square miles
- Curve Number = 74 for reclaimed areas

Time of concentration is calculated on the next page.

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - CENTER CHANNEL

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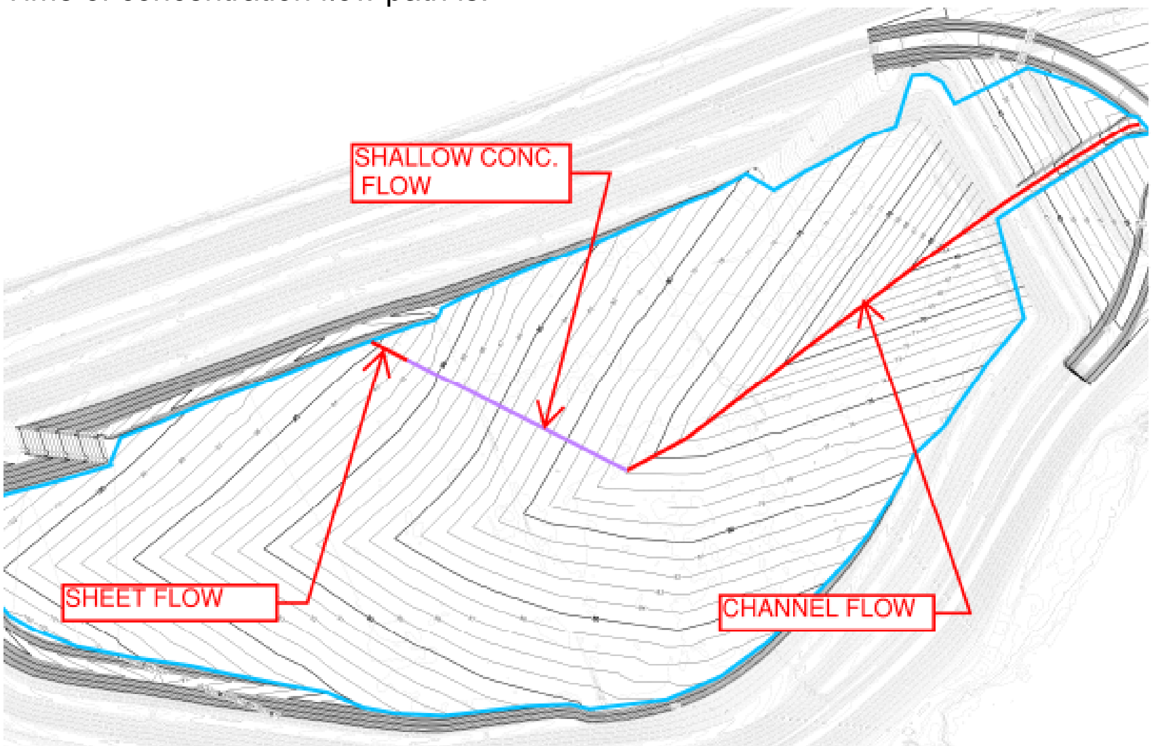
Time of concentration flow path is:



Chesterfield Upper (East) Pond

Time of Concentration

This will calculate the times of concentration for the watersheds.

GIVENS:	2-yr P	Factors for Shallow Concentrated Flow					
	3.36	unpaved	16.1345	paved	20.3282		

For channel flow, assume 10' bottom width 3 ' deep, 3:1 side slopes, USM lining

Area = 57.0 sf
Perimeter = 29.0 ft

WATERSHED	Center Channel						
Sheet Flow	Surface	n	L	Slope	Tt		
covered top	dense grass	0.24	100	0.030	0.20		
Shallow Conc.	Surface	L	S	Calc. V	Tt		
	unpaved	625	0.026	2.6	0.07		
Channel	Area	Perim	L	Slope	n	V	Tt
top of fill	57.0	29.0	1110	0.012	0.015	16.9	0.02
face	57.0	29.0	100	0.210	0.015	71.5	0.00 (average slope)
toe of fill	57.0	29.0	360	0.044	0.015	32.9	0.00
					Tc =	0.29 hr	17.1 minutes

Lag time = 0.6 * time of concentration = 10.3 minutes

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - CENTER CHANNEL

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

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The computer program HEC-HMS will be run to determine flows for a PMF event.

Project: Ches UAP top Simulation Run: Center 6hr PMF

Start of Run: 09Mar2015, 00:00 Basin Model: UAP Center Channel
End of Run: 10Mar2015, 23:00 Meteorologic Model: 6 hour PMP
Compute Time: 15Jun2015, 14:27:40 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
UAP Center Channel	0.0861	864.2	09Mar2015, 03:18	24.01

Project: Ches UAP top Simulation Run: Center 12hr PMF

Start of Run: 09Mar2015, 00:00 Basin Model: UAP Center Channel
End of Run: 10Mar2015, 23:00 Meteorologic Model: 12 hour PMP
Compute Time: 15Jun2015, 14:29:25 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
UAP Center Channel	0.0861	873.1	09Mar2015, 06:18	29.25

Project: Ches UAP top Simulation Run: Center 24hr PMF

Start of Run: 09Mar2015, 00:00 Basin Model: UAP Center Channel
End of Run: 10Mar2015, 23:00 Meteorologic Model: 24 hour PMP
Compute Time: 15Jun2015, 14:30:26 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
UAP Center Channel	0.0861	879.0	09Mar2015, 12:18	34.59

From the above, use a target design for drainage at the top of the landfill of 880 cfs for a PMF event.

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - CENTER CHANNEL

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

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Size a concrete/uniform section mat channel for this flow.
Use Manning's n = 0.015

Maximum slope = 0.33 (3:1 east side interior face, as the channel comes down from the top)
Minimum slope = 0.0125 along the top
Also analyze for slope = 0.025 at the downstream end of the channel

The channel will be sized for the PMF event with no freeboard, or the 25-year event with 0.5-foot freeboard, whichever is greater.

PMF design:

Channel	Center (min slope)	Center (max slope)	Center (d/s end)
Protective Lining	Uniform Section Mat	Uniform Section Mat	Uniform Section Mat
Channel Width at Flow Depth (ft)	32.66	25.04	29.24
Channel Side Slopes (H:V)	3	3	3
Channel Bottom Width (ft)	20	20	20
Flow Depth (ft)	2.11	0.84	1.54
Area (square feet)	55.6	18.9	37.9
Wetted Perimeter (ft)	33.3	25.3	29.7
Hydraulic Radius (ft)	1.67	0.75	1.27
Slope	0.013	0.333	0.040
Manning's n	0.015	0.015	0.015
Velocity at Flow Depth (ft/s)	15.92	47.23	23.36
Flow at Flow Depth (cfs)	884.3	893.4	885.6
Required Capacity (cfs)	879.0	879.0	879.0
Minimum Required Freeboard (ft)	N/A	N/A	N/A
Total Depth Required (ft)	2.11	0.84	1.54
Allowable Velocity (ft/s)	N/A	N/A	N/A
Actual Velocity (ft/s)	15.92	47.23	23.36
Shear Stress at Flow Depth (lb /sf)	1.71	17.47	3.84
Shear Stress Factor of Safety	1.50	1.50	1.50
Design Shear Stress	2.57	26.21	5.77
Required Lining	3" USM	6" USM	3" USM
Max. Allowable Shear Stress (lb/sf)	14.00	28.00	14.00
Use Lining	6" USM	6" USM	6" USM
Hydraulic Depth (ft)	1.70	0.76	1.30
Froude Number	2.15	9.58	3.61

Design depth for this case is 2.11 feet. Use 2.25 feet deep.

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - CENTER CHANNEL

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

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Also evaluate the channel for a 25-year flow plus a minimum 6 inches of freeboard.

Project: Ches UAP top Simulation Run: Center 25yr

Start of Run: 09Mar2015, 00:00 Basin Model: UAP Center Channel
End of Run: 10Mar2015, 23:00 Meteorologic Model: 25-year storm
Compute Time: 15Jun2015, 14:32:11 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

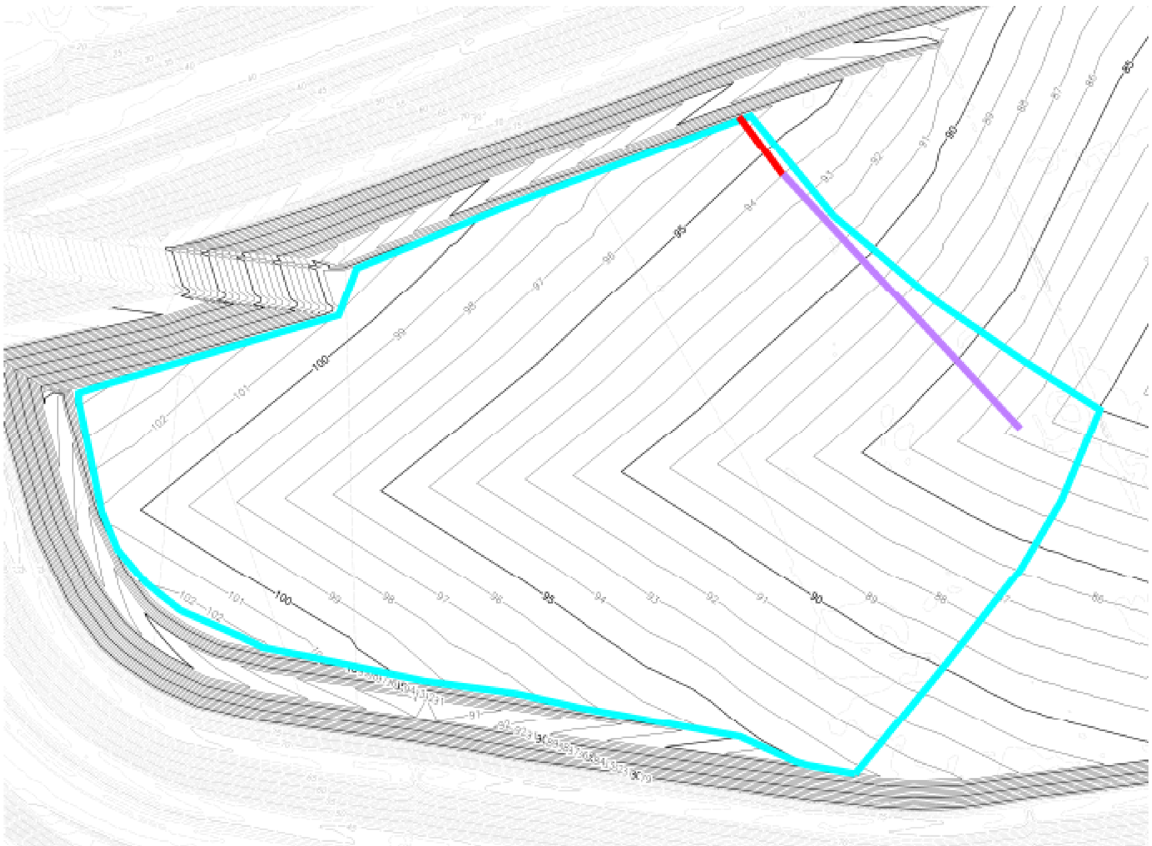
Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
UAP Center Channel	0.0861	233.4	09Mar2015, 12:04	3.45

Channel	Center (min slope)	Center (max slope)	Center (d/s end)
Protective Lining	Uniform Section Mat	Uniform Section Mat	Uniform Section Mat
Channel Width at Flow Depth (ft)	25.94	22.34	24.32
Channel Side Slopes (H:V)	3	3	3
Channel Bottom Width (ft)	20	20	20
Flow Depth (ft)	0.99	0.39	0.72
Area (square feet)	22.7	8.3	16.0
Wetted Perimeter (ft)	26.3	22.5	24.6
Hydraulic Radius (ft)	0.87	0.37	0.65
Slope	0.013	0.333	0.040
Manning's n	0.015	0.015	0.015
Velocity at Flow Depth (ft/s)	10.29	29.42	14.90
Flow at Flow Depth (cfs)	234.0	242.9	237.8
Required Capacity (cfs)	234.0	234.0	234.0
Minimum Required Freeboard (ft)	0.50	0.50	0.50
Total Depth Required (ft)	1.49	0.89	1.22
Allowable Velocity (ft/s)	N/A	N/A	N/A
Actual Velocity (ft/s)	10.29	29.42	14.90
Shear Stress at Flow Depth (lb /sf)	0.80	8.11	1.80
Shear Stress Factor of Safety	1.50	1.50	1.50
Design Shear Stress	1.20	12.17	2.70
Required Lining	3" USM	3" USM	3" USM
Max. Allowable Shear Stress (lb/sf)	14.00	14.00	14.00
Use Lining	6" USM	6" USM	6" USM
Hydraulic Depth (ft)	0.88	0.37	0.66
Froude Number	1.94	8.53	3.24

Design depth for this case is 1.50 feet.
The center channel will be 20 foot bottom width, 2.25 feet deep.

All cases show Froude Number > 1, so no hydraulic jumps are anticipated.

Evaluate the watershed and channel design for the upper half of the Center Channel.



Area = 17.8 acres = 0.0278 sq. miles
Tc = 14.9 minutes; lag time = 8.9 minutes:

GIVENS:	2-yr P	Factors for Shallow Concentrated Flow				
	3.36	unpaved	16.1345	paved	20.3282	
	For the concrete channel, assume 5' bottom width 3 ' deep, 3:1 side slopes					
				Area =	42.0 sf	
				Perimeter =	24.0 ft	

WATERSHED	Center Channel, upper half						
Sheet Flow	Surface	n	L	Slope	Tt		
covered top	dense grass	0.24	100	0.030	0.20		
Shallow Conc.	Surface	L	S	Calc. V	Tt		
	unpaved	480	0.026	2.6	0.05		
					Tc =	0.25 hr	14.9 minutes

For a 24-hour PMF, the design flow is:

Project: Ches UAP top Simulation Run: Center upper half 24hr

Start of Run: 09Mar2015, 00:00 Basin Model: UAP top upper half
End of Run: 10Mar2015, 23:00 Meteorologic Model: 24 hour PMP
Compute Time: 28May2015, 18:36:04 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
UAP upper half	0.0278	286.4	09Mar2015, 12:18	34.59

Evaluate flow in the grass swale formed along the center of the grading. Use a triangular channel with 38:1 side slopes. Use Manning's n = 0.045 for grass, and provide turf reinforcement mat up to the level of the flow depth. Slope of the swale = 0.015. Parameters for the Turf Mat are shown on the next page.

Channel	Center (upper half)	Center (upper half)
Protective Lining	Grass w/TRM	TRM only
Channel Width at Flow Depth (ft)	114.76	93.48
Channel Side Slopes (H:V)	38	38
Channel Bottom Width (ft)	0	0
Flow Depth (ft)	1.51	1.23
Area (square feet)	86.6	57.5
Wetted Perimeter (ft)	114.8	93.5
Hydraulic Radius (ft)	0.75	0.61
Slope	0.015	0.015
Manning's n	0.045	0.026
Velocity at Flow Depth (ft/s)	3.36	5.07
Flow at Flow Depth (cfs)	291.3	291.7
Required Capacity (cfs)	287.0	287.0
Minimum Required Freeboard (ft)	0.50	0.50
Total Depth Required (ft)	2.01	1.73
Allowable Velocity (ft/s)	15.00	9.50
Actual Velocity (ft/s)	3.36	5.07
Shear Stress at Flow Depth (lb /sf)	1.41	1.15
Shear Stress Factor of Safety	1.50	1.50
Design Shear Stress	2.12	1.73
Max. Allowable Shear Stress (lb/sf)	8.00	2.50
Hydraulic Depth (ft)	0.76	0.62
Froude Number	0.68	1.14

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - CENTER CHANNEL

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Allowable velocity of 4 ft/s for grass-lined channels is from Table 3.17-A of the Virginia E&S manual (below). Provide a TRM to provide additional reinforcement to the vegetation to a depth of 2.0 feet above the swale invert.

TABLE 3.17-A

PERMISSIBLE VELOCITIES FOR GRASS-LINED CHANNELS

CHANNEL SLOPE	LINING	PERMISSIBLE VELOCITY[a]
0 - 5%	Bermudagrass	6 ft./second
	Reed canarygrass Tall fescue Kentucky bluegrass	5 ft./second
	Grass-legume mixture	4 ft./second
	Red fescue Redtop Sericea lespedeza Annual lespedeza Small grains (temporary)	2.5 ft./second

[a] For highly erodible soils, permissible velocities should be decreased by 25%. An erodibility factor (K) greater than 0.35 would indicate a highly erodible soil. Erodibility factors (K-factors) for many Virginia soils are listed in Chapter 6.

For North American Green SC-250 lining, the allowable shear stresses and velocities are:

Design Permissible Shear Stress

	Short Duration	Long Duration
Phase 1: Unvegetated	3.0 psf (144 Pa)	2.5 psf (120 Pa)
Phase 2: Partially Veg.	8.0 psf (383 Pa)	8.0 psf (383 Pa)
Phase 3: Fully Veg.	10.0 psf (480 Pa)	8.0 psf (383 Pa)
Unvegetated Velocity	9.5 fps (2.9 m/s)	
Vegetated Velocity	15 fps (4.6 m/s)	

Roughness Coefficients – Unveg.

Flow Depth	Manning's n
≤ 0.50 ft (0.15 m)	0.040
0.50 – 2.0 ft	0.040-0.012
≥ 2.0 ft (0.60 m)	0.011

INTRODUCTION

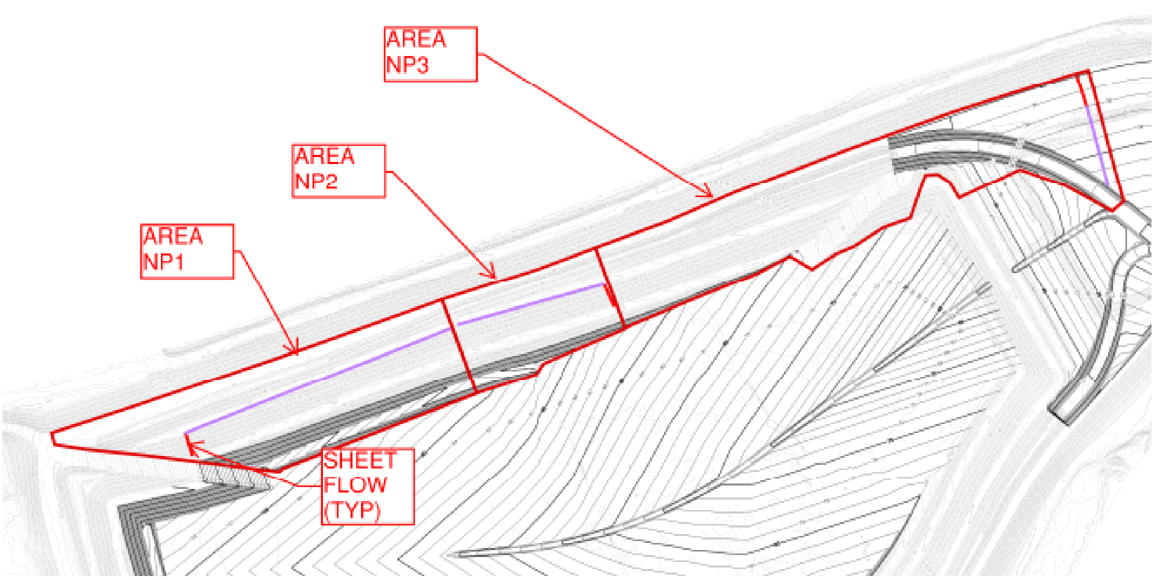
Assess peak flow and channel design for the perimeter channels at the Upper (East) Pond. The channels will be sized for the Probable Maximum Flood (PMF) event with no freeboard or the 25-year event with 0.5-foot freeboard.

CONDITIONS

The north perimeter channel will consist of 3 segments, and the south perimeter channel 4 segments, with the segments divided by slope drain locations. To assess the effect of the PMF design event, assume that flow will cascade down the interior face for areas upstream of slope drains.

The high point in the perimeter channels is currently situated in the middle of the western edge of the site. The high point will be shifted to the western haul road, so that a culvert will not need to be placed under the road.

The north perimeter channel watersheds will consist of:



SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - PERIMETER CHANNELS

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Determine Curve Number by assuming:
- CN = 74 for final reclaimed areas (from 2003 closure package)
- CN = 90 for gravel roads
- CN = 90 for fabric form channels

Channel	Total Area (ac)	Road Length (ft)	Road Width (ft)	Road Area (ac)	Channel Length (ft)	Channel Width (ft)	Channel Area (ac)	Vegetated Area (ac)		CN
NP1	7.6	1290	20	0.59	1150	16	0.42	6.6		76.1
NP2	3.4	500	20	0.23	500	16	0.18	3.0		75.9
NP3	10.2	1650	20	0.76	1720	16	0.63	8.8		76.2
Curve Number				90			90	74		

Channel	Total Area (sq mi)
NP1	0.0119
NP2	0.0053
NP3	0.0159

Time of concentration is calculated on the next page. Assume flow in concrete channels is negligible.

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - PERIMETER CHANNELS

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GIVENS:	2-yr P	Factors for Shallow Concentrated Flow			
	3.36	unpaved	16.1345	paved	20.3282

Benches are 1' deep triangular channels with one 3:1 and one 20:1 side slope.
Use these as the basis for bench flow: Area = 11.5 sf
Perimeter = 23.2 ft
For the concrete channel, assume 2' bottom width 2 ' deep
Area = 12.0 sf
Perimeter = 10.9 ft

WATERSHED	NP1								Slope Calculation		
Sheet Flow	Surface	n	L	Slope	Tt				U/S El.	D/S El.	Length
face	dense grass	0.24	75	0.333	0.06				1	0	3
Channel	Area	Perim	L	Slope	n	V	Tt		U/S El.	D/S El.	Length
bench	11.5	23.2	895	0.014	0.045	2.5	0.10		52.5	40	895
from slope drain					Tc =	0.16	hr	9.7	minutes		
				Lag time = 0.6 tc =				5.8	minutes		

WATERSHED	NP2								Slope Calculation		
Sheet Flow	Surface	n	L	Slope	Tt				U/S El.	D/S El.	Length
face	dense grass	0.24	75	0.333	0.06				1	0	3
Channel	Area	Perim	L	Slope	n	V	Tt		U/S El.	D/S El.	Length
bench	11.5	23.2	475	0.008	0.045	1.9	0.07		44	40	475
from slope drain					Tc =	0.13	hr	7.7	minutes		
				Lag time = 0.6 tc =				4.6	minutes		

WATERSHED	NP3								Slope Calculation		
Sheet Flow	Surface	n	L	Slope	Tt				U/S El.	D/S El.	Length
face	dense grass	0.24	100	0.020	0.23				40	38	100
Shallow Conc.	Surface	L	S	Calc. V	Tt						
bench	unpaved	250	0.020	2.3	0.03				38	33	250
from slope drain					Tc =	0.23	hr	13.9	minutes		
				Lag time = 0.6 tc =				8.4	minutes		

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - PERIMETER CHANNELS

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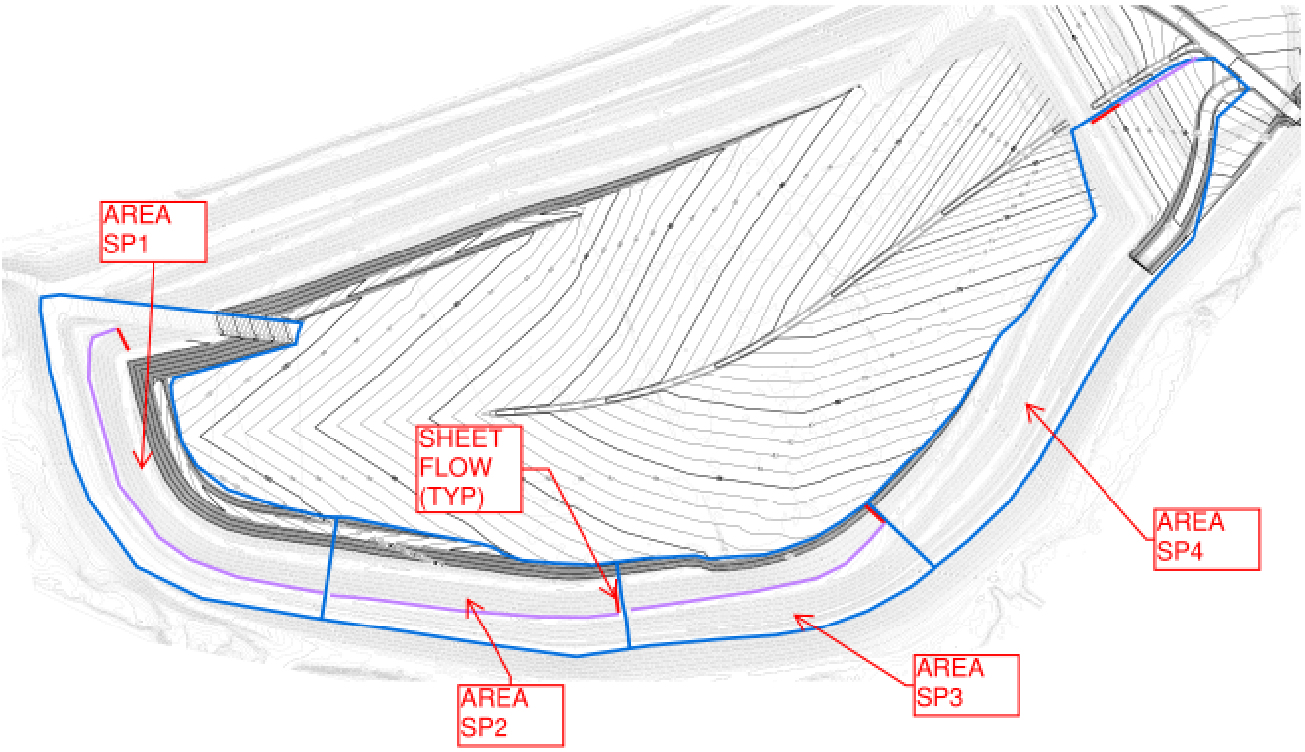
The south perimeter channel watersheds will consist of:



Curve Numbers for the watersheds are:

Channel	Total Area (ac)	Road Length (ft)	Road Width (ft)	Road Area (ac)	Channel Length (ft)	Channel Width (ft)	Channel Area (ac)	Vegetated Area (ac)		CN
SP1	9.8	2000	30	1.38	1250	16	0.46	8.0		77.0
SP2	5.5	900	20	0.41	900	16	0.33	4.8		76.2
SP3	4.9	920	20	0.42	920	16	0.34	4.1		76.5
SP4	10.8	1290	20	0.59	1650	16	0.61	9.6		75.8
Curve Number				90			90	74		

(For area SP1, the haul road is 45’ wide and the perimeter road 20’ wide. Use 30’ as a typical average for the watershed)

Channel	Total Area (sq mi)
SP1	0.0153
SP2	0.0086
SP3	0.0077
SP4	0.0169

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UPPER (EAST) POND - PERIMETER CHANNELS

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Time of concentration calculations:

GIVENS:	2-yr P	Factors for Shallow Concentrated Flow			
	3.36	unpaved	16.1345	paved	20.3282

Benches are 1' deep triangular channels with one 3:1 and one 20:1 side slope.
Use these as the basis for bench flow: Area = 11.5 sf
Perimeter = 23.2 ft
For the concrete channel, assume 2' bottom width 2 ' deep
Area = 12.0 sf
Perimeter = 10.9 ft

WATERSHED	SP1								Slope Calculation		
Sheet Flow	Surface	n	L	Slope	Tt				U/S El.	D/S El.	Length
face	dense grass	0.24	75	0.333	0.06				1	0	3
Channel	Area	Perim	L	Slope	n	V	Tt		U/S El.	D/S El.	Length
bench	11.5	23.2	1213	0.009	0.045	2.0	0.17		55	44	1213
from slope drain					Tc =	0.23	hr	13.8	minutes		
					Lag time = 0.6 tc =			8.3	minutes		

WATERSHED	SP2								Slope Calculation		
Sheet Flow	Surface	n	L	Slope	Tt				U/S El.	D/S El.	Length
face	dense grass	0.24	75	0.333	0.06				1	0	3
Channel	Area	Perim	L	Slope	n	V	Tt		U/S El.	D/S El.	Length
bench	11.5	23.2	840	0.011	0.045	2.1	0.11		52	43	840
from slope drain					Tc =	0.17	hr	10.1	minutes		
					Lag time = 0.6 tc =			6.1	minutes		

WATERSHED	SP3								Slope Calculation		
Sheet Flow	Surface	n	L	Slope	Tt				U/S El.	D/S El.	Length
face	dense grass	0.24	75	0.333	0.06				1	0	3
Channel	Area	Perim	L	Slope	n	V	Tt		U/S El.	D/S El.	Length
bench	11.5	23.2	800	0.013	0.045	2.4	0.09		64	53	840
from slope drain					Tc =	0.15	hr	9.2	minutes		
					Lag time = 0.6 tc =			5.5	minutes		

WATERSHED	SP4								Slope Calculation		
Sheet Flow	Surface	n	L	Slope	Tt				U/S El.	D/S El.	Length
face	dense grass	0.24	100	0.030	0.20				42	39	100
Shallow Conc.	Surface	L	S	Calc. V	Tt						
bench	unpaved	260	0.031	2.8	0.03				39	31	260
from slope drain					Tc =	0.20	hr	11.8	minutes		
					Lag time = 0.6 tc =			7.1	minutes		

MODELING AND CHANNEL DESIGN

The computer program HEC-HMS will be run to flows for a PMF event and for a 25-year event. The PMF event will be run for a 6-hour, 12-hour, and 24-hour duration.

The HEC-HMS summary tables for the North Perimeter channels are on the next page.

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UPPER (EAST) POND - PERIMETER CHANNELS

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Project: Per Channels Simulation Run: North 6hr PMF

Start of Run: 09Mar2015, 00:00 Basin Model: North Perim
End of Run: 10Mar2015, 23:00 Meteorologic Model: 6 hour PMP
Compute Time: 26May2015, 15:42:21 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
North Perimeter Chan...	0.0119	124.4	09Mar2015, 03:16	24.38
North Perimeter Chan...	0.0053	56.0	09Mar2015, 03:16	24.35
Junction-1	0.0172	180.4	09Mar2015, 03:16	24.37
North Perimeter Chan...	0.0159	162.9	09Mar2015, 03:17	24.40
Junction-2	0.0331	342.7	09Mar2015, 03:16	24.38

Project: Per Channels Simulation Run: North 12hr PMF

Start of Run: 09Mar2015, 00:00 Basin Model: North Perim
End of Run: 10Mar2015, 23:00 Meteorologic Model: 12 hour PMP
Compute Time: 28May2015, 13:15:40 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
North Perimeter Chan...	0.0119	125.2	09Mar2015, 06:16	29.63
North Perimeter Chan...	0.0053	56.4	09Mar2015, 06:16	29.59
Junction-1	0.0172	181.6	09Mar2015, 06:16	29.62
North Perimeter Chan...	0.0159	164.2	09Mar2015, 06:17	29.65
Junction-2	0.0331	345.2	09Mar2015, 06:16	29.63

Project: Per Channels Simulation Run: North 24hr PMF

Start of Run: 09Mar2015, 00:00 Basin Model: North Perim
End of Run: 10Mar2015, 23:00 Meteorologic Model: 24 hour PMP
Compute Time: 28May2015, 13:16:03 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
North Perimeter Chan...	0.0119	125.8	09Mar2015, 12:16	34.98
North Perimeter Chan...	0.0053	56.6	09Mar2015, 12:16	34.95
Junction-1	0.0172	182.5	09Mar2015, 12:16	34.97
North Perimeter Chan...	0.0159	165.0	09Mar2015, 12:17	35.00
Junction-2	0.0331	347.0	09Mar2015, 12:16	34.99

Based on the results, use the following design flows:

North Perimeter Channel 1 = 126 cfs
North Perimeter Channel 2 = 183 cfs
North Perimeter Channel 3 = 347 cfs

Consider the 25-year 24-hour rain event:

Project: Per Channels Simulation Run: North 25 year

Start of Run: 09Mar2015, 00:00 Basin Model: North Perim
End of Run: 10Mar2015, 23:00 Meteorologic Model: 25-year SCS
Compute Time: 28May2015, 13:20:03 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
North Perimeter Chan...	0.0119	41.2	09Mar2015, 11:59	3.66
North Perimeter Chan...	0.0053	19.2	09Mar2015, 11:58	3.64
Junction-1	0.0172	60.2	09Mar2015, 11:59	3.65
North Perimeter Chan...	0.0159	49.6	09Mar2015, 12:02	3.67
Junction-2	0.0331	107.5	09Mar2015, 12:00	3.66

25-year flows are:
North Perimeter Channel 1 = 41 cfs
North Perimeter Channel 2 = 60 cfs
North Perimeter Channel 3 = 108 cfs

The existing channels are constructed at 0.4% slope, and the replacement channels at final cover will duplicate this slope. New channel locations will be at 0.5% slope. Use 0.4% for analyses.

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UPPER (EAST) POND - PERIMETER CHANNELS

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PMF design event:

Channel	North Per. 1	North Per. 2	North Per. 3
Protective Lining	Uniform Section Mat	Uniform Section Mat	Uniform Section Mat
Channel Width at Flow Depth (ft)	15.12	17.5	22.22
Channel Side Slopes (H:V)	3	3	3
Channel Bottom Width (ft)	3	4	5
Flow Depth (ft)	2.02	2.25	2.87
Area (square feet)	18.3	24.2	39.1
Wetted Perimeter (ft)	15.8	18.2	23.2
Hydraulic Radius (ft)	1.16	1.33	1.69
Slope	0.004	0.004	0.004
Manning's n	0.015	0.015	0.015
Velocity at Flow Depth (ft/s)	6.94	7.59	8.90
Flow at Flow Depth (cfs)	126.9	183.5	347.8
Required Capacity (cfs)	126.0	183.0	347.0
Minimum Required Freeboard (ft)	N/A	N/A	N/A
Total Depth Required (ft)	2.02	2.25	2.87
Allowable Velocity (ft/s)	N/A	N/A	N/A
Actual Velocity (ft/s)	6.94	7.59	8.90
Shear Stress at Flow Depth (lb /sf)	0.50	0.56	0.72
Shear Stress Factor of Safety	1.50	1.50	1.50
Design Shear Stress	0.76	0.84	1.07
Lining	4" USM	4" USM	4" USM
Max. Allowable Shear Stress (lb/sf)	18.00	18.00	18.00
Hydraulic Depth (ft)	1.21	1.38	1.76
Froude Number	1.11	1.14	1.18

25-year event:

Channel	North Per. 1	North Per. 2	North Per. 3
Protective Lining	Uniform Section Mat	Uniform Section Mat	Uniform Section Mat
Channel Width at Flow Depth (ft)	10.2	11.98	14.96
Channel Side Slopes (H:V)	3	3	3
Channel Bottom Width (ft)	3	4	5
Flow Depth (ft)	1.2	1.33	1.66
Area (square feet)	7.9	10.6	16.6
Wetted Perimeter (ft)	10.6	12.4	15.5
Hydraulic Radius (ft)	0.75	0.86	1.07
Slope	0.004	0.004	0.004
Manning's n	0.015	0.015	0.015
Velocity at Flow Depth (ft/s)	5.18	5.66	6.57
Flow at Flow Depth (cfs)	41.0	60.2	108.8
Required Capacity (cfs)	41.0	60.0	108.0
Minimum Required Freeboard (ft)	0.50	0.50	0.50
Total Depth Required (ft)	1.70	1.83	2.16
Allowable Velocity (ft/s)	N/A	N/A	N/A
Actual Velocity (ft/s)	5.18	5.66	6.57
Shear Stress at Flow Depth (lb /sf)	0.30	0.33	0.41
Shear Stress Factor of Safety	1.50	1.50	1.50
Design Shear Stress	0.45	0.50	0.62
Lining	4" USM	4" USM	4" USM
Max. Allowable Shear Stress (lb/sf)	18.00	18.00	18.00
Hydraulic Depth (ft)	0.78	0.89	1.11
Froude Number	1.04	1.06	1.10

Comparing the results, use the following:

Channel	North Per. 1	North Per. 2	North Per. 3
Bottom width	3	4	5
Side Slopes	3	3	3
PMF required depth	2.02	2.25	2.87
25-year required depth	1.70	1.83	2.16
USE DEPTH =	2.25	2.50	3.25
USE LINING =	4" USM	4" USM	4" USM

The HEC-HMS results for the South Perimeter Channels are:

Project: Per Channels Simulation Run: South 6 hr PMF

Start of Run: 09Mar2015, 00:00 Basin Model: South Perim
End of Run: 10Mar2015, 23:00 Meteorologic Model: 6 hour PMP
Compute Time: 28May2015, 13:59:47 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
South Perimeter Chan...	0.0153	157.3	09Mar2015, 03:17	24.54
South Perimeter Chan...	0.0086	89.7	09Mar2015, 03:17	24.40
Junction-1	0.0239	246.9	09Mar2015, 03:17	24.49
South Perimeter Chan...	0.0077	80.8	09Mar2015, 03:16	24.45
Junction-2	0.0316	327.5	09Mar2015, 03:17	24.48
South Perimeter Chan...	0.0169	174.7	09Mar2015, 03:17	24.33
Junction-3	0.0485	502.2	09Mar2015, 03:17	24.43

Project: Per Channels Simulation Run: South 12hr PMF

Start of Run: 09Mar2015, 00:00 Basin Model: South Perim
End of Run: 10Mar2015, 23:00 Meteorologic Model: 12 hour PMP
Compute Time: 28May2015, 14:09:40 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
South Perimeter Chan...	0.0153	158.4	09Mar2015, 06:17	29.79
South Perimeter Chan...	0.0086	90.3	09Mar2015, 06:16	29.65
Junction-1	0.0239	248.6	09Mar2015, 06:17	29.74
South Perimeter Chan...	0.0077	81.3	09Mar2015, 06:16	29.70
Junction-2	0.0316	329.7	09Mar2015, 06:17	29.73
South Perimeter Chan...	0.0169	176.0	09Mar2015, 06:17	29.57
Junction-3	0.0485	505.7	09Mar2015, 06:17	29.67

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UPPER (EAST) POND - PERIMETER CHANNELS

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Project: Per Channels Simulation Run: South 24hr PMF

Start of Run: 09Mar2015, 00:00 Basin Model: South Perim
End of Run: 10Mar2015, 23:00 Meteorologic Model: 24 hour PMP
Compute Time: 28May2015, 14:11:25 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
South Perimeter Chan...	0.0153	159.2	09Mar2015, 12:17	35.14
South Perimeter Chan...	0.0086	90.7	09Mar2015, 12:16	35.00
Junction-1	0.0239	249.8	09Mar2015, 12:17	35.09
South Perimeter Chan...	0.0077	81.7	09Mar2015, 12:16	35.06
Junction-2	0.0316	331.3	09Mar2015, 12:17	35.08
South Perimeter Chan...	0.0169	176.9	09Mar2015, 12:17	34.93
Junction-3	0.0485	508.2	09Mar2015, 12:17	35.03

Based on the results, use the following design flows:

South Perimeter Channel 1 = 160 cfs
South Perimeter Channel 2 = 250 cfs
South Perimeter Channel 3 = 332 cfs
South Perimeter Channel 3 = 509 cfs

Consider the 25-year 24-hour rain event:

Project: Per Channels Simulation Run: South 25 yr

Start of Run: 09Mar2015, 00:00 Basin Model: South Perim
End of Run: 10Mar2015, 23:00 Meteorologic Model: 25-year SCS
Compute Time: 28May2015, 14:13:25 Control Specifications: Control 1

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
South Perimeter Chan...	0.0153	48.9	09Mar2015, 12:02	3.75
South Perimeter Chan...	0.0086	29.5	09Mar2015, 12:00	3.67
Junction-1	0.0239	77.6	09Mar2015, 12:01	3.72
South Perimeter Chan...	0.0077	27.3	09Mar2015, 11:59	3.70
Junction-2	0.0316	104.1	09Mar2015, 12:00	3.72
South Perimeter Chan...	0.0169	55.0	09Mar2015, 12:01	3.63
Junction-3	0.0485	159.0	09Mar2015, 12:00	3.69

25-year flows are:
South Perimeter Channel 1 = 49 cfs
South Perimeter Channel 2 = 78 cfs
South Perimeter Channel 3 = 105 cfs
South Perimeter Channel 3 = 159 cfs

The existing channels are constructed at 0.4% slope, and the replacement channels at final cover will duplicate this slope. New channel locations will be at 0.5% slope. Use 0.4% for analyses. At the very downstream end, the south channel will be at 2.7% (check for lining stability under the PMF event).

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PMF design event:

Channel	South Per. 1	South Per. 2	South Per. 3	South Per. 4	South Per. 4 max slope
Protective Lining	Uniform Section Mat	Uniform Section Mat	Uniform Section Mat	Uniform Section Mat	Uniform Section Mat
Channel Width at Flow Depth (ft)	16.44	19.32	21.48	25.24	17.92
Channel Side Slopes (H:V)	3	3	3	3	3
Channel Bottom Width (ft)	3	3	3	4	4
Flow Depth (ft)	2.24	2.72	3.08	3.54	2.32
Area (square feet)	21.8	30.4	37.7	51.8	25.4
Wetted Perimeter (ft)	17.2	20.2	22.5	26.4	18.7
Hydraulic Radius (ft)	1.27	1.50	1.68	1.96	1.36
Slope	0.004	0.004	0.004	0.004	0.027
Manning's n	0.015	0.015	0.015	0.015	0.015
Velocity at Flow Depth (ft/s)	7.36	8.24	8.87	9.84	20.05
Flow at Flow Depth (cfs)	160.3	250.2	334.3	509.4	509.9
Required Capacity (cfs)	160.0	250.0	332.0	509.0	509.0
Minimum Required Freeboard (ft)	N/A	N/A	N/A	N/A	N/A
Total Depth Required (ft)	2.24	2.72	3.08	3.54	2.32
Allowable Velocity (ft/s)	N/A	N/A	N/A	N/A	N/A
Actual Velocity (ft/s)	7.36	8.24	8.87	9.84	20.05
Shear Stress at Flow Depth (lb /sf)	0.56	0.68	0.77	0.88	3.91
Shear Stress Factor of Safety	1.50	1.50	1.50	1.50	1.50
Design Shear Stress	0.84	1.02	1.15	1.33	5.86
Lining	4" USM	4" USM	4" USM	4" USM	4" USM
Max. Allowable Shear Stress (lb/sf)	18.00	18.00	18.00	18.00	18.00
Hydraulic Depth (ft)	1.32	1.57	1.76	2.05	1.42
Froude Number	1.13	1.16	1.18	1.21	2.97

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - PERIMETER CHANNELS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

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25-year event

Channel	South Per. 1	South Per. 2	South Per. 3	South Per. 4
Protective Lining	Uniform Section Mat	Uniform Section Mat	Uniform Section Mat	Uniform Section Mat
Channel Width at Flow Depth (ft)	10.86	12.78	14.16	16.66
Channel Side Slopes (H:V)	3	3	3	3
Channel Bottom Width (ft)	3	3	3	4
Flow Depth (ft)	1.31	1.63	1.86	2.11
Area (square feet)	9.1	12.9	16.0	21.8
Wetted Perimeter (ft)	11.3	13.3	14.8	17.3
Hydraulic Radius (ft)	0.80	0.97	1.08	1.26
Slope	0.004	0.004	0.004	0.004
Manning's n	0.015	0.015	0.015	0.015
Velocity at Flow Depth (ft/s)	5.43	6.14	6.62	7.32
Flow at Flow Depth (cfs)	49.3	79.0	105.6	159.5
Required Capacity (cfs)	49.0	78.0	105.0	159.0
Minimum Required Freeboard (ft)	0.50	0.50	0.50	0.50
Total Depth Required (ft)	1.81	2.13	2.36	2.61
Allowable Velocity (ft/s)	N/A	N/A	N/A	N/A
Actual Velocity (ft/s)	5.43	6.14	6.62	7.32
Shear Stress at Flow Depth (lb /sf)	0.33	0.41	0.46	0.53
Shear Stress Factor of Safety	1.50	1.50	1.50	1.50
Design Shear Stress	0.49	0.61	0.70	0.79
Lining	4" USM	4" USM	4" USM	4" USM
Max. Allowable Shear Stress (lb/sf)	18.00	18.00	18.00	18.00
Hydraulic Depth (ft)	0.84	1.01	1.13	1.31
Froude Number	1.05	1.08	1.10	1.13

Comparing the results, use the following:

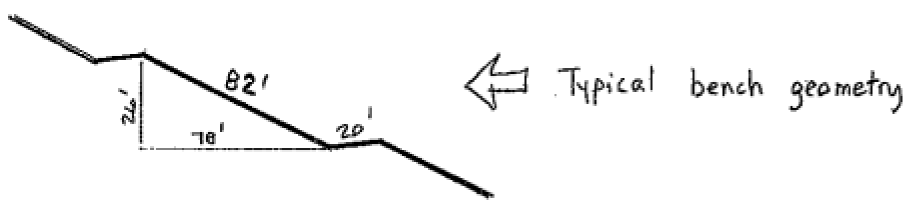
Channel	South Per. 1	South Per. 2	South Per. 3	South Per. 4
Bottom width	3	3	3	4
Side Slopes	3	3	3	3
PMF required depth	2.24	2.72	3.08	3.54
25-year required depth	1.81	2.13	2.36	2.61
USE DEPTH =	2.50	3.00	3.25	3.75
USE LINING =	4" USM	4" USM	4" USM	4" USM

INTRODUCTION

Benches are located in the dry disposal area of the Upper (East) Pond. Evaluate the bench capacity under closed conditions so that slope drain locations can be verified.

CONDITIONS

Benches are located every 25 feet vertically, and are 20 feet wide with a 1 foot vertical drop:



- *Reference, 2003 Closure Plan*

Minimum longitudinal bench slope = 1%
The Upper (East) Pond will be capped and closed. Evaluate bench flow with a vegetated cover, mowed up to 4 times a year.

Runoff Curve Number = 74 (from TR-55; use vegetation > 75% and C soil, as a drainage medium will be placed below the cover soil)

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80

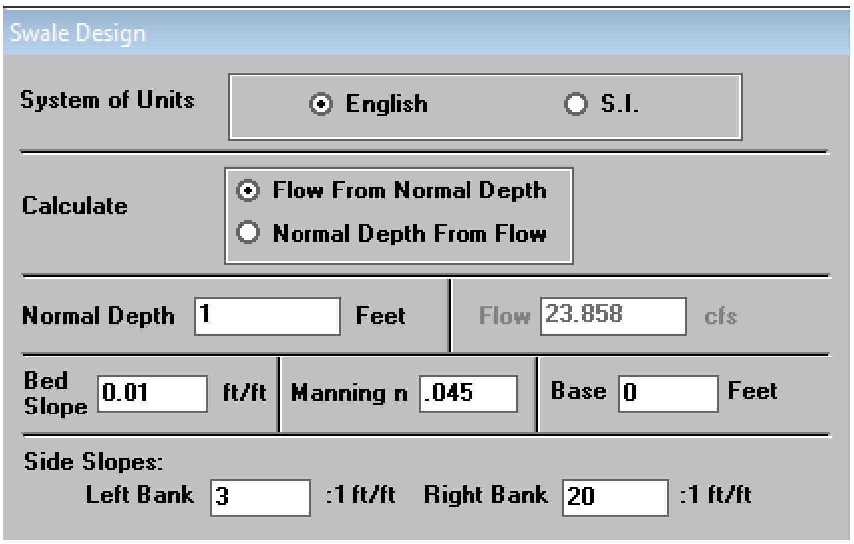
Use Manning's n = 0.045 for bench flow in grass

HYDRAULICS

Bench configuration is as shown on sheet 1. There is 1 foot of flow depth available, with a 20:1 slope on the bench and 3:1 slope on the landfill face.

Estimate the flow capacity of a bench at a 1% slope. Use Manning's n of 0.045 for grassed channels.

Use the computer program VT-PSHUM (Virginia Tech/Penn State Urban Hydrology Model), version 6.0 to estimate the full flow capacity of a bench:



Flow capacity at full depth = 23.8 cfs

HYDROLOGY

Benches are located as shown in the sketch on sheet 1. There is 98 feet of width (26 vertical feet at 3:1 slopes, plus 20 feet of bench) for every longitudinal foot of bench. (Area = 98 sf per foot)

Determine appropriate times of concentration for different bench lengths. The 2-year precipitation is 3.36 inches. Use n = 0.24 (dense grass) for sheet flow conditions. Assume a 2% bench slope for time of concentration purposes.

Chesterfield Upper Ash Pond

Time of Concentration - Bench Flow

This will calculate the times of concentration for the watersheds.

GIVENS:	2-yr P	Factors for Shallow Concentrated Flow						
	3.36	unpaved	16.1345	paved	20.3282			
	Benches are 1' deep triangular channels with one 3:1 and one 20:1 side slope.							
	Use these as the basis for bench flow:			Area =	11.5 sf			
				Perimeter =	23.2 ft			

WATERSHED	Typical face							
Sheet Flow	Surface	n	L	Slope	Tt			
face	grass	0.24	78	0.333	0.06			
Channel	Area	Perim	L	Slope	n	V	Tt	
bench	11.5	23.2	1000	0.020	0.045	2.9	0.09	
					Tc =	0.16 hr		9.4 minutes
Channel	Area	Perim	L	Slope	n	V	Tt	
bench	11.5	23.2	1200	0.020	0.045	2.9	0.11	
					Tc =	0.18 hr		10.5 minutes
Channel	Area	Perim	L	Slope	n	V	Tt	
bench	11.5	23.2	1400	0.020	0.045	2.9	0.13	
					Tc =	0.19 hr		11.7 minutes
Channel	Area	Perim	L	Slope	n	V	Tt	
bench	11.5	23.2	1600	0.020	0.045	2.9	0.15	
					Tc =	0.21 hr		12.8 minutes
Channel	Area	Perim	L	Slope	n	V	Tt	
bench	11.5	23.2	1800	0.020	0.045	2.9	0.17	
					Tc =	0.23 hr		13.9 minutes
Channel	Area	Perim	L	Slope	n	V	Tt	
bench	11.5	23.2	2000	0.020	0.045	2.9	0.19	
					Tc =	0.25 hr		15.1 minutes

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - BENCH FLOW / CAPACITY ASSESSMENT

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

CHKD. BY CRM DATE 12/10/2015 SHEET NO. 4 OF 5

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Summarizing the hydrology:

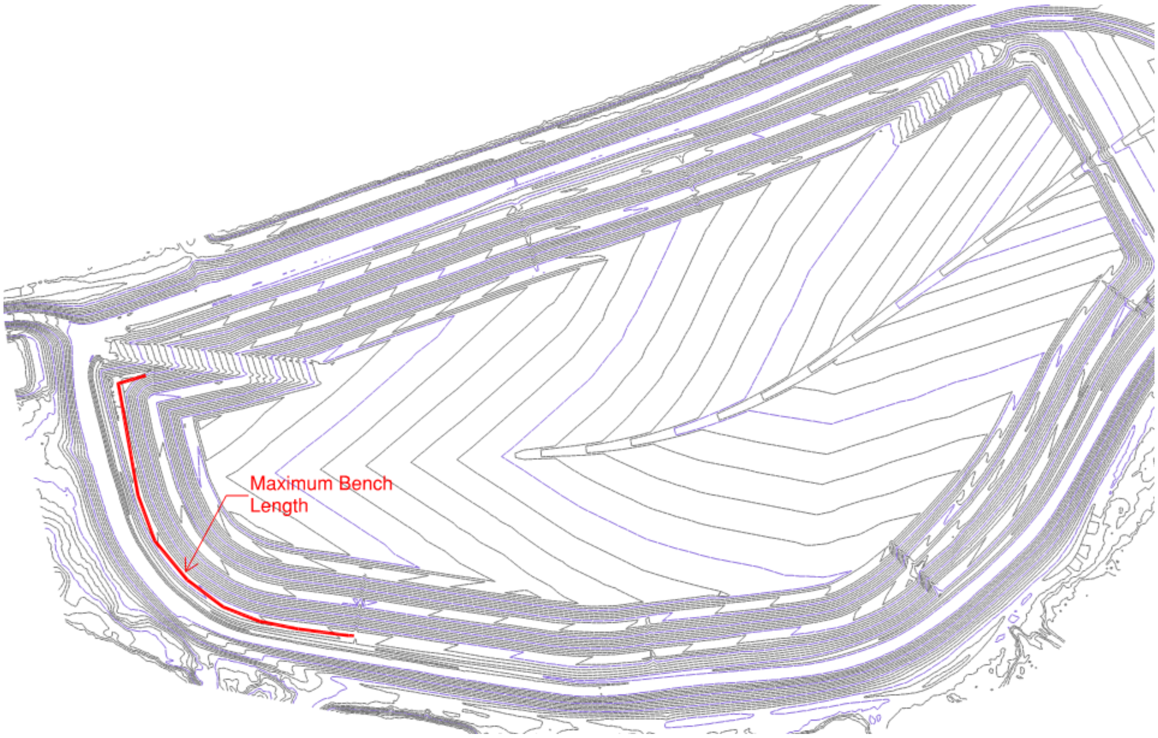
Bench Length (feet)	Area (sf)	Area (ac)	tc (minutes)
1000	98,000	2.25	9.4
1200	117,600	2.70	10.5
1400	137,200	3.15	11.7
1600	156,800	3.60	12.8
1800	176,400	4.05	13.9
2000	196,000	4.50	15.1

Using the above parameters,

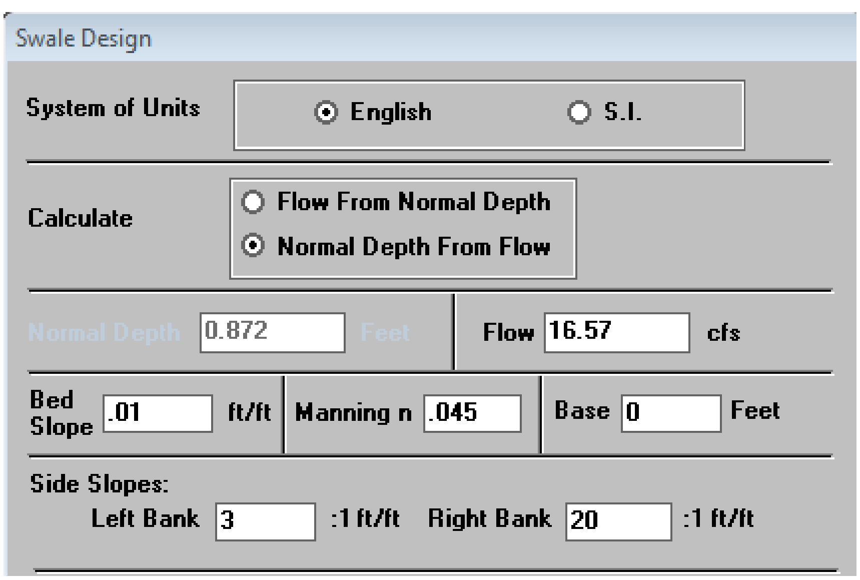
Hydrograph Summary Report

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to peak (min)	Volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Maximum storage (cuft)	Hydrograph description
1	SCS Runoff	12.41	1	719	28,155	----	------	------	1000 foot bench
2	SCS Runoff	14.20	1	720	34,390	----	------	------	1200 foot bench
3	SCS Runoff	16.57	1	720	40,122	----	------	------	1400 foot bench
4	SCS Runoff	17.54	1	721	44,345	----	------	------	1600 foot bench
5	SCS Runoff	18.84	1	722	50,680	----	------	------	1800 foot bench
6	SCS Runoff	20.94	1	722	56,311	----	------	------	2000 foot bench

The longest run of bench in the closure configuration is approximately 1250 feet, as shown below:



Using the flow for a 1400-foot bench, the flow depth is:



The maximum 25-year bench flow will have a freeboard of 0.13 feet. This will be sufficient for bench flow.

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - SLOPE DRAINS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

CHKD. BY CRM DATE 12/10/2015 SHEET NO. 1 OF 3

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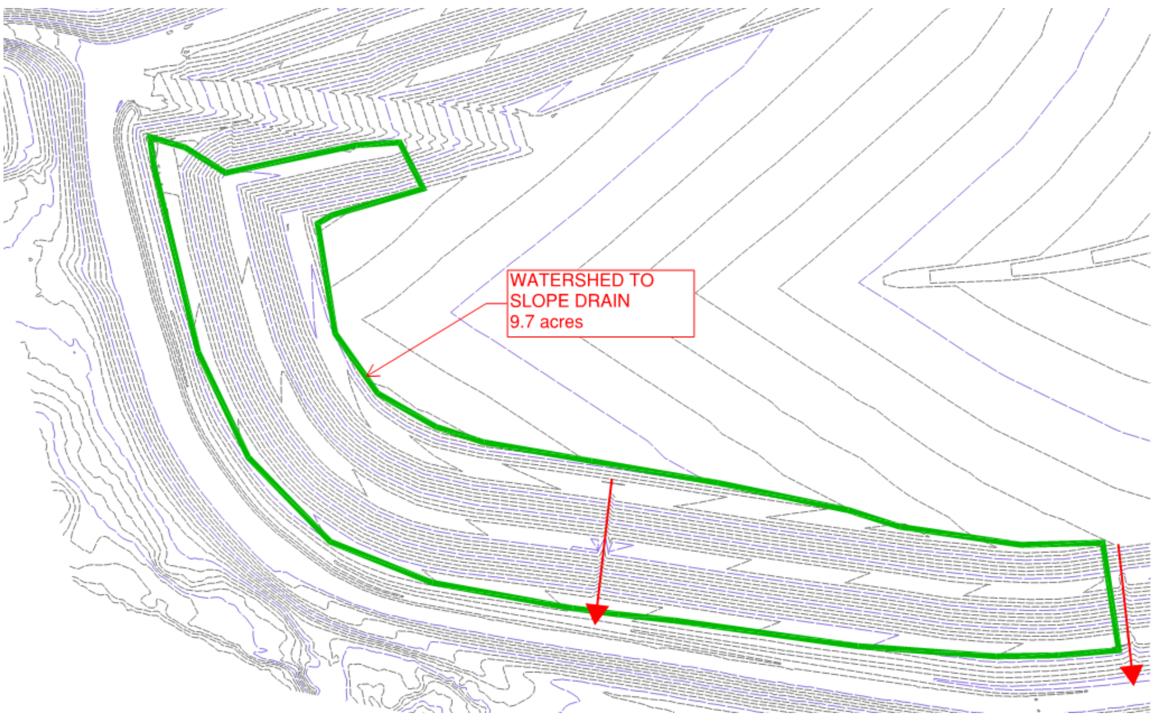
INTRODUCTION

Slope drains intercept the flow along benches in the dry disposal area of the Upper (East) Pond. Design the slope drains for a 25-year storm event based on the maximum area draining to any slope drain.

CONDITIONS

The upstream-most slope drain on the south side of the Upper (East) Pond will receive the most watershed of all slope drains:



Since the design flow for bench capacity is the 25-year flow, size the slope drains for a 25-year flow. Higher storm events will bypass the benches and slope drain and will flow over the landfill face.

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - SLOPE DRAINS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

CHKD. BY CRM DATE 12/10/2015 SHEET NO. 2 OF 3

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Since the flow to the slope drains will be directed along benches, use the same time of concentration as developed for a 1200-foot bench length.

t_c = 10.5 minutes

Runoff Curve Number = 74 for closed conditions

Rainfall = 6.31 inches

Design flow = 51 cfs:

Hydraflow Hydrographs by Intelisolve Sunday, May 31 2015, 11:9 AM

Hyd. No. 1

Slope drain design

Hydrograph type	= SCS Runoff	Peak discharge	= 51.01 cfs
Storm frequency	= 25 yrs	Time interval	= 1 min
Drainage area	= 9.700 ac	Curve number	= 74
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= USER	Time of conc. (Tc)	= 10.50 min
Total precip.	= 6.31 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

Hydrograph Volume = 123,549 cuft

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - SLOPE DRAINS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

CHKD. BY CRM DATE 12/10/2015 SHEET NO. 3 OF 3

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HYDRAULICS

Current slope drains are concrete channels with 2-foot bottom width and 1.5-foot depth. These will be replaced with fabric form channels having 3:1 side slopes. The channels will have a 3:1 slope on the landfill face and a 2% slope across benches.

Channel	Slope Drain max slope	Slope drain Min slope
Protective Lining	Uniform Section Mat	Uniform Section Mat
Channel Width at Flow Depth (ft)	5.19	8.19
Channel Side Slopes (H:V)	3	3
Channel Bottom Width (ft)	2.25	2.25
Flow Depth (ft)	0.49	0.99
Area (square feet)	1.8	5.2
Wetted Perimeter (ft)	5.3	8.5
Hydraulic Radius (ft)	0.34	0.61
Slope	0.333	0.020
Manning's n	0.015	0.015
Velocity at Flow Depth (ft/s)	27.97	10.07
Flow at Flow Depth (cfs)	51.0	52.1
Required Capacity (cfs)	51.0	51.0
Minimum Required Freeboard (ft)	0.50	0.50
Total Depth Required (ft)	0.99	1.49
Allowable Velocity (ft/s)	N/A	N/A
Actual Velocity (ft/s)	27.97	10.07
Shear Stress at Flow Depth (lb /sf)	10.18	1.24
Shear Stress Factor of Safety	1.50	1.50
Design Shear Stress	15.27	1.85
Lining	4" USM	4" USM
Max. Allowable Shear Stress (lb/sf)	18.00	18.00
Hydraulic Depth (ft)	0.35	0.63
Froude Number	8.32	2.23

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - HAUL AND ACCESS ROAD CHANNELS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

CHKD. BY CRM DATE 12/10/2015 SHEET NO. 1 OF 5

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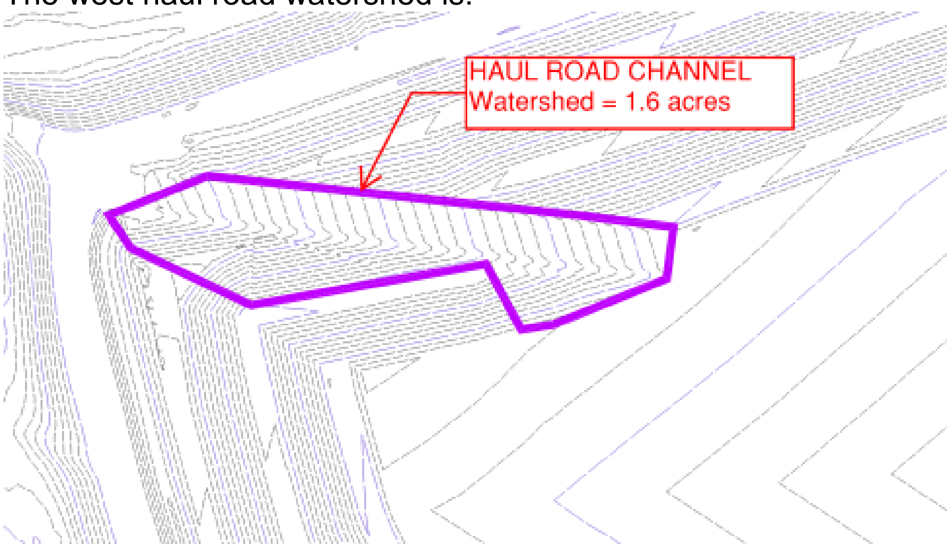
INTRODUCTION

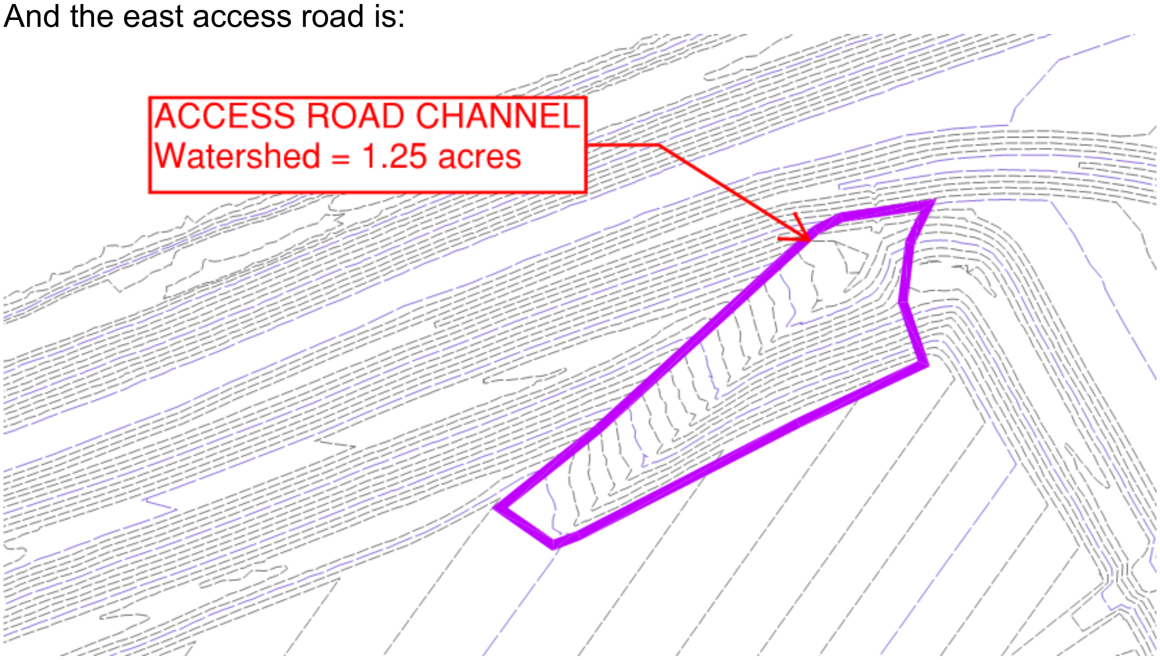
Size the channels along the west haul road and east access road at the Upper (East) Pond. Design the channels for a 25-year storm event.

CONDITIONS

The west haul road watershed is:



And the east access road is:



Design for a Curve Number = 90 for a gravel road surface, and a time of concentration of 5 minutes. Use the 1.6 acre design for both channels.

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - HAUL AND ACCESS ROAD CHANNELS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

CHKD. BY CRM DATE 12/10/2015 SHEET NO. 2 OF 5

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Rainfall = 6.31 inches

Design flow = 14 cfs:

Hydrograph Plot

Hydraflow Hydrographs by Intelisolve

Wednesday, Jun 3 2015, 7:12 PM

Hyd. No. 1

haul road channel

Hydrograph type	= SCS Runoff	Peak discharge	= 13.96 cfs
Storm frequency	= 25 yrs	Time interval	= 1 min
Drainage area	= 1.600 ac	Curve number	= 90
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= USER	Time of conc. (Tc)	= 5.00 min
Total precip.	= 6.31 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

Hydrograph Volume = 30,835 cuft

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - HAUL AND ACCESS ROAD CHANNELS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

CHKD. BY CRM DATE 12/10/2015 SHEET NO. 3 OF 5

gai consultants

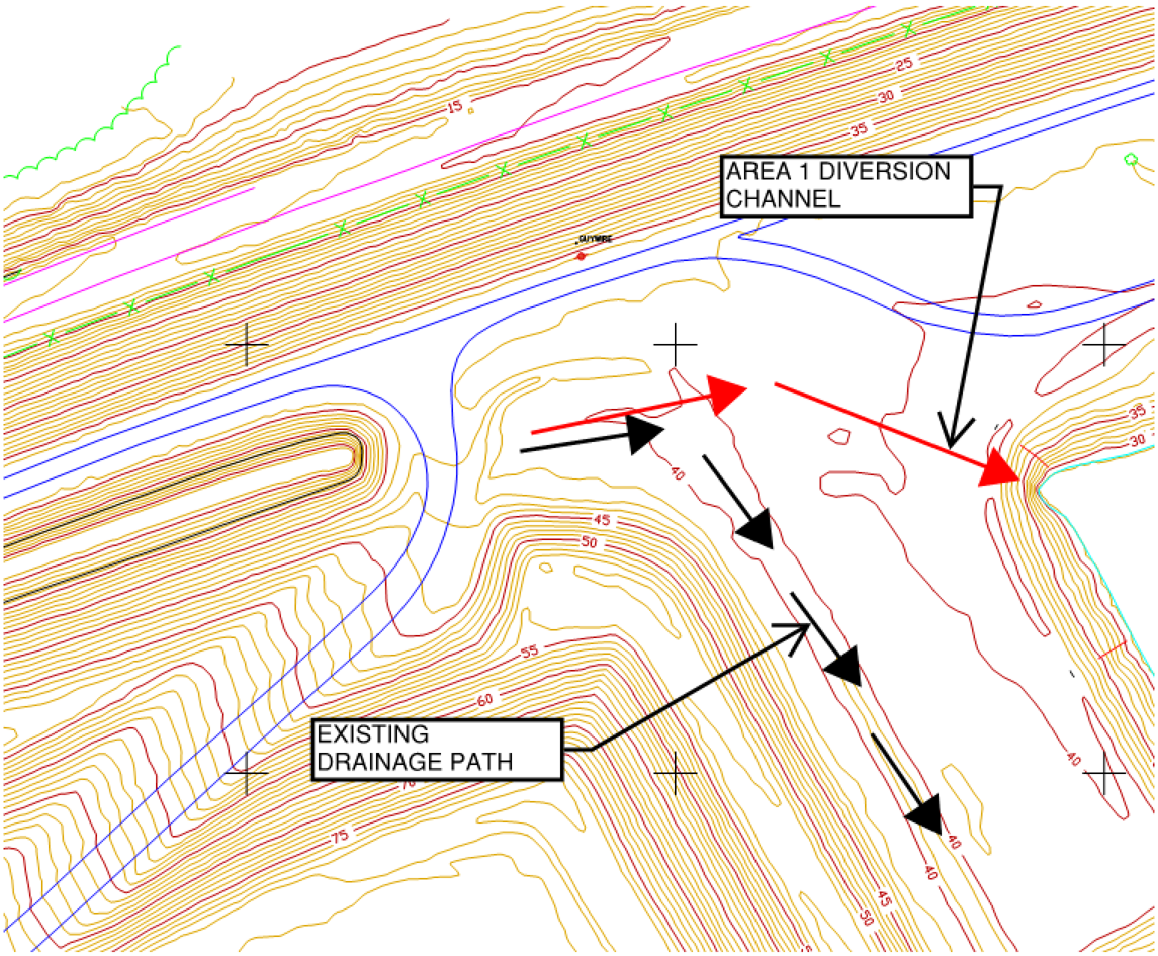
HYDRAULICS

Use a fabric form channels having 3:1 side slopes. Slope of the road = 10%

Channel	Haul Road
Protective Lining	Uniform Section Mat
Channel Width at Flow Depth (ft)	4.16
Channel Side Slopes (H:V)	3
Channel Bottom Width (ft)	2
Flow Depth (ft)	0.36
Area (square feet)	1.1
Wetted Perimeter (ft)	4.3
Hydraulic Radius (ft)	0.26
Slope	0.100
Manning's n	0.015
Velocity at Flow Depth (ft/s)	12.77
Flow at Flow Depth (cfs)	14.2
Required Capacity (cfs)	14.0
Minimum Required Freeboard (ft)	0.50
Total Depth Required (ft)	0.86
Allowable Velocity (ft/s)	N/A
Actual Velocity (ft/s)	12.77
Shear Stress at Flow Depth (lb /sf)	2.25
Shear Stress Factor of Safety	1.50
Design Shear Stress	3.37
Lining	4" USM
Max. Allowable Shear Stress (lb/sf)	18.00
Hydraulic Depth (ft)	0.27
Froude Number	4.36

Use a 2-foot bottom width channel 1 foot deep.

During construction of cap and cover of Area 1 adjacent to the sediment pond, a diversion will be installed at the toe of the eastern access road channel:



Slope of the diversion channel will be 2 feet in 300 = 0.007

SUBJECT DOMINION – CHESTERFIELD POWER STATION
UPPER (EAST) POND - HAUL AND ACCESS ROAD CHANNELS

BY KMB DATE 08/31/2015 PROJ. NO. C150035.00

CHKD. BY CRM DATE 12/10/2015 SHEET NO. 5 OF 5

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Channel	Area 1 Diversion	Area 1 Diversion
Protective Lining	Grass with TRM	TRM only
Channel Width at Flow Depth (ft)	7.68	6.84
Channel Side Slopes (H:V)	2	2
Channel Bottom Width (ft)	3	3
Flow Depth (ft)	1.17	0.96
Area (square feet)	6.2	4.7
Wetted Perimeter (ft)	8.2	7.3
Hydraulic Radius (ft)	0.76	0.65
Slope	0.007	0.007
Manning's n	0.045	0.031
Velocity at Flow Depth (ft/s)	2.30	3.01
Flow at Flow Depth (cfs)	14.4	14.2
Required Capacity (cfs)	14.0	14.0
Minimum Required Freeboard (ft)	0.50	0.50
Total Depth Required (ft)	1.67	1.46
Allowable Velocity (ft/s)	15.00	9.50
Actual Velocity (ft/s)	2.30	3.01
Shear Stress at Flow Depth (lb /sf)	0.51	0.42
Shear Stress Factor of Safety	1.50	1.50
Design Shear Stress	0.77	0.63
Lining	TRM	TRM only
Max. Allowable Shear Stress (lb/sf)	8.00	2.50
Hydraulic Depth (ft)	0.81	0.69
Froude Number	0.45	0.64

Channel depth will be 1.75 feet.

INTRODUCTION

A toe drain pumping and conveyance system will be constructed at the Upper (East) Pond. Model the proposed system.

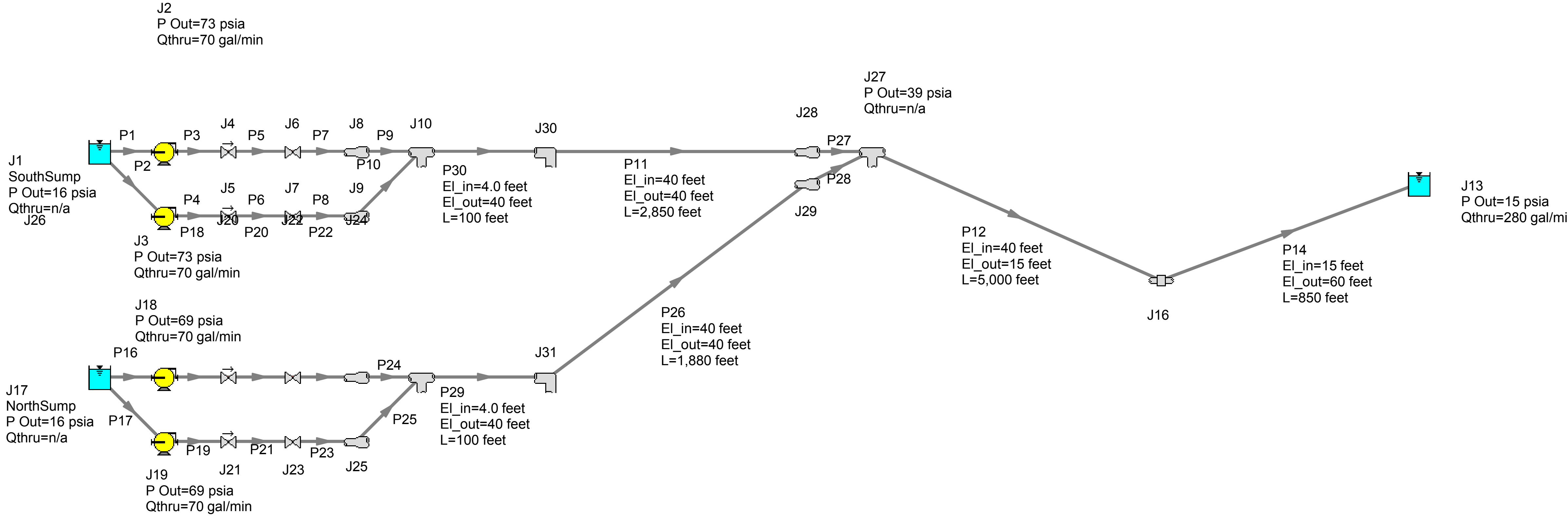
MODEL

The next page depicts the modeled toe drain pumps and force main conveyance.

AFT Fathom Model
C:\Users\BergeMA\Desktop\DominionChesterfield\Chesterfield_N&S_Combination.fth
Base Scenario

CHESTERFIELD UPPER ASH POND

TOE DRAIN PUMPS AND FORCE MAIN MODEL



Fathom 8 (Visual Report) 20-May-15 Page 1 of 1 bergema	AFT Fathom Model	

APPENDIX G
Settlement, Displacement, and Subsidence Calculations

This appendix contains the following geotechnical calculations:

- Settlement Analysis (pages 1-9)
- Bearing Capacity Analysis (10-16)

SUBJECT: DOMINION CHESTERFIELD POWER STATION – UPPER EAST POND CLOSURE - SETTLEMENT ANALYSIS

BY TIM DATE 10/26/2015 PROJ. NO. C150035.00

CHKD. BY MEZ DATE 11/10/2015 SHEET NO. 1 OF 9

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OBJECTIVE:

This calculation was completed to estimate settlement within the existing CCR material of the Upper (East) Pond and evaluate potential impacts on the engineered final cover system for the proposed closure of the Upper (East) Pond at the Chesterfield Power Station.

BACKGROUND:

Dominion is proposing the closure of the Upper (East) Pond located at the Chesterfield Power Station in Chesterfield County, Virginia. The Upper (East) Pond will include an approximate 113 acre geosynthetic cap area.

The proposed closure will include placing fly ash to modify the existing grades to facilitate surface water runoff and reduce ponding. Fly ash will generally be placed in relatively thin lifts spread out over large areas where it will be able to consolidate during placement. Therefore, the calculation presented here will estimate settlement within the existing ash material as it is not anticipated that significant settlement will occur within the fill material.

Reviewing the attached Existing Conditions Plan, the top of the Pond elevations range from 78′ on the east end of the Pond to 92′ on the west end. Based on the attached proposed subgrade plan, the final elevations of ash will range from ~100′ on the west end to ~65′ on the east end. 2′ of cover will be placed on CCR material bringing final elevations to ~102′ on the west and ~67′. The bottom of ash is assumed to be approximately 1.5′. Groundwater elevation is taken to be approximately 1.5′, according to the 2014 Schnabel report (Reference 3).

METHODOLOGY:

Settlement within the existing ash material was estimated using conventional geotechnical engineering methods along with as-built drawings and proposed grading plans to model bottom of existing ash, top of existing ash, and proposed top of subgrade surfaces. The thicknesses of existing ash, proposed ash, and final cover system were estimated to determine the estimated settlement.

The settlement calculations presented here evaluated total settlement in the form of primary and secondary consolidation. After estimating total settlement, differential settlement was examined by evaluating pre-settlement and post-settlement liner slopes.

REFERENCES:

1. "Revised Closure Plan Upper (East) Pond, Chesterfield Power Station". GAI Consultants, September 2003.

2. X.Qian, R.M. Koerner, D.H.Gray. Geotechnical Aspects of Landfill Design and Construction. Prentice Hall, 2002.

3. "Geotechnical Engineering Report: Upper Pond Stability Evaluation, Chesterfield Power Station." Schnabel Engineering Consultants, Inc., August 15, 2014.

CALCULATION:

SUBJECT: DOMINION CHESTERFIELD POWER STATION – UPPER EAST POND CLOSURE - SETTLEMENT ANALYSIS

BY TIM DATE 10/26/2015 PROJ. NO. C150035.00

CHKD. BY MEZ DATE 11/10/2015 SHEET NO. 2 OF 9

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Primary and secondary settlements were evaluated for the in place ash material using the following conventional geotechnical engineering equations for consolidation.

Primary Settlement:
The primary settlement will be treated as a consolidation settlement in soil and is estimated as follows.

$S_c = C_r / (1 + e_o) * (H) * \log(\sigma_f / \sigma_o)$

Where;
- S_c = Primary (consolidation) settlement, (ft)
- C_r = Primary recompression index
- e_o = Initial void ratio
- H = Thickness of layer to be evaluated, (ft)
- σ_f = Total effective vertical stress after loading (middle of layer), (psf)
- σ_o = Effective vertical stress before loading (middle of layer), (psf)

Layer thicknesses and effective stresses were calculated using the attached drawings, along with unit weights obtained from laboratory test data of the ash material and typical values for cover material. The recompression index and void ratio used in the primary settlement equation were estimated from one-dimensional laboratory test results included in Attachment 2 and summarized below.

C_r= 0.02;
e_o=1.0;
σ_f=5,570 psf;
σ_o= 3,530 psf; and
H= 78.5'.

$S_c = 0.02 / (1 + 1.0) * (78.5) * \log(5{,}570 / 3{,}530)$ = 1.9 inches.

Secondary Settlement:
The total primary settlement estimated using the primary consolidation methods described above resulted in minimal primary settlement with the maximum estimated to be up to 2-inches. Therefore, any settlement which may result from secondary consolidation would not have a significant impact on the performance of the proposed final cover system.

Material Properties:
The material properties used in the settlement analysis for the in-place ash material included void ratio, unit weight, and recompression index were obtained from laboratory test results presented in Attachment 2. Unit weights used for the ash / soil fill and final cover soil were estimated as typical unit weights for representative material. The following material properties were used in the settlement equations presented above are summarized as follows:

In-Place Ash
Unit Weight, γ = 90-pcf
Initial Void Ratio, e_o = 1.0
Primary Recompression Index, Cr = 0.02

Soil Fill

Unit Weight, γ = 90-pcf

Final Cover Soil
Unit Weight, γ = 120-pcf

SUMMARY:

This calculation was completed to estimate settlement within the existing in-place ash material resulting from the grading included as part of the proposed Upper (East) Pond Closure. The anticipated settlement of the existing ash is expected to be no more than 2 inches. Based on the amount of anticipated settlement, it is expected that differential settlement should not affect the slopes of the cap system.

Based upon the results of the estimated total settlement, it is not anticipated that settlement of the in-place ash material will significantly impact the proposed final cover system.

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ATTACHMENT 1

DRAWINGS

REFERENCES:

1. BASE MAPPING PROVIDED BY TUCK MAPPING SOLUTIONS, INC. NORTH AMERICAN DATUM 1983 (U.S. FEET). AIRBORNE SURVEY PHOTOGRAPHY FLIGHT FLOWN OCTOBER 8, 2014. COMPLETED APRIL 30, 2015.
2. PROPERTY BOUNDARY IS FROM D&M SURVEYORS, P.C., TAX ID NO. 811-660-3332-0000, 05/08/15.
3. APPROXIMATE RPA BOUNDARY IS FROM GOLDER ASSOCIATES, "WETLAND DELINEATION MAP", DRAWING 1520610F01, 04/03/15.

NOTES:

1. UNDERGROUND ELECTRIC FOLLOWS ONE OF THE ROADS ALONG THE SOUTH POND PERIMETER. ACTUAL LOCATION IS TO BE DETERMINED.
2. ANY FENCING OR UTILITIES DAMAGED OR DISTURBED DURING CONSTRUCTION ACTIVITIES SHALL BE REPLACED AT THE OWNER'S DIRECTION.
3. GRADING SHOWN IN THE ACTIVE DISPOSAL AREA DEPICTS ANTICIPATED CONDITIONS ON JANUARY 1, 2016.

SCALE: 1" = 120'

120 0 120 240

NO.:	DATE:	DWN:	CHK:	APV:	DESCRIPTION:

REVISION RECORD

This drawing was produced with computer aided drafting technology and is supported by electronic drawing files. Do not revise this drawing via manual drafting methods.

GAI CAD FILE PATH: Z:\Energy\2015\C150035.00 – DOM-Chesterfld Pond Closu\CAD\Production Drawings\C150035-00-000-00-C-E1-002.dwg PLOTTED ON: 7/17/2015 2:42:23 PM PLOTTED BY: William Harris PLOT FILE: GAI.stb

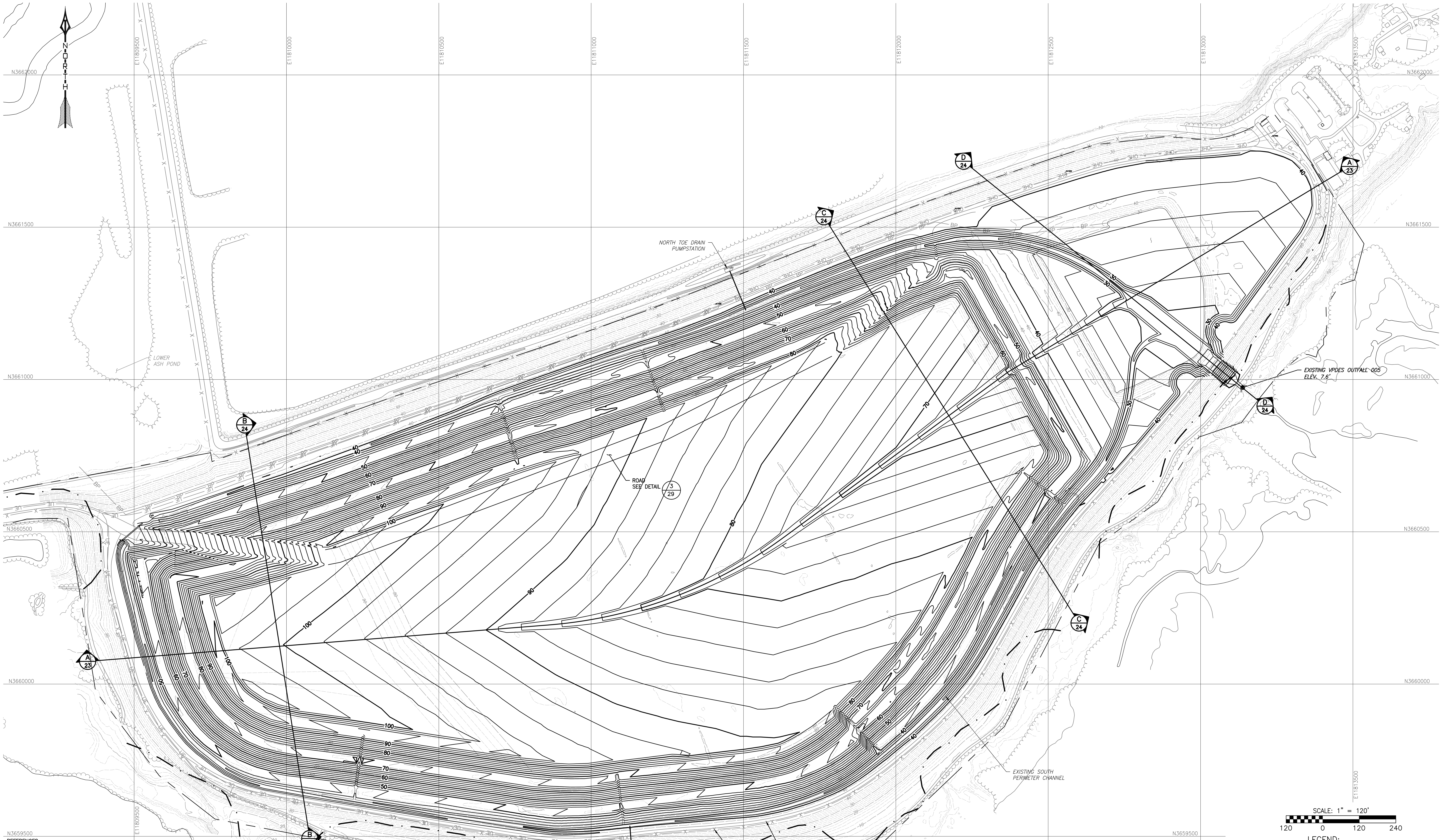
DRAWING TITLE

PLAN VIEW - EXISTING CONDITIONS

PROJECT		CLIENT
UPPER ASH POND CLOSURE PLAN CHESTERFIELD POWER STATION CHESTER CHESTERFIELD COUNTY, VIRGINIA	gai consultants	DOMINION GLEN ALLEN, VIRGINIA

ISSUING OFFICE: Murrysville | 4200 Triangle Lane, Export, PA 15632-1358

DRAWN BY:	CHECKED BY:	APPROVED BY:	GAI FILE NUMBER:
LEIDYJM	BORTZKM	PHILLJD	C150035-00-000-00-C-E1-002
DWG TYPE:	SCALE:	ISSUE DATE:	ALT./CLIENT DRAWING NUMBER:
	AS SHOWN	07/22/2015	
REVISION	SHEET NO.:		GAI DRAWING NUMBER:
A/0	2 OF 41		E1-002



SCALE: 1" = 120'

120 0 120 240

REFERENCES:

1. BASE MAPPING PROVIDED BY TUCK MAPPING SOLUTIONS, INC. NORTH AMERICAN DATUM 1983 (U.S. FEET). AIRBORNE SURVEY PHOTOGRAPHY FLIGHT FLOWN OCTOBER 8, 2014. COMPLETED APRIL 30, 2015.
2. PROPERTY BOUNDARY IS FROM D&M SURVEYORS, P.C., TAX ID NO. 811-660-3332-0000, 05/08/15.
3. APPROXIMATE RPA BOUNDARY IS FROM GOLDER ASSOCIATES, "WETLAND DELINEATION MAP", DRAWING 1520610F01, 04/03/15.

NOTES:

1. CONTOURS SHOWN ARE FINAL COVER CONTOURS SHOWING THE TOP OF THE PROTECTIVE SYSTEM.
2. UPON COMPLETION OF CLOSURE ACTIVITIES, RESTORE SURFACE AT ROAD AROUND ENTIRE PERIMETER. SEE DETAIL 3/29 AND SHEET 26.
3. ALL FENCING THAT HAS BEEN DAMAGED OR DISTURBED SHALL EITHER BE REPLACED OR REPAIRED.

LEGEND:

EXISTING CONTOUR	1850
PROPERTY LINE	
FENCING	X
TREES	
APPROXIMATE RPA BOUNDARY	
BURIED PIPE	BP BP BP

NO.:	DATE:	DWN:	CHK:	APV:	DESCRIPTION:

REVISION RECORD

This drawing was produced with computer aided drafting technology and is supported by electronic drawing files. Do not revise this drawing via manual drafting methods.

DRAWING TITLE

UPPER ASH POND FINAL CLOSURE CONFIGURATION

PROJECT	CLIENT
UPPER ASH POND CLOSURE PLAN CHESTERFIELD POWER STATION CHESTER CHESTERFIELD COUNTY, VIRGINIA	DOMINION GLEN ALLEN, VIRGINIA

gai consultants

ISSUING OFFICE: Murrysville | 4200 Triangle Lane, Export, PA 15632-1358

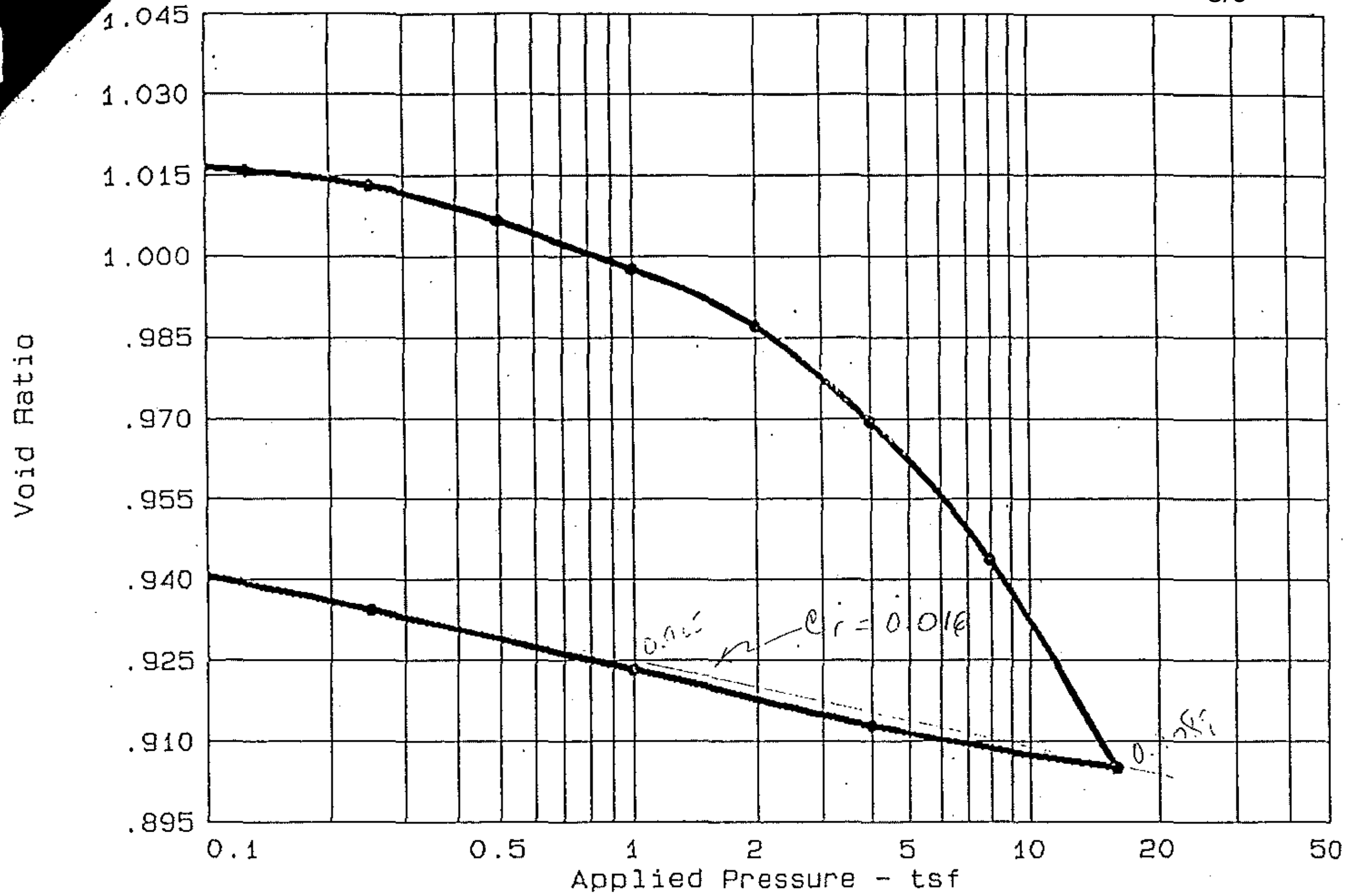
DRAWN BY:	CHECKED BY:	APPROVED BY:	GAI FILE NUMBER:
LEIDYJM	BORTZKM	PHILLJD	C150035-00-000-00-C-E1-030
DWG TYPE:	SCALE:	ISSUE DATE:	ALT./CLIENT DRAWING NUMBER:
	AS SHOWN	07/22/2015	
REVISION	SHEET NO.:		GAI DRAWING NUMBER:
A/0	22 OF 41		C150035-00-000-00-C-E1-030

GAI CAD FILE PATH: Z:\Energy\2015\C150035.00 – DOM–Chesterfld Pond Closu\CAD\Production Drawings\C150035–00–000–00–C–E1–030.dwg PLOTTED ON: 7/23/2015 4:25:32 PM PLOTTED BY: Jeremy Leidy PLOT FILE: GAI.stb

ATTACHMENT 2
LABORATORY RESULTS

CONSOLIDATION TEST REPORT

8/9



Coefficients of Consolidation (sq.in./min.)								
No.	Load	Cv	No.	Load	Cv	No.	Load	Cv
1	0.06	0.201	9	16.00	0.205			
2	0.13	0.138						
3	0.25	0.152						
4	0.50	0.159						
5	1.00	0.148						
6	2.00	0.199						
7	4.00	0.220						
8	8.00	0.201						

Natural Saturation	Natural Moisture	Dry Density	LL	PI	Sp.Gr.	C_c	e_0
74.9 %	33.8	69.8			2.26	0.13	1.0199

TEST RESULTS	MATERIAL DESCRIPTION
Compression Index = 0.13	POND ASH

Project No.: 97-071-01
Project: VIRGINIA POWER
Location: CF #1

Date: 3 MARCH 1997

CONSOLIDATION TEST REPORT

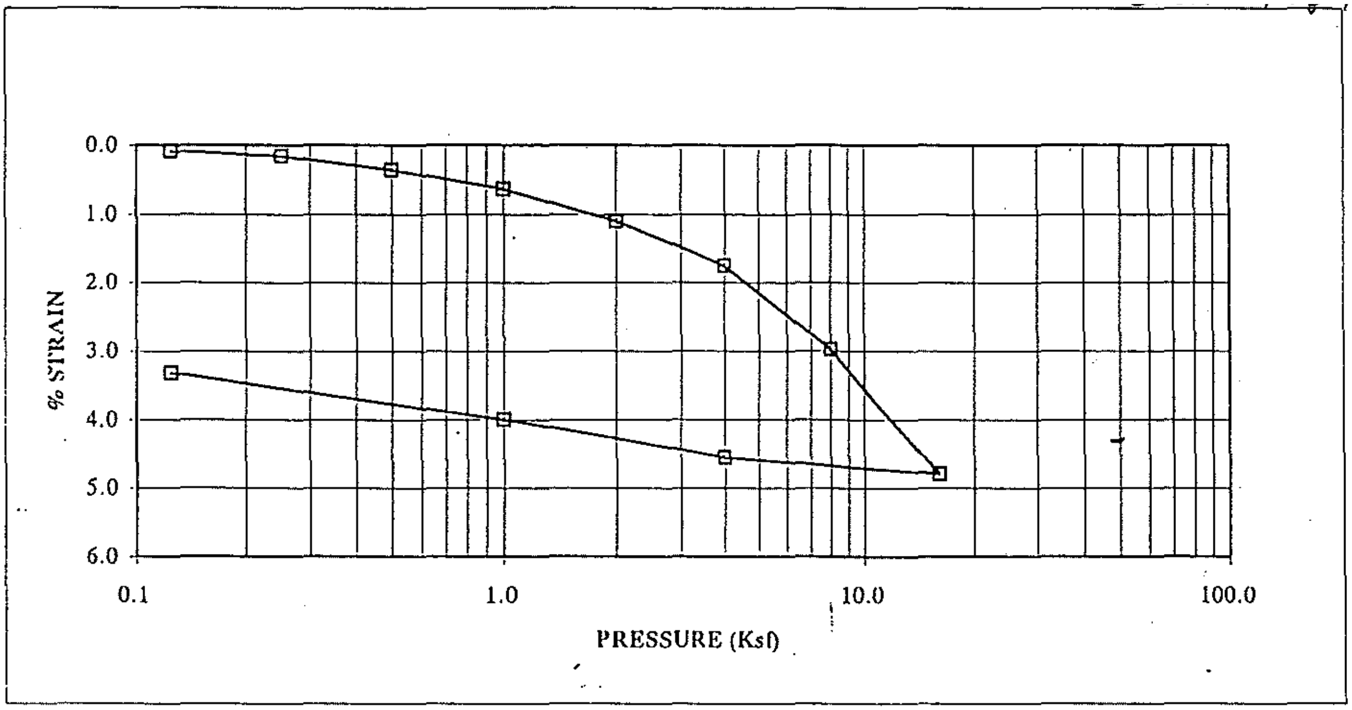
GAI Consultants, Inc.

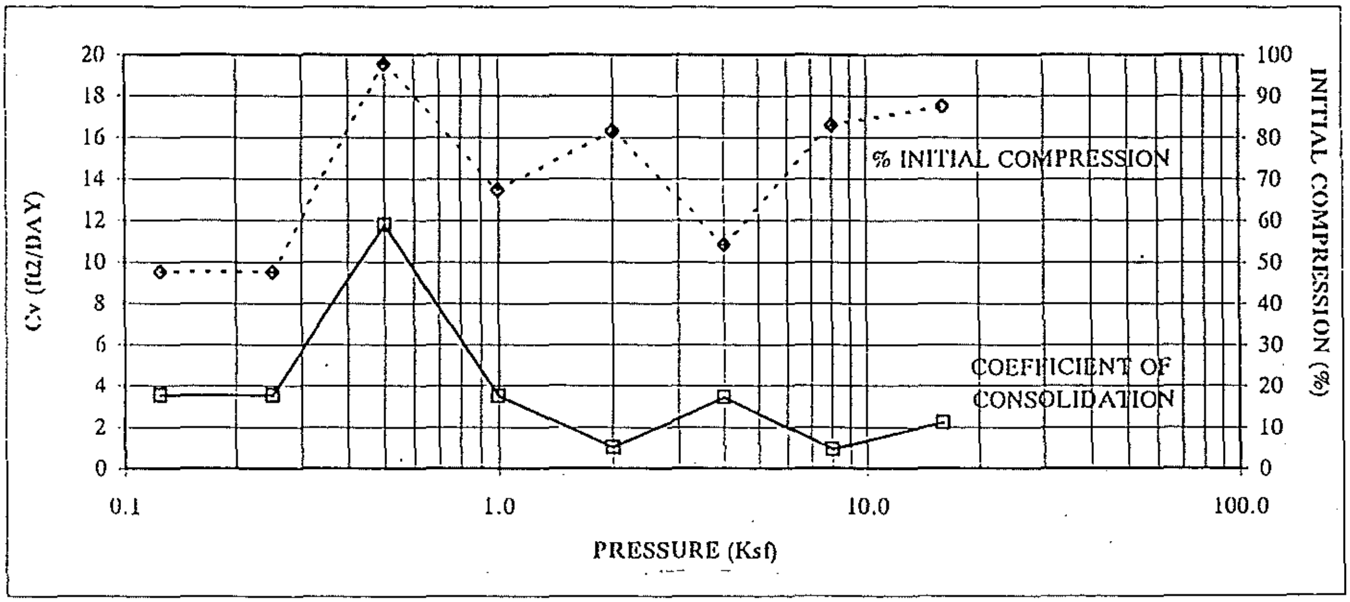
Remarks:

TESTED BY DDK
ENTERED BY DDK
CHECKED BY KM

Fig. No.______

ONE - DIMENSIONAL CONSOLIDATION
ASTM D 2435





SAMPLE ID	COAL ASH	LL	-		
SAMPLE TYPE	Bulk	PL	-		
SAMPLE DEPTH	-	PI	-		
		Gs	2.25		

	Initial	Final
Dry Unit Weight (pcf)	75.2	78.0
Wet Unit Weight (pcf)	94.7	106.6
Moisture Content	26.0%	36.6%
Void Ratio	0.8677	0.8000
Degree of Saturation	67.4%	103.0%

DESCRIPTION Coal Ash

USCS -

$$C_c = \frac{\Delta \varepsilon}{\Delta P} = \frac{0.048 - 0.03}{\log 16 - \log 8} = \frac{0.018}{0.30} = 0.06$$

TECH	PWM
DATE	8/5/97
CHECK	
REVIEW	

VIRGINIA / CHESTERFIELD COAL ASH TESTING / VA
977-8032

SUBJECT: DOMINION CHESTERFIELD POWER STATION – UPPER (EAST) POND CLOSURE - BEARING CAPACITY ANALYSIS

BY TIM DATE 7/17/2015 PROJ. NO. C1500035.00

CHKD. BY KLS DATE 9/25/2015 SHEET NO. 1 OF 7

gai consultants

OBJECTIVE:

This calculation was performed to evaluate the bearing capacity of the in-place CCR material within the Upper (East) Pond at the Chesterfield Power Station.

BACKGROUND:

Dominion is proposing the closure of Upper (East) Pond located at the Chesterfield Power Station located in Chesterfield County, Virginia. The Upper (East) Pond will include an approximate 113 acre geosynthetic cap area.

The proposed closure will include placing CCR material to modify the existing grades to facilitate surface water runoff and reduce ponding.

In addition to the soil and CCR fill material the geosynthetic and soil components of the proposed final cover system above the subgrade consist of the following layers (from top to bottom):

- 24 Inches of Cover Soil;
- Geocomposite Drainage Net (GDN) consisting of an HDPE geonet core with nonwoven, needle-punched geotextiles heat-bonded to its upper and lower surfaces;
- 40-mil LLDPE geomembrane;
- Subgrade (Existing ground, regraded CCR material, or a cushion (nonwoven) geotextile;

The existing or in-place CCR material will support the fill required to bring the existing surface to proposed subgrade and the final cover system.

METHODOLOGY:

The bearing resistance of the regraded in-place CCR material was evaluated using a Mathcad calculation developed from methods presented in AASHTO LRFD Bridge Design Specification (2010), Section 10.6.3.1. The nominal bearing resistance will be representative of the estimated shear strength of CCR material underlying the bearing surface.

REFERENCES:

1. AASHTO LRFD Bridge Design Specification (2010), Section 10.6.3.1.
2. Geotechnical Engineering Report, Upper Pond Stability Evaluation, Chesterfield Power Station by Schnabel Engineering. Dated August 15, 2014.

CALCULATION:

The bearing resistance of the in-place CCR material was evaluated using the following equation.

$$q_{ult} := c \cdot N_c \cdot s_c \cdot i_c + 0.5 \cdot \gamma_{eff} \cdot B_{eff} \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma + \gamma_{eff} \cdot D_f \cdot N_q \cdot s_q \cdot i_q$$

The soil parameters utilized in the analysis were selected to represent the in-place CCR material and were estimated from laboratory results of on-site samples obtained from historical field investigations monitored by Schnabel. Based upon two consolidated undrained triaxial tests performed on the CCR material in 2003 (Reference 2), the total or drained shear strengths were estimated to be Φ’ = 31.0 degrees and Φ’ = 30.0 degrees. For this analysis a lower than tested value was selected to account for uncertainties that may exist. The following material parameters were used as input into the Mathcad calculation.

In-place CCR Material
Moist Unit Weight, γ_m = 93-pcf
Saturated Unit Weight, γ_s = 98-pcf
Drained Shear Strength, Φ’ = 28 degrees

As shown in the bearing capacity equation above, foundation (or fill) geometry input including depth, width and length are required. For this analysis, the bearing resistance was estimated considering initial fill placement over the in-place CCR material using a dozer pushing material outward in lifts. The fill width and length was taken to be approximately 12-ft by 20-ft and the depth was taken as 0-ft as it will be at the ground surface. This scenario is considered to be a “worst-case” scenario where as the fill area increases and additional lifts are placed, the bearing resistance will increase.

A water surface or depth to water, Dw, was taken to be 1.5-ft to represent the approximate groundwater elevation based on the information in Schnabel’s 2014 report, in the southeast area of the Upper (East) Pond near the spillway approach channel and baffled chute spillway.

A detailed summary of all equations and inputs utilized in the analysis are shown in the Mathcad calculation included here as Attachment 1.

SUMMARY:

This calculation was completed to estimate the bearing resistance of the in-place CCR material that will support additional fill and final cover system as part of the proposed Chesterfield Upper (East) Pond closure. The ultimate bearing resistance has been estimated to be 3,300-psf representing the regraded CCR material during initial placement of the material. It is anticipated that during placement of the material in lifts, the bearing resistance near finished grade would be higher because of the increased distance to the water elevation. Applying a factor of safety of 3, the allowable bearing capacity is equal to approximately 1,090 psf, the proposed cover system will have 2’ of soil on top of the liner. With the estimated unit weight of the cover soil being 120 pounds per cubic foot (pcf), the stress on the liner is estimated to be 240 psf.

ATTACHMENT 1

MATHCAD BEARING RESISTANCE CALCULATION

Reduced Bearing Resistance Following AASHTO (2010)

1.0 Purpose:

The purpose of this calculation is to estimate the bearing resistance of supporting material under the fill at the strength limit state for the Dominion's Chesterfield Power Station. This calculation follows the theoretical method given in AASHTO LRFD Bridge Design Specification (2010), Section 10.6.3.1.

The factored bearing resistance (q_r) and the nominal bearing resistance (q_{ult}) are calculated per equations:

$$q_{ult} := c \cdot N_c \cdot s_c \cdot i_c + 0.5 \cdot \gamma_{eff} \cdot B_{eff} \cdot N_\gamma \cdot s_\gamma \cdot i_\gamma + \gamma_{eff} \cdot D_f \cdot N_q \cdot s_q \cdot i_q \qquad (10.6.3.1.2a\text{-}10P)$$

$$q_r := \varphi_b \cdot q_{ult} \qquad (A10.6.3.1.1\text{-}1)$$

2.0 Methodology:

The nominal bearing resistance, q_{ult}, is based on estimated soil parameters and should be representative of the soil shear strength under the considered loading and subsurface conditions. The bearing resistance should also be determined based on the highest anticipated position of groundwater level at the fill location or at $1.5B+D_f$, whichever is greater.

The bearing resistance of fill supported on granular soils should be evaluated for both permanent dead loading conditions and short-duration live loading conditions using effective stress methods of analysis and drained soil shear strength parameters.

Refer to section 5.0 for final estimated bearing capacity.

3.0 Input Values

Soil Inputs:

$\gamma_m := 93\text{pcf}$ — Moist unit weight of fill (pcf)

$\gamma_{sat} := 98\text{pcf}$ — Saturated unit weight of fill (pcf)

$\phi_f := 28\text{deg}$ — Internal Friction Angle of fill (degrees)

$c := 0\text{psi}$ — Cohesion (psi)

Footing Geometry Inputs:

$D_f := 0\text{ft}$ — Depth of the fill (ft)

$B := 12\text{ft}$ — Width of the foundation (ft)

$L := 20\text{ft}$ — Length of the foundation (ft)

Water Input:

$D_w := 1.5\text{ft}$ — Highest anticipated water table depth below the Foundation (ft)

Strength Limit State:

$\varphi_b := 0.45$ — Bearing Resistance factor specified in (Table 10.5.5.2.2-1)

4.0 Calculation

Effects of Water Table

Nominal bearing resistance shall be determined using the highest anticipated groundwater level at the footing location or 1.5B+Df, whichever is greater.The effect of ground water level on the ultimate bearing resistance shall be considered by using a weighted average soil unit weight.

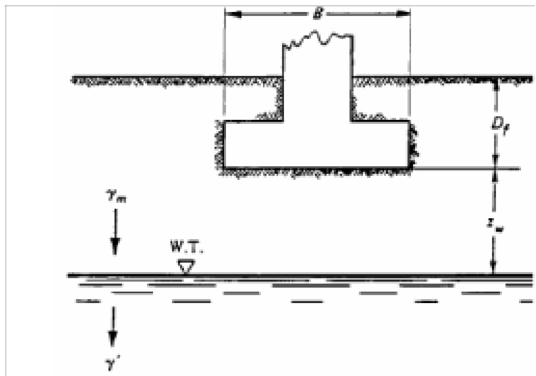


Figure 10.6.3.1.2g-1 - Definition Sketch for Influence of Groundwater Table on Bearing Capacity

$\gamma_{water} := 62.4pcf$

$z_w := D_w - D_f = 1.5 \cdot ft$

$$D := 0.5 \cdot B \cdot \tan\left(45deg + \frac{\phi_f}{2}\right) = 10 \cdot ft \qquad (10.6.3.1.2g\text{-}6)$$

$$\gamma_{eff} := \begin{cases} \gamma_m \text{ if } \phi_f < 37deg \wedge z_w \geq B & (10.6.3.1.2g\text{-}1) \\ \left[\left(\gamma_{sat} - \gamma_{water}\right) + \left[\left(\frac{z_w}{B}\right) \cdot \left[\gamma_m - \left(\gamma_{sat} - \gamma_{water}\right)\right]\right]\right] \text{ if } \phi_f < 37deg \wedge z_w < B & (10.6.3.1.2g\text{-}2) \\ \gamma_m \text{ if } \phi_f \geq 37deg \wedge z_w \geq D & (10.6.3.1.2g\text{-}4) \\ \left[\left(2 \cdot D - z_w\right) \cdot \frac{z_w \cdot \gamma_m}{D^2} + \frac{\gamma_{sat} - \gamma_{water}}{D^2} \cdot \left(D - z_w\right)^2\right] \text{ if } \phi_f \geq 37deg \wedge z_w < D & (10.6.3.1.2g\text{-}5) \\ \left(\gamma_{sat} - \gamma_{water}\right) \text{ if } z_w \leq 0 & (10.6.3.1.2g\text{-}7) \end{cases}$$

$\gamma_{eff} = 42.8 \cdot pcf$ Effective unit weight of soil (pcf)

Bearing Capacity Factors:

The bearing capacity factors relate to the drained angle of friction (ϕ'). The cN_c term is the contribution from soil shear strength, the γ*D_f*N_q term is the contribution from the surcharge pressure above the founding level, the 0.5*B*γ_{eff}*N_γ term is the contribution from the self weight of the soil.*

$$N_q := \tan\left(45deg + \frac{\phi_f}{2}\right)^2 e^{\pi \cdot \tan(\phi_f)} = 14.7$$

$$N_c := \begin{vmatrix} \left[(N_q - 1)\cdot\cot(\phi_f)\right] \text{ if } \phi_f > 0 \\ (2 + \pi) \text{ if } \phi_f = 0 \end{vmatrix} = 25.8$$

$$N_\gamma := 2\cdot(N_q + 1)\cdot\tan(\phi_f) = 16.7$$

Shape Factors Table 10.6.3.1.2a-3:

The shape factors are semi-empirical factors based on load tests of footings with various shapes.Equations shown in A Table 10.6.3.1.2a-3 should be used to calculate these factors.

$$s_q := \begin{vmatrix} 1 \text{ if } \phi_f = 0 \\ 1 + \left(\frac{B}{L}\cdot\tan(\phi_f)\right) \text{ if } \phi_f > 0 \end{vmatrix} = 1.3$$

$$s_c := \begin{vmatrix} 1 + \left(\frac{B}{5\cdot L}\right) \text{ if } \phi_f = 0 \\ \left[1 + \left(\frac{B}{L}\right)\cdot\left(\frac{N_q}{N_c}\right)\right] \text{ if } \phi_f > 0 \end{vmatrix} = 1.3$$

$$s_\gamma := \begin{vmatrix} 1 \text{ if } \phi_f = 0 \\ 1 - 0.4\cdot\left(\frac{B}{L}\right) \end{vmatrix} = 0.8$$

5.0 Results

Theoretical Solution for Bearing Capacity of Spread Footings:

The modified form of the general bearing capacity equation accounts for the effects of footing shape, ground surface slope and inclined loading as follows:

$$q_n := c\cdot N_c\cdot s_c + 0.5\cdot\gamma_{eff}\cdot B\cdot N_\gamma\cdot s_\gamma + \gamma_{eff}\cdot D_f\cdot N_q\cdot s_q \qquad (10.6.3.1.2a\text{-}1)$$

$$q_n = 3.3\cdot ksf$$

Bearing Capacity at the Strength Limit State:

$$q_r := \varphi_b\cdot q_n \qquad (10.6.3.1.1\text{-}1)$$

$$q_r = 1467\cdot psf \qquad ASD := \frac{q_n}{3} = 1087\cdot psf$$

APPENDIX H
Closure Cost Estimate Calculations

VIRGINIA DEPARTMENT OF ENVIRONMENTAL QUALITY

Solid Waste Disposal Facility Cost Estimate Form

Facility Name:	Chesterfield Power Station, Upper (East) Pond				Permit No. SWP	
Address:	451 Coxendale Road					
City:	Chester	State:	Virginia		Zip:	23836
FA Holder:	Virginia Electric and Power Company					
Estimate Prepared By:	Kevin M. Bortz, GAI Consultants, Inc.					

Indicate the plan versions for which this cost estimate was prepared, identifying the following information for each plan:

Closure Plan				**Post-Closure Care Plan**			
Title:	CCR Closure Plan			Title:	Post-Closure Care Plan, UEP CCR Closure		
Plan Date:	January 2016	Approved:		Plan Date:	January 2016	Approved:	
Consultant:	GAI Consultants, Inc.			Consultant:	GAI Consultants, Inc.		
Corrective Action Plan				**Corrective Action Monitoring Plan**			
Title:	N/A			Title:	N/A		
Plan Date:		Approved:		Plan Date:		Approved:	
Consultant:				Consultant:			

Cost Estimate Summary

Total Closure Cost:	$29,401,434
Total Post-Closure Cost:	$11,066,023
Total Corrective Action Cost:	$N/A
TOTAL:	**$40,467,457**

References

Please indicate references used to develop this cost estimate:
CCR Closure Drawings from the CCR Closure Plan
Unit Costs from Remedial Construction Services, L.P., Bid Estimate
RSMeans Site Work and Landscape Cost Data, 2014, 33rd Edition
Treatment Costs developed by Geosyntec Consultants, Inc

Certification by Preparer:

This is to certify that the cost estimates pertaining to the engineering features and monitoring requirements of this solid waste management facility have been prepared by me and are representative of the design specified in the facility's approved Closure, Post-Closure and Corrective Action Plans. The estimate is based on the cost of hiring a third party and does not incorporate any salvage value that may be realized by the sale of wastes, facility structures, or equipment, land or other facility assets at the time of partial or final closure. In my professional judgment, the cost estimates are a true, correct, and complete representation of the financial liabilities for closure, post-closure care, and corrective action of the facility and comply with the requirements of 9 VAC 20-70 and all other DEQ rules and statutes of the Commonwealth of Virginia.

Name:	Kevin M. Bortz	Signature:	Kevin M Bortz
Title:	Assistant Engineering Manager	Date:	1/7/2016

Acknowledgement by Owner/Operator :

Name:	DAVID A. CRAYMER	Signature:	David A Craymer
Title:	VP POWER GEN SYSTEM OPERATIONS	Date:	1/7/16

Worksheet CEW-01: FORMAT FOR THE ESTIMATION OF CLOSURE COSTS

FILL IN THE BOXES. THE REST WILL BE CALCULATED FOR YOU

Soil Cap Components

				Calculation or Conversion	
I.	**Slope & Fill**				
a.	Area to be capped	112	acres	x 4,840yd2/ac	542,080 yd2
b.	Depth of soil needed for slope and fill	20	inches avg.	x 1yd/36in	0.56 yd
c.	Quantity of soil needed			a x b	301,156 yd3
d.	Percentage of soil from off-site	0%			
e.	Purchace unit cost for off-site material	$0.00	/yd3		
f.	Percentage of soil from on-site			(1 - d)	100%
g.	Excavation unit cost (on-site material)	$2.74	/yd3		0
h.	Total soil unit cost			(d x e) + (f x g)	$2.74 /yd3
i.	Hauling, Placement and Spredding unit cost	$0.80	/yd3		0
j.	Compaction unit cost	$0.00	/yd3	Included in item I.i.	
k.	Total soil unit cost			h + i + j	$3.54 /yd3
l.	Soil subtotal			k x b	$1,066,091
m.	Percent compaction (shrinkage factor)	5%			
	Total Slope & Fill Cost			l x (1 + m)	***$1,119,395***
II.	**Infiltration Layer Soil**				
	Infiltration Soil Cost				
a.	Area to be capped	112	acres	x 4,840yd2/ac	542,080 yd2
b.	Depth of infiltration soil needed	18	inches	x 1yd/36in	0.50 yd
c.	Quantity of infiltration soil needed			a x b	271,040 yd3
d.	Percentage of soil from off-site	0%			
e.	Purchace unit cost for off-site material	$0.00	/yd3		
f.	Percentage of soil from on-site			(1 - d)	100%
g.	Excavation unit cost (on-site material)	$0.00	/yd3	Included in item II.i.	
h.	Total infiltration soil unit cost			(d x e) + (f x g)	$0.00 /yd3
i.	Excavate, Hauling, Placement and Spredding unit cost	$10.03	/yd3		
j.	Compaction unit cost	$1.48	/yd3		
k.	Total infiltration soil unit cost			h + i + j	$11.51 /yd3
l.	Infiltration soil subtotal			k x b	$3,119,670
m.	Percent compaction (shrinkage factor)	5%			
n.	*Subtotal Infiltration Soil Cost*	.		l x (1 + m)	*$3,275,654*
	Soil Admixture Cost				
o.	Area to be capped	112	acres	x 4,840yd2/ac	542,080 yd2
p.	Soil admixture unit cost	$0.00	/yd2		
q.	*Subtotal admixture cost*			a x b	*$0*
	Soil Testing				
r.	Area to be capped	112	acres		
s.	Testing unit cost	$0.00	/acre		
t.	*Subtotal soil testing cost*			a x b	*$0*
	Total Infiltration Soil Cost (soil, admixtures, and testing)			n + q + t	***$3,275,654***

III. Erosion Control / Protective Cover Soil

a.	Area to be capped	112	acres	x 4,840yd2/ac	542,080 yd2
b.	Depth of soil needed	0	inches	x 1yd/36in	0.00 yd
c.	Quantity of soil needed			a x b	0 yd3
d.	Percentage of soil from off-site	0%			
e.	Purchace unit cost for off-site material	$0.00	/yd3		
f.	Percentage of soil from on-site			(1 - d)	100%
g.	Excavation unit cost (on-site material)	$0.00	/yd3		
h.	Total erosion/protective soil unit cost			(d x e) + (f x g)	$0.00 /yd3
i.	Hauling, Placement and Spredding unit cost	$0.00	/yd3		
j.	Compaction unit cost	$0.00	/yd3		
k.	Total soil unit cost			h + i + j	$0.00 /yd3
l.	Erosion/Protective soil subtotal			k x b	$0
m.	Percent compaction	0%			
	Total Erosion Control/Protective Cover Soil Cost			l x (1 + m)	***$0***

IV. Vegetative support soil (Topsoil)

a.	Area to be capped	112	acres	x 4,840yd2/ac	542,080 yd2
b.	Depth of topsoil needed	6	inches	x 1yd/36in	0.17 yd
c.	Quantity of topsoil needed			a x b	90,347 yd3
d.	Percentage of topsoil from off-site	0%			
e.	Purchace unit cost for off-site material	$0.00	/yd3		
f.	Percentage of topsoil from on-site			(1 - d)	100%
g.	Excavation unit cost (on-site material)	$0.00	/yd3		
h.	Total topsoil unit cost			(d x e) + (f x g)	$0.00 /yd3
i.	Excavate, Hauling, Placement and Spredding unit cost	$5.16	/yd3	Includes reutilization of stripped soil	
j.	Total soil unit cost			h + i	$5.16 /yd3
	Total Topsoil Cost			c x j	***$466,189***

V. Vegetative Cover

a.	Area to be vegetated	112	acres		
b.	Vegetative cover (seeding) unit cost	$3,323	/acre		
c.	Erosion control matting unit cost	$6,098	/acre		
	Total Vegetative Cover Cost			a x (b + c)	***$1,055,152***

Soil Cap Component Subtotal (I + II + III + IV + V): $5,916,390

Geosynthetic Barrier & Infiltration Layers

VI. Flexible Membrane Liner

				Calculation or Conversion	
a.	Quantity of FML needed	112	acres	x 43,560ft2/ac	4,878,720 ft2
b.	Purchase unit cost	$0.31	/ft2		
c.	Installation unit cost	$0.15	/ft2		
d.	Total FML unit cost			b + c	$0.46
	Total FML cost			a x d	***$2,244,211***

VII. Geosynthetic Clay Liner

a.	Quantity of GCL needed	0	acres	x 43,560ft2/ac	0 ft2
b.	Purchase unit cost	$0.00	/ft2		
c.	Installation unit cost	$0.00	/ft2		
d.	Total GCL unit cost			b + c	$0.00 /ft2
	Total GCL Cost			a x d	***$0***

Geosynthetic Layers Subtotal (VI + VII): $2,244,211

Drainage Components

		Input	Unit	Calculation or Conversion	Result	Unit
VIII.	**Sand or Gravel Drainage**					
a.	Area to be capped	112	acres	x 4,840yd2/ac	542,080	yd2
b.	Depth of sand or gravel needed	0	inches	x 1yd/36in	0.00	yd
c.	Quantity of drainage material needed			a x b	0	yd3
d.	Percentage of media from off-site					
e.	Purchace unit cost for off-site material		/yd3			
f.	Percentage of material from on-site			(1 - d)	100%	
g.	Excavation unit cost (on-site material)		/yd3			
h.	Total drainage material unit cost			(d x e) + (f x g)	$0.00	/yd3
i.	Hauling, Placement and Spredding unit cost		/yd3			
j.	Compaction unit cost		/yd3			
k.	Total drainage material unit cost			h + i + j	$0.00	/yd3
l.	Drainage material subtotal			k x b	$0.00	
m.	Percent compaction					
	Total drainage material cost			l x (1 + m)	***$0***	
IX.	**Geotextile**					
a.	Quantity of geotextile needed	112	acres	x 43,560ft2/ac	4,878,720	ft2
b.	Purchase unit cost	$0.15	/ft2			
c.	Installation unit cost	$0.00	/ft2			
d.	Total geotextile unit cost			b + c	$0.15	/ft2
	Total Geotextile Cost			a x d	***$731,808***	
X.	**Geonet Composite**					
a.	Quantity of geonet composite needed	112	acres	x 43,560ft2/ac	4,878,720	ft2
b.	Purchase unit cost	$0.50	/ft2			
c.	Installation unit cost	$0.11	/ft2			
d.	Total geonet composite unit cost			b + c	$0.61	/ft2
	Total Geonet Composite Cost			a x d	***$2,976,019***	
XI.	***Cap Drains***					
a.	Length of ***cap drains*** needed	30,545	LF			
b.	Purchase unit cost	$14.43	/LF			
c.	Trenching and backfilling cost	$0.00	/LF			
d.	Total ***cap drain*** unit cost			b + c	$14.43	/ft2
	Total Cap Drain Cost			a x d	***$440,764***	

XII. Drainage Channels (Stormwater Control)

Drainage benches and berms

a.	Size of drainage bench needed	0	LF		
b.	Drainage bench unit cost		/LF		
c.	*Subtotal drainage bench cost*			a x b	*$0*
d.	Size of drainage swale/berm needed	14,246	LF		
e.	Drainage swale/berm unit cost	$110	/LF		
f.	*Subtotal drainage swale/berm cost*	0		d x e	*$1,568,770*

Emergency Spillway System

g.	Installed Cost, Emergency Spillway	$797,723			
h.	Installed Cost, Replacement Bridge on Road	$27,048			
i.	*Total Spillway System cost*			g + h	*$824,771*

Drainage and Erosion Control during Construction

j.	Pumps and Water Control	$250,000			
k.	Cleaning Channels after Construction	$75,000			
l.	Misc. E&S items, E&S maintenance	$122,680		j+k+l	*$447,680*
	Total Stormwater Control			c + f + i + l	***$2,841,221***

Drainage Component Subtotal (VIII + IX + X + XI+ XII): $6,989,812

Landfill Gas and Groundwater Features

XIII. Landfill Gas Monitoring & Control Components

Landfill Perimeter System

				Calculation	
a.	Number of probes to be installed	0	probes		
b.	LFG probe unit cost		/probe		
c.	*Subtotal LFG probe cost*			a x b	*$0*

Landfill Control Systems

d.	Area to be closed	112	acres		
e.	Average number of vents per acre	0	vents / acre		
f.	LFG vent unit cost		/vent		
g.	*Subtotal LFG vent cost*			d x e x f	*$0*
h.	Length of header pipe needed		LF		
i.	Header pipe unit cost		/LF		
j.	Header pipe installation cost		/LF		
k.	*Subtotal LFG active vent hook-up*			h x (i + j)	*$0*
	Total Landfill Gas Management Cost			c + g + k	***$0***

XIV. Groundwater Monitoring *and Toe Drain* Components

a.	Hydrogeologic study cost				
b.	Number of wells to be installed	17	wells		
c.	GW Monitoring Well unit cost	$11,500	/well		
d.	Number of wells > 50 ft length	0	wells		
e.	Additional well length over 50 ft	0	LF/well		
f.	Unit cost for additional well length	$0	/LF		
	Total Groundwater Monitoring Well Cost			a + (b x c) + (d x e x f)	***$195,500***
g.	*Toe Drain Pumps to be Installed*	2			
h.	*Pump Unit Cost*	$19,668			
i.	*Pipe Length to be installed*	4,219			
j.	*Pipe Unit Cost*	$10			
k.	*Valves to be installed*	5			
l.	*Valve unit cost*	$10,142			
m.	*Electrical Cost*	$512,551			
	Total Toe Drain Pumping and Piping Cost			(g x h) + (i x j) + (k x l) + m	***$645,419.85***

Landfill Gas & Groundwater Features Subtotal (XIII + XIV): $840,920

Miscellaneous

XV.	**Site Preparation (ncludes Demolition/Stripping of Vege**			Calculation	
a.	Quantity of *stripped vegetation*	78,650	yd3		
b.	Stripping, Loading and Hauling unit cost	$17.45	/yd3		
d.	*Additional Demolition, Lump Sum*	$63,402.00			
e.	*Decanting, Dewatering, and Stabilization*	$897,600.00			
e.	***Total Site Preparation Cost***			*(a x b) + c + d + e*	***$2,333,445***
XVI.	**Erosion/Sediment Control**				
a.	Quantity of silt fence*/filter sock/straw bales*	25,950	LF		
b.	Silt Fence unit cost	$6.07	/LF		
	Total Silt Fence Cost			a x b	***$157,517***
XVII.	***Roads***				
a.	Size of LF access road	13,800	yd2		
b.	Depth of gravel needed	12	inches	x 1yd/36in	0.3 yd
c.	Depth of asphalt needed	0	inches	x 1yd/36in	0.0 yd
d.	Total material needed			a x (b + c)	4,600 yd3
e.	Road material unit cost	$40.22	/yd3		
f.	Placement/Spreading unit cost	$0.00	/yd3		
g.	Geotextile unit cost	$0.54	/yd2		
h.	Perimeter road restoration, gravel needed	5110	Tons		
i.	Unit cost of gravel	$20.48	/ton		
j.	Overflow parking area paver size	87,120	ft2		
k.	Overflow parking area paver unit cost	$3.14	/ft2		
l.	Geotextile unit cost	$0.12	/ft2		
h.	Parking area, gravel needed	3700	Tons		
i.	Unit cost of gravel	$20.48	/ton		
	Total access road cost			d x (e + f + g) + (h x i) + j x (k + l) + (h x i)	***$651,936***
XVIII.	**Site Security**				
Fencing					
a.	Length of fencing needed	690	ft		
b.	Fence unit cost	$54.37	/ft		
c.	*Subtotal fencing cost*			a x b	*$37,515*
Gate or Barrier					
d.	Number of gates required			Gates are including in the fencing costs	
e.	Gate unit cost		/gate		
f.	*Subtotal gate cost*			d x e	*$0*
Closed Sign					
g.	Number of signs required	2		c + f + i	
h.	Sign unit cost	$250.00	/gate		
i.	*Subtotal sign cost*			g x h	*$500*
	Total site security cost			c + f + i	***$38,015***
XIX.	**Mobilization / Demobilization / General Conditions**				
a.	Cost for mobilization/demobilization/gen. conds.	$6,596,556			
	Total mobilization/demobilization cost				***$6,596,556***

Miscellaneous Subtotal (XV + ... + XIX): $9,777,468

Closure Cost Subtotal (CCS):	(I + ... + XIX)	$25,768,801
Contingency (10%):	CCS x 0.10	$2,576,880
Engineering & Documentation:		
Construction QA/QC (1%)	CCS x 0.01	$257,688
Closure Certification and CQA Report (1%)	CCS x 0.01	$257,688
Survey and as-builts (2%)	CCS x 0.02	$515,376
Cost for survey and deed notation		$25,000
Total Engineering & Documentation Costs		***$1,055,752***
Total Closure Cost:	CCS + Contingency + Engineering	**$29,401,434**

APPENDIX I
Geotechnical References

This appendix contains selected portions of the following reports prepared by Schnabel Engineering, Inc.:

- Upper Pond Stability Evaluation, 2014 (pages 1-19)
- Geotechnical Engineering and Groundwater Hydrology Study, 1982 (pages 20-52)
- Geotechnical Engineering Study Long Term Ash Storage Pond Dike, 1996 (pages 53-119)

Portions of the reports that are applicable to the closure of the Upper (East) Pond have been included.

Upper Pond Stability Evaluation, 2014

GEOTECHNICAL ENGINEERING REPORT

Upper Pond Stability Evaluation
Chesterfield Power Station
Coxendale Road
Chesterfield County, Virginia

Schnabel Reference 14213000
August 15, 2014

Prepared For:

Dominion

Schnabel
ENGINEERING

Schnabel
ENGINEERING

August 15, 2014

Mr. Chris Gee, PE
Dominion Resources Services, Inc.
5000 Dominion Boulevard
Glen Allen, VA 23060

Subject: **Project 14213000, Geotechnical Engineering Report, Upper Ash Pond Stability Evaluation, Chesterfield Power Station, Coxendale Road, Chesterfield County, Virginia**

Dear Mr. Gee:

SCHNABEL ENGINEERING CONSULTANTS, INC. (Schnabel) is pleased to submit our geotechnical engineering report summarizing our stability evaluation. This document includes tables, figures and appendices with relevant data utilized for this study. This study was performed in accordance with our revised proposal dated June 26, 2014. This work was authorized by Purchase Order No. 70273893, and the email from Dominion on July 11, 2014. This report presents the results of our geotechnical engineering analysis for the slope on the south side of the lower ash pond at Chesterfield Power Station in Chesterfield, Virginia.

SCOPE OF SERVICES

Our original scope was presented in our proposal dated April 9, 2014 and included drilling, laboratory testing and evaluation of the slopes on the east and south sides of the lower ash pond. At Dominion's request, we revised our scope to include evaluation of only the south slope.

Additionally, our work would be divided into two phases. Our evaluation during the first phase would be based on subsurface data and laboratory test results from our previous work at the upper and lower ash ponds. If the evaluation performed during the first phase indicates the south slope exhibits marginal stability, then we will perform a second phase including additional subsurface exploration, laboratory testing and evaluation. This modified scope was included in our revised proposal dated June 26, 2014.

The scope of services for the first phase includes the following:

- Initial evaluation including the following:
  - Reviewing density and classification information obtained during our earthwork observation and testing performed over the past 12 years as the ash was placed as fill and compacted. A total of 58 ash fill compaction summary reports have been issued over this time.
  - Reviewing assumed and measured material properties used for design of this facility.

- o Perform slope stability analysis of proposed and existing conditions for one cross section through the south slope.
- o This report summarizing our analyses and results.

SITE DESCRIPTION

Site Description

The site is located at the upper ash pond, south of Chesterfield Power Station. A vicinity map is included as Figure 1. As requested, our evaluation focused on the slope on the south side of the ash pond. The ash pond and the section we evaluated are shown on the Location Plan included as Figure 2.

The original construction included an earth berm around the ash pond to contain the wet ash as it was sluiced into the containment area. The berm was constructed using soil fill. The top of the berm is at about El 42. After the berm was completed, ash was sluiced into the containment area. Around 2002, a stormwater basin was constructed within the sluiced ash at the east end of the upper ash pond so that stormwater could be contained within the upper ash pond. A splitter dike was then built with compacted ash just west of the basin so that ash could continue to be sluiced into the upper ash pond.

Once the sluiced material was placed to within a few feet from the top of the berm, it was allowed to drain and gain enough strength to support additional ash. After the sluiced material was strong enough, ash material that had been partially dried at the lower ash pond was transported by truck to the upper ash pond and placed as fill above the sluiced ash.

Ash placement is currently ongoing. Compacted ash has been placed up to about El 80, and the proposed final condition will include ash placed up to about El 130, as shown on Figure 3. The existing slope grades are slightly different than the grades originally proposed in the closure plan. The originally proposed grades and the existing grades are both shown on Figure 3. The toe of the existing compacted ash slope is about 10 ft closer to the crest of the containment berm than the toe of the compacted ash slope included in the closure plan.

The ash material placed above the level of the top of the containment berm was moisture conditioned and compacted to meet the project density requirements. Ash material within 50 ft of the face of the slope is dried to within about +4 percent to -6 percent of optimum moisture content and compacted to at least 95 percent of maximum dry density according to ASTM D698. Ash material more than 50 ft from the face of the slope can be up to 8 percent wet of its optimum moisture content and compacted to at least 92 percent of its maximum dry density.

The ash above the berm has been filled about 10 ft, laterally, past the planned stockpile limits. The face of the slope is graded at the planned angles, but the toe of the ash slope is about 10 ft closer to the paved drainage ditch along the crest of the berm than originally planned. Plans provided to us show the top of the ash on the south side of the pond is at about El 78 to El 80, and a bench about 25 ft wide at about El 57. The existing ash slope is graded at about 3H:1V. Including the benches, the slope has an average slope of about 4H:1V.

Based on our review of classification and density data, the ash consists of sandy silt, silty sand and clayey sand ASH FILL. The underlying natural soils consist of recent alluvial and terrace sands, silts and clays and Cretaceous age sands and gravels.

We obtained the site information from the isopach plan by Golder Associates titled “Remaining Capacity as of 12/11/13 Areal Survey” dated February 14, 2014, the electronic topographic plan provided to us by your office, information in our files and communication with Dominion personnel.

PREVIOUS SUBSURFACE EXPLORATION AND LABORATORY TESTING

We have conducted subsurface exploration and field testing programs on several occasions over the past 30+ years. The locations of the borings drilled in the area of the upper ash pond are shown on Figure 2. Logs for these borings are included in Appendix A. The results of laboratory testing performed as part of our previous work are included in Appendix B.

Subsurface Exploration Methods

Test Borings

The test borings were performed under our observation between 1982 and 2005. The Standard Penetration Test (SPT) was conducted at selected depths in the borings. Appendix A includes remarks, and logs for the borings; classification criteria; drilling methods; and sampling protocols. Figure 2 (included at the end of this report) indicates the approximate test boring locations.

Many correlations with SPT N values are used in the development of our geotechnical engineering recommendations. These correlations are usually based on SPT N values obtained using a Safety Hammer. Some of the SPTs for this project were performed using an Automatic Trip Hammer (ATH), and some were performed using a standard Safety Hammer. The energy applied to the split-spoon sampler using the ATH is about 33 percent greater than that applied using the Safety Hammer. The hammer blows shown on the boring logs are uncorrected for the higher energy. However, where appropriate, we corrected SPT N values for the higher energy when using N values with correlations in our analyses.

Previous Soil Laboratory Testing

Our laboratory performed tests on selected samples collected during the subsurface exploration. The testing aided in the classification of materials encountered in the subsurface exploration and provided data that we used to develop our recommendations over the years. The results of the laboratory tests performed on samples collected during drilling in the upper ash pond over the past three decades are included in Appendix B. Selected test results are also shown on the boring logs in Appendix A.

EARTHWORK OBSERVATION AND TESTING FOR THE UPPPER ASH POND

We have reviewed our “Earthwork Observation and Testing, Upper (East) Pond, Phase 1” Reports No. 1 through No. 58 (Schnabel Reference No. 02131106301) and other correspondence and data in our files. Based on this review, we believe the existing ash tested has been placed in general accordance with the project requirements for compaction and moisture content.

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Based on the borings and laboratory testing performed over the past 30+ years and our work with the ongoing ash placement operations, we characterized the following generalized subsurface stratigraphy:

Containment Berm Fill

The containment berm consists of soil fill. Generally, the fill materials within the containment berm include lean clay, clayey sand, silty sand and poorly graded sand. Borings B-8 and B-9 drilled in 1982 and Borings B-501 through B-505 drilled in 2004 were drilled in the area of the south ash pond slope. Logs for the borings drilled in this area indicate that the berm consists of clayey sand, silty sand and poorly graded sand with silt FILL.

The laboratory test results in our files for samples of soils to be used as embankment fill are summarized in the table below.

Table 1: Embankment Fill Laboratory Test Results and Design Values

	Classification	γ (pcf)	ϕ' (degrees)	C' (psi)
1982 CU Triaxial Test	SM	132.0	36	0
1982 CU Triaxial Test	SC	129.4	34	0
1983 CU Triaxial Test	CL	129.3	28	1.0
1983 Design Values	-	130.0	32	0
2014 Design Values	-	*130.0*	*32*	*0*

While the 1983 design values consider the test results for the sands and the clays, these values are likely conservative for our current evaluation, considering only sandy materials were encountered in the borings drilled in the area of the south slope. However, due to the potential for variability in fill materials, we conservatively used the 1983 values in our current evaluation.

Ash Fill

The ash that was sluiced into the pond and consolidated in place is below the level of the top of the soil containment berm. Topographic maps in our files indicate the top of the sluiced ash was at about El 35 including about 6 inches of soil cover. Ash that has been trucked in from the lower ash pond has been placed as compacted fill above the sluiced ash since about 2002. We sampled and tested these materials in borings drilled from 1999 to 2005. Average SPT N values are provided below.

Table 2: Average SPT N Values from Borings Drilled in Ash

Year	Ash Materials Sampled	Average SPT N Values (blows/foot)
1999	Compacted Ash in Splitter Dike	10
	Sluiced Ash	2
2003	Compacted Ash Fill	8
2005	Compacted Ash Fill	9

The compacted ash was generally loose to firm with average SPT N values of about 8 to 10 bpf. The sluiced as was very loose with an average SPT N value of 2 bpf. The ash generally classified as silt, silty sand and clayey sand FILL. The ash tested in our laboratory generally classified as silt (ML).

We have not performed strength testing on any undisturbed sluiced or compacted ash samples, but in 2002 and 2003 we performed strength testing on reconstituted ash samples. We tested ash samples compacted with varying compaction effort to evaluate the relationship between density and shear strength. We recommended that the ash within 50 ft of the slope face be compacted to 95% of its maximum dry density (MDD) according to ASTM D698, Standard Proctor, and the ash more than 50 ft from the face of the slope be compacted to at least 92% of MDD according to the same standard. Since then, spot field density testing has confirmed that the ash was placed in general accordance with these requirements.

Previous laboratory test results and the design values considered in our current evaluation are presented in the table below.

Table 3: Ash Fill Laboratory Test Results and Design Values

	Classification	**Moist Unit Weight (pcf)**	**Percent Compaction ASTM D698**	**ϕ' (degrees)**	**C' (psf)**
2002 Direct Shear Test (Sample #3)	ML	103	88	32	0
2002 Direct Shear Test (Sample #3)	ML	105	92	32	0
2002 Direct Shear Test (Sample #4)	ML	91	92	39	1.5
2003 CU Triaxial Test	ML (visual)	94	85 – 90 (est)	31	0
2003 CU Triaxial Test	ML (visual)	93	85 – 90 (est)	30	0
Compacted Ash 2014 Design Values	ML	100	92 - 95	31	0
Compacted Ash 2014 Design Values	ML	105	>95	32	0
Sluiced Ash 2014 Design Values	ML	95	<88 (est)	24	0

The strength testing summarized in the table above was performed on materials sampled from various stockpiles of ash, before it was placed and compacted in the pond. For our stability evaluation, we considered the results to represent the strength of the compacted ash.

The strength testing performed on reconstituted ash samples at less than 92% relative compaction generally resulted in friction angle values of about 31° to 32°. For our stability evaluation, we conservatively disregarded the sample with a friction angle of 39°. We conservatively considered a friction angle for the compacted ash at the low end of the range.

The sluiced ash material has been consolidating for decades under its own weight and for the past 10+ years under the weight of the compacted ash placed above the sluiced ash. For this evaluation, we considered a lower friction angle for the sluiced ash to reflect a lower estimated density based on the SPT N values in Table 2.

Alluvial and Terrace Soils

Alluvial and terrace soils associated with the James River were encountered below the ash and the containment berm. Generally, the alluvial materials consist of a mixture of sand, silt and clay. However, borings drilled in the area of the south slope indicate the alluvial materials in this area of the site consist of sands and gravels classifying as CLAYEY SAND (SC), SILTY SAND (SM), POORLY GRADED SAND WITH SILT (SP-SM), POORLY GRADED SAND (SP) and SILTY GRAVEL (GM). Recommended design values considered in our current evaluation are presented in the table below.

Table 4: Alluvium and Terrace Design Values

	Classification	γ (pcf)	ϕ' (degrees)	C' (psf)
2014 Design Values	Course-grained	120	30	0

Fine grained alluvial and terrace soils were encountered in several of the previous borings, but not in the area of the current stability evaluation.

Cretaceous Age Soils

The Cretaceous age soils of the Patuxent Formation were encountered below the alluvial and terrace deposits to the maximum depth of exploration in most of the deep borings. SPT N-values indicate these soils are generally dense to very dense sands and gravels classifying as CLAYEY SAND (SC), SILTY SAND (SM), CLAYEY GRAVEL (GC), and SILTY GRAVEL (GM). Previous design values and the design values considered in our current evaluation are presented in the table below.

Table 5: Cretaceous Sediments Design Values

	Classification	γ (pcf)	ϕ' (degrees)	C' (psi)
1983 Design Values	Coarse-grained	140	40	0
2014 Design Values	*Coarse-grained*	*140*	*40*	*0*

Generally, both fine-grained and coarse-grained soils were encountered on this site. However, the cretaceous age materials in the area of the south slope are coarse-grained. Accordingly, we only considered a unit weight, friction angle and cohesion that reflect the coarse-grained soils encountered. We believe the values used in 1983 are still suitable.

Groundwater

We considered the groundwater level to be at about the level of the top of the sluiced ash fill, approximate El 35. Over the life of the embankment, water has been observed seeping from the toe of the containment embankment. However, we are unaware of any current seepage issues. Accordingly, we considered the groundwater level is near the toe of the embankment, but not seeping from the toe. Design ground water levels were obtained from Borings B-501 through B-505 that were drilled in 2004.

Peak Ground Acceleration for Seismic Evaluation

We considered the peak ground acceleration (PGA) for this site according to the interactive seismic hazard app based on the International Building Code (IBC) Section 1615 (2008 and 2012) available on the United States Geological Survey website (http://earthquake.usgs.gov/hazards/apps/map/). We adjusted the peak ground acceleration (PGA) based on the site class (D) and the structure in the evaluation (earthen embankment). We considered a peak ground acceleration of 0.0725g for our seismic stability evaluation.

SLOPE STABILITY EVALUATION

We based our geotechnical engineering analysis on the information developed from our previous subsurface exploration and soil laboratory testing, along with the topographic plans furnished to our office. We evaluated the existing condition of the south slope with the ash placed up to about El 80 and the proposed final condition, with ash placed up to about El 130. We considered that the final slope above El 80 will extend up from the current slope face, instead of considering that it will be stepped back 10 ft to match the final slope face included in the closure plan. The slope geometry we considered in our evaluation is shown in Figure 3.

We analyzed the global stability of the existing slope using the software program Slope/W 2007 for both the current condition with the top of the ash fill at about El 80 and the final condition with the top of the ash fill at about El 130. We considered the soil properties discussed above for both conditions. The most probable failure surfaces for the existing and final conditions were the same. The results of our evaluation for the final condition are summarized in the table below. We included the printout of our analysis of the final condition in Appendix C.

Table 6: Computed Factors of Safety

Analysis Condition	Factor of Safety Existing and Final Conditions
Effective Stress	1.9
Effective Stress - Seismic	1.5

The results of our evaluation indicate the slope is stable in its current condition. The factors of safety for the effective stress conditions exceed the values generally recommended for new embankments of 1.5 for the condition without seismic forces and 1.1 for the condition where seismic forces are included.

Confidence in shear strength parameter selection and water level assumptions can influence the factor of safety. We are not aware of any observed movement of the slope on the south side of the upper ash pond, and we believe the shear strength parameters and the water levels considered in our evaluation are appropriately conservative. Accordingly, we believe the factors of safety in our analysis indicate adequate slope stability.

RECOMMENDATIONS FOR ADDITIONAL STUDIES

The results of our evaluation indicate the slope is stable in its current condition. However, we have based this analysis on existing data that do not include recent ground water levels, shear strength of the sluiced ash, or data needed for a liquefaction analysis. We recommend implementing the second phase of our proposal in order to obtain parameters to confirm our slope stability analyses and to perform a liquefaction analysis.

LIMITATIONS

This report has been prepared to aid in the evaluation of the slope on the south side of the upper ash pond and is intended for use concerning this specific area of the site. We based the analyses and recommendations submitted in this report on topographic information provided to us and on subsurface and laboratory test data already in our files. Any changes to the future grades from what is included in our evaluation should be brought to our attention so we can review our recommendations as needed. We attempted to provide for normal contingencies, but the possibility remains that differing conditions may be present.

We have endeavored to complete the services identified herein in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions as this project. No other representation, express or implied, is included or intended, and no warranty or guarantee is included or intended in this report, or other instrument of service.

We appreciate the opportunity to be of service for this project. Please call us if you have any questions regarding this report.

Sincerely,
SCHNABEL ENGINEERING CONSULTANTS, INC.

Evan B. Morris, PE
Associate

COMMONWEALTH OF VIRGINIA
EVAN B. MORRIS
Lic. No. 037983
PROFESSIONAL ENGINEER

Edward G. Drahos

Edward G. Drahos, PE
Principal

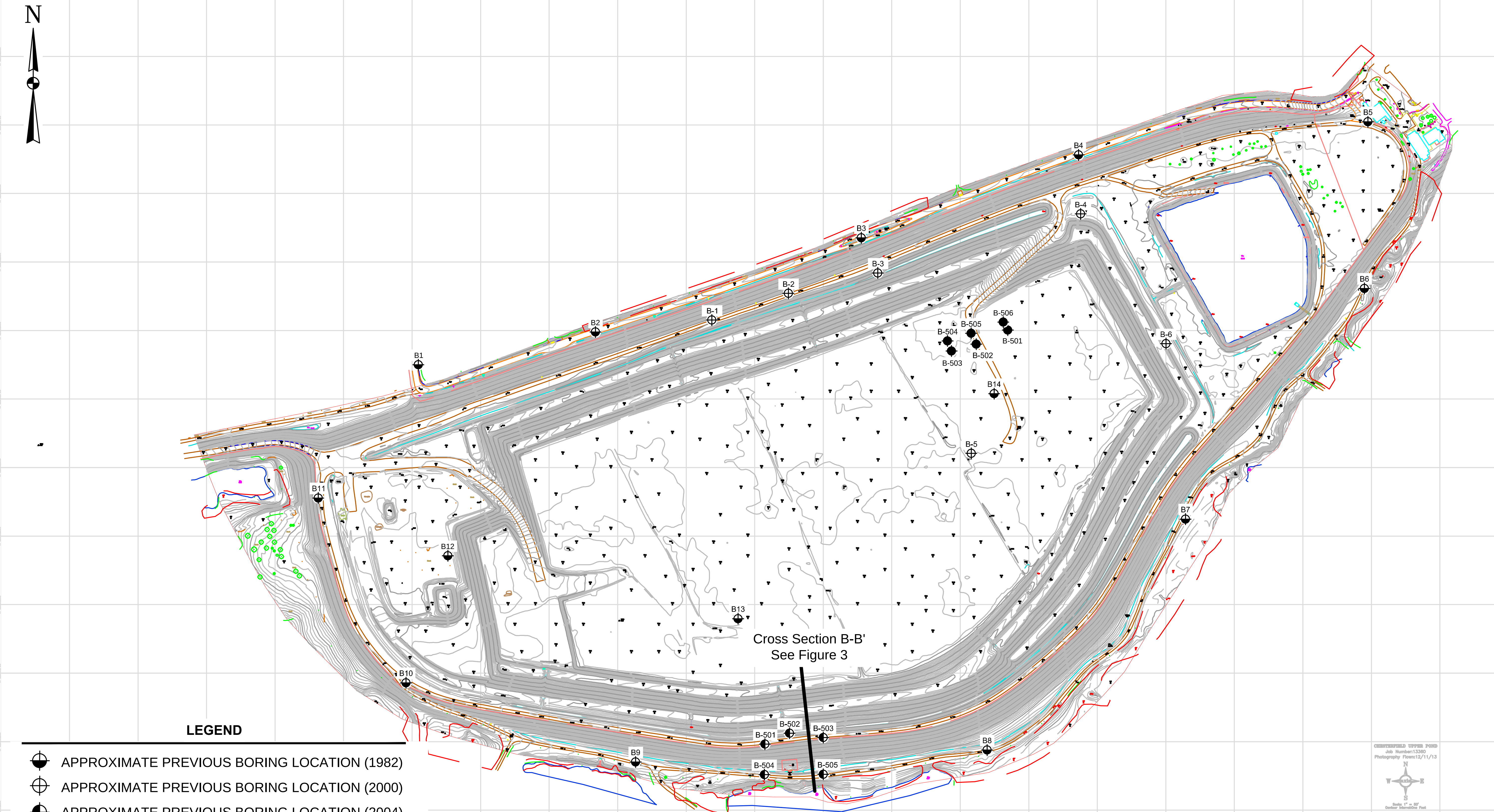
COMMONWEALTH OF VIRGINIA
EDWARD G. DRAHOS
Lic. No. 015605
PROFESSIONAL ENGINEER

EBM:EGD:ms

Figures: 1. Site Vicinity Map
2. Location Plan
3. South Slope Cross Section BB'

Appendix A: Previous Subsurface Exploration Data
Appendix B: Previous Soil Laboratory Test Data
Appendix C: Slope Stability Evaluation

FIGURES



LEGEND

- APPROXIMATE PREVIOUS BORING LOCATION (1982)
- APPROXIMATE PREVIOUS BORING LOCATION (2000)
- APPROXIMATE PREVIOUS BORING LOCATION (2004)
- APPROXIMATE PREVIOUS BORING LOCATION (2003 / 2005)
- APPROXIMATE CROSS SECTION LOCATION (SEE FIG. 3)

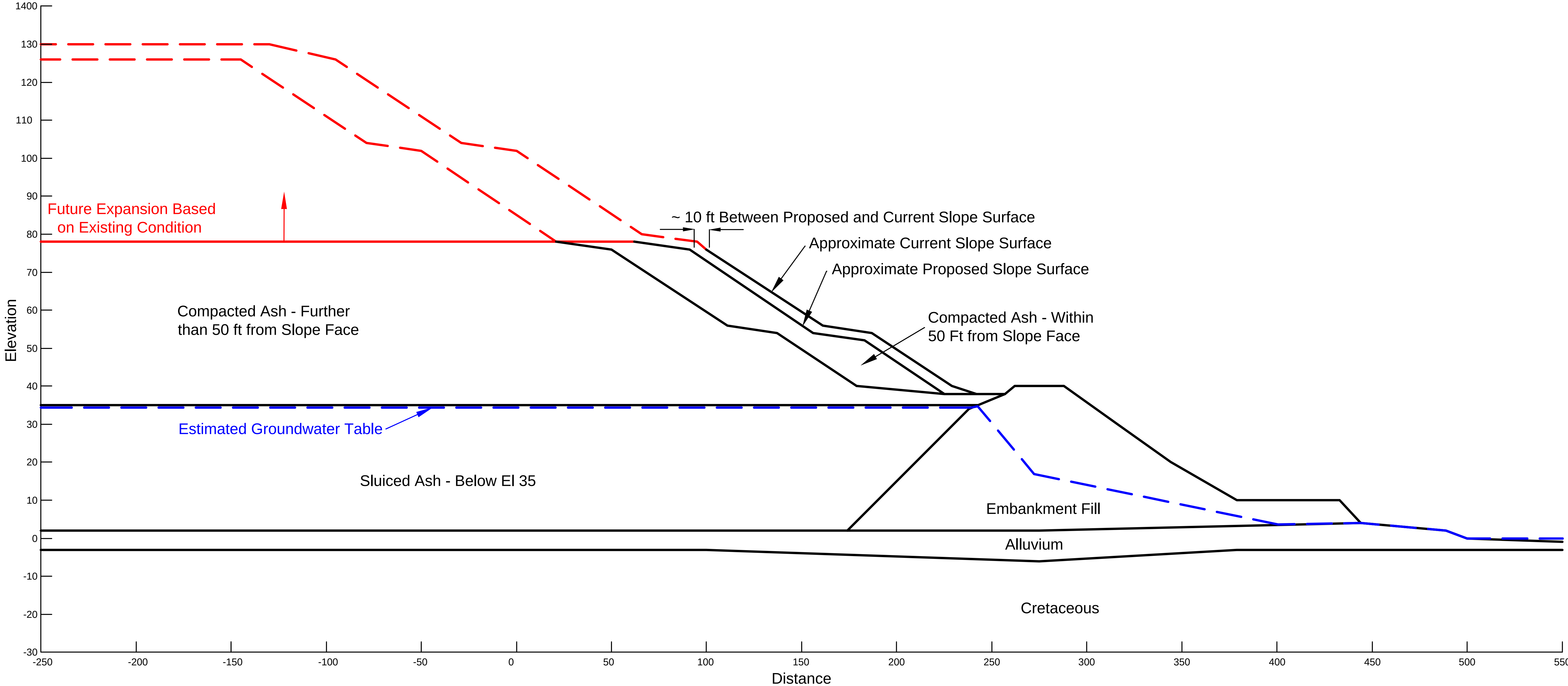
Base Plan by Ash AxisGeoSpatial LLC, Dated 12/11/13.

SCALE: 1"=350'

0 350' 700'

Schnabel ENGINEERING	CHESTERFIELD POWER STATION UPPER (EAST) ASH POND CHESTERFIELD COUNTY, VIRGINIA	Figure Name: BORING LOCATION PLAN	Done: P. JOHNSTON	Figure Number: 2
		Project Number: 14213000	Reviewed: E. MORRIS	Date: AUGUST 2014

G:\2014\Richmond\14213000 Dominion Chesterfield PS Ash Pond Study\03-SE Products\01-CAD\Ash Pond BLP.dwg, 8/13/2014 1:24:19 PM



NOTES:

1) Existing ground surface based on plans by AxisGeoSpatial LLC, dated 12/11/13

2) Closure plan ground surface based on plans by agi Consultants dated 10/27/97

Schnabel ENGINEERING	CHESTERFIELD POWER STATION UPPER (EAST) ASH POND CHESTERFIELD COUNTY, VIRGINIA	Figure Name: CROSS SECTION B-B'	Done: P. JOHNSTON	Figure Number: 3
		Project Number: 14213000	Reviewed: E. MORRIS	Date: AUGUST 2014

SCHNABEL ENGINEERING ASSOCIATES CONSULTING ENGINEERS	TEST BORING LOG	BORING NO.: B-5

PROJECT: ASH DISPOSAL POND; CHESTERFIELD POWER STATION							SHEET NO. 1 OF 1
CLIENT: VEPCO							JOB NO. V82481
BORING CONTRACTOR: FOUNDATION TEST SERVICE					DRILL: CME-55		ELEVATION: 35.5±
WATER LEVEL DATA					DRIVE	SAMPLER	CASING SIZE: 2½"
	DATE	TIME	DEPTH	CAVED	TYPE	S.S.	DATE START: 11/3/82
ENCOUNTERED	11/3	-	DRY	-	DIA.	2" O.D.	DATE FINISHED: 11/3/82
AFTER CASING PULLED	11/3	11:03	30.0'	32.0'	WT.	140 #	DRILLER: B. SPIERENBURG
22 HR. READING	11/4	8:50	33.2'	34.2'	FALL	30"	INSPECTOR: S. COPLEY

STRATUM	DEPTH FT.	ELEV. 35.5±	BLOWS ON SAMPLE SPOON, PER 6	SYMBOL	IDENTIFICATION	REMARKS
A			16+19+14	S	FINE SILTY SAND, PROBABLE FILL, TRACE ORGANIC MATTER, DRY - BROWN (SM)	PROBABLE FILL
	4.0	30	8+11+13	S	FINE SILTY SAND, WITH MICA, TRACE ORGANIC MATTER, MOIST - BROWN (SM)	
			6+9+11	S	do, TRACE SILT	PLEISTOCENE SEDIMENTS
		20	7+11+12	S	do, TRACE SILT	
C			8+11+11	S	do, SOME SILT	
		10	20+33+40	S	do, WITH FINE TO MEDIUM GRAVEL	
	29.0		77+23/3"	S	FINE TO MEDIUM SAND, TRACE SILT WITH FINE TO MEDIUM GRAVEL, DRY - BROWN (SP)	
	34.0	0	33+24+24	S	FINE TO COARSE SILTY CLAYEY SAND WITH FINE TO MEDIUM GRAVEL, MOIST - TAN TO BROWN (SC)	CRETACEOUS SEDIMENTS
F	40.0		34+30+17	S	do, LIGHT GRAY	
					BORING TERMINATED AT 40.0 FT	Caved and dry at 8.7' on 11/10/82

SCHNABEL ENGINEERING ASSOCIATES CONSULTING ENGINEERS	TEST BORING LOG	BORING NO.: B-6

PROJECT: ASH DISPOSAL POND: CHESTERFIELD POWER STATION							SHEET NO. 1 OF 1
CLIENT: VEPCO							JOB NO.: V82401
BORING CONTRACTOR: FOUNDATION TEST SERVICE					DRILL: CME-55		ELEVATION: 15.0±
WATER LEVEL DATA					DRIVE	SAMPLER	CASING SIZE: 3½"
	DATE	TIME	DEPTH	CAVED	TYPE	S. S.	DATE START: 11/3/82
ENCOUNTERED	11/3	-	-	-	DIA.	2" O.D	DATE FINISHED: 11/3/82
AFTER CASING PULLED	11/4	-	-	-	WT.	140 #	DRILLER: B. SPIERENBURG
__ HR. READING	SEE TABLE BELOW				FALL	30"	INSPECTOR: S. COPLEY

STRATUM	DEPTH FT.	ELEV. 15.0±	BLOWS ON SAMPLE SPOON, PER 6	SYMBOL	IDENTIFICATION	REMARKS
			12+14+17	S	FINE SILTY SAND, PROBABLE FILL, TRACE FINE GRAVEL, MOIST - BROWN (SM)	PROBABLE FILL
A	4.0					
		10	4+4+5	S	FINE TO MEDIUM SANDY SILT PROBABLE FILL, TRACE ORGANIC MATTER, MOIST - BROWN (ML)	
	9.0					
			2+3+2	S	FINE SILTY SAND, TRACE ORGANIC MATTER, VERY MOIST - BROWN (SM)	
C		0	11+65+15	S	do, FINE TO COARSE, SOME SILT, WITH FINE TO MEDIUM GRAVEL, WET	PLEISTOCENE SEDIMENTS
			7+9+11	S	do, FINE TO MEDIUM SAND	
	20.3					
F					CLAYEY SILT, TRACE FINE SAND, MOIST - BROWN (MH)	CRETACEOUS SEDIMENTS
	23.5					
E	25.0	-10	5+9+15	S	FINE TO MEDIUM SANDY SILTY CLAY, WITH FINE TO MEDIUM GRAVEL, MOIST - LIGHT GRAY (CL)	

BORING TERMINATED AT 25.0 FT

Water Observation Well Installed to 25.0 ft

Date	Time	Elev. W.L.	Remarks
11/4	9:05 a.m.	1.5	1¼" pvc
11/5	9:30 a.m.	1.7	1 day reading
11/10	9:54 a.m.	1.8	6 day reading

SCHNABEL ENGINEERING ASSOCIATES CONSULTING ENGINEERS	TEST BORING LOG	BORING NO.: B-7

PROJECT: ASH DISPOSAL POND; CHESTERFIELD POWER STATION	SHEET NO. 1 OF 1
CLIENT: VEPCO	JOB NO.: V82481
BORING CONTRACTOR: FOUNDATION TEST SERVICE DRILL: CME-55	ELEVATION: 12.0±

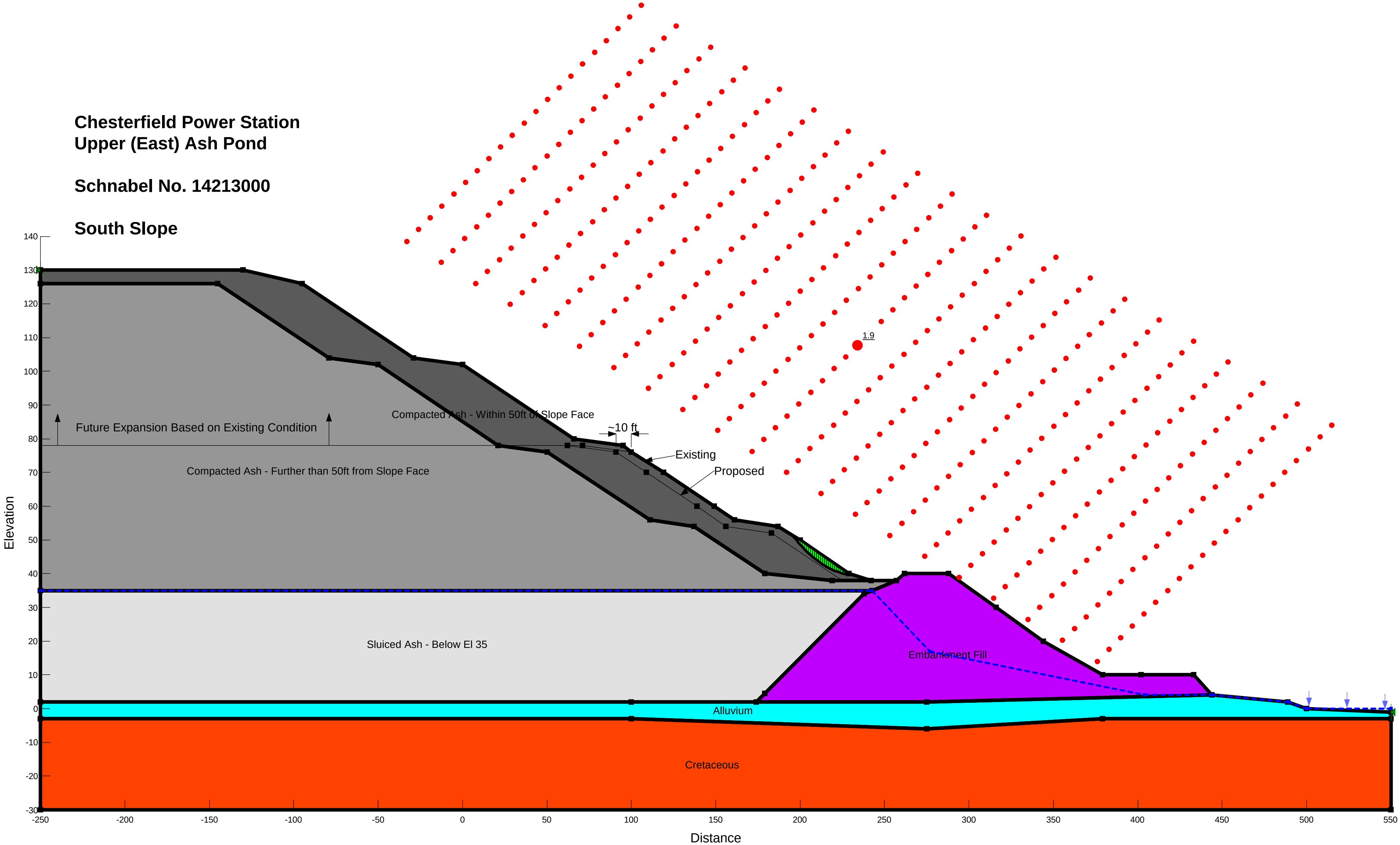
WATER LEVEL DATA	DATE	TIME	DEPTH	CAVED	DRIVE	SAMPLER	
					TYPE	S. S.	CASING SIZE: 2½"
ENCOUNTERED	11/3	8:00	19.0'	-	DIA.	2" O.D.	DATE START: 11/2/82
AFTER CASING PULLED	11/3	8:38	DRY	20.0'	WT.	140 #	DATE FINISHED: 11/3/82
2 DAY READING	11/5	9:26	DRY	9.3'	FALL	30"	DRILLER: B. SPIERENBURG
							INSPECTOR: S. COPLEY

STRATUM	DEPTH FT.	ELEV. 12.0±	BLOWS ON SAMPLE SPOON, PER 6"	SYMBOL	IDENTIFICATION	REMARKS
A		10	14+17+19	S	FINE SILTY SAND, PROBABLE FILL, TRACE FINE GRAVEL, DRY - BROWN (SM)	PROBABLE FILL
	4.0					
C			14+18+12	S	FINE SILTY SAND, MOIST - BROWN (SM)	PLEISTOCENE SEDIMENTS
			5+3+2	S	do, TRACE FINE GRAVEL	
		0				
	14.0					
F			12+41+33	S	MEDIUM TO COARSE SILTY CLAYEY SAND WITH FINE TO COARSE GRAVEL, MOIST - LIGHT GRAY (SC TO CC)	CRETACEOUS SEDIMENTS
			20+33+51	S	do, WET - TAN	
		-10				
	25.0		18+31+37	S	do, GRAY	
					BORING TERMINATED AT 25.0 FT	

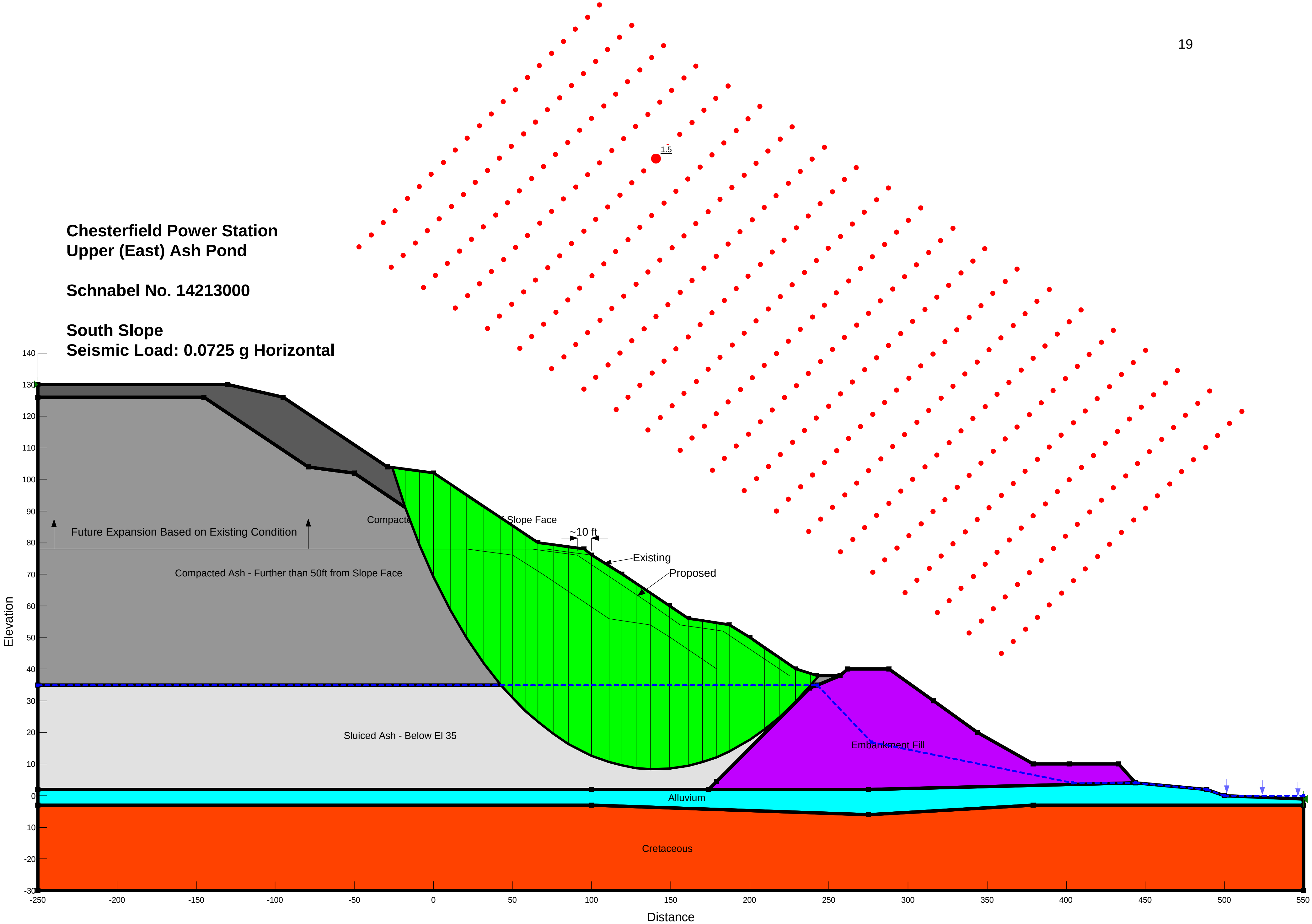
APPENDIX C

SLOPE STABILITY EVALUATION

Effective Stress Analysis
Effective Stress Analysis - Seismic



Name: Cretaceous Model: Mohr-Coulomb Unit Weight: 140 pcf Cohesion': 0 psf Phi': 40 °
Name: Alluvium Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: Compacted Ash - Further than 50ft from Slope Face Model: Mohr-Coulomb Unit Weight: 100 pcf Cohesion': 0 psf Phi': 31 °
Name: Compacted Ash - Within 50ft of Slope Face Model: Mohr-Coulomb Unit Weight: 105 pcf Cohesion': 0 psf Phi': 32 °
Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °
Name: Sluiced Ash - Below El 35 Model: Mohr-Coulomb Unit Weight: 95 pcf Cohesion': 0 psf Phi': 24 °



Name: Cretaceous Model: Mohr-Coulomb Unit Weight: 140 pcf Cohesion': 0 psf Phi': 40 °
Name: Alluvium Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: Compacted Ash - Further than 50ft from Slope Face Model: Mohr-Coulomb Unit Weight: 100 pcf Cohesion': 0 psf Phi': 31 °
Name: Compacted Ash - Within 50ft of Slope Face Model: Mohr-Coulomb Unit Weight: 105 pcf Cohesion': 0 psf Phi': 32 °
Name: Embankment Fill Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °
Name: Sluiced Ash - Below El 35 Model: Mohr-Coulomb Unit Weight: 95 pcf Cohesion': 0 psf Phi': 24 °

Geotechnical Engineering and Groundwater Hydrology Study, 1982

Contract V82481, Geotechnical Engineering and Groundwater Hydrology Services, Ash Disposal Pond, Chesterfield Power Station

Final Copy

SCHNABEL ENGINEERING ASSOCIATES
P. C.
CONSULTING GEOTECHNICAL ENGINEERS

December 20, 1982

JAMES J. SCHNABEL P. E.
RAY E. MARTIN PH. D., P. E.
RAYMOND A. DeSTEPHEN P. E.

ONE WEST CARY STREET
RICHMOND, VIRGINIA 23220
804: 649-7035

Virginia Electric and Power Company
700 East Franklin Street
P. O. Box 564
Richmond, Virginia 23204

Attn: Mr. R. W. Olney

Subject: Contract V82481, Geotechnical Engineering and Groundwater Hydrology Services, Ash Disposal Pond, Chesterfield Power Station

Gentlemen:

Submitted herewith are six copies of our geotechnical engineering report for the above referenced project. This study was conducted under our agreement for MPPA-1026, Task Item 23, W. O. No. 18100476.

The study included: (A) Subsurface Investigation and Sampling, (B) Logging of Test Borings, (C) Soil Laboratory Testing, and (D) a Geotechnical Engineering and Groundwater Hydrology Study. The engineering analysis included evaluation of test borings, geological and soil test data to develop the following:

1. Estimated subsurface profiles along the proposed dike and within the pond, and subsurface conditions within the bottom ash recovery area.

2. Development of regional geology and assessment of groundwater flow direction and gradient within the study area.

3. Recommendations regarding dike configuration including practical limitations of pond base grade, dike side slopes, etc.

4. Development of soil strength parameters and slope stability analysis for the proposed dike, including natural and fill slopes.

5. Assessment of groundwater contamination potential and determination of liner requirements if necessary.

We appreciate the opportunity to be of service for this project. Please do not hesitate to contact us if clarification is needed for any aspect of this report.

Very truly yours,

SCHNABEL ENGINEERING ASSOCIATES, P.C.

James J. Seli
Senior Staff Engineer

Raymond A. DeStephen, P.E.
Commonwealth of Virginia

JJS:RAD:maj

GEOTECHNICAL ENGINEERING
AND
GROUNDWATER HYDROLOGY STUDY
ASH DISPOSAL POND
VEPCO CHESTERFIELD POWER STATION

Table of Contents

1. CONCLUSIONS AND RECOMMENDATIONS

Based on the information contained in this report, the following summary of conclusions and recommendations is presented:

a. The site consists of sandy Pleistocene age surface soils (Stratum C) which have been partially mined. These soils will be excavated for construction of the pond and are underlain by much older pre-consolidated soils of Cretaceous age, as shown on the soil profiles of Sheets 1 through 3. The Cretaceous soils are low to very low permeable silty clays and clayey sands designated Strata E and F, respectively.

b. A large excess of excavated material is anticipated at the site. We recommend that this material be stockpiled north of and adjacent to the site either for future use at the site, as borrow fill, or to reclaim this area. Consideration should be given to compaction and placement requirements if the filled area is to be used for future construction of facilities.

c. Groundwater flow at the site is east to southeast, with discharge to the old river channel which borders the site. Present seepage is slow, moving under a very low gradient of about 0.01 ft/ft.

d. Contamination potential for water supply systems in the area due to leaching from the new facility is remote. However, if no environmental controls are incorporated in the design, there is some potential for elevated contaminant levels within the seepage discharging to the James River.

e. Seepage beneath and within the new pond embankment under the imposed head conditions can be adequately controlled by use of a slurry cutoff wall around the perimeter dike. Comparative budget costs indicate this to be a less expensive environmental control than a pond liner as described herein, and will prevent any significant seepage flow to the James River.

f. Background water quality samples should be obtained prior to construction in monitoring wells B-1 and B-4, within the Old James River Channel and in proposed monitoring wells.

g. The new dike will be partially comprised of the existing natural river banks and roadway embankments. These will be regraded and new fill added as necessary to reach design crest grade, El 24$^{\pm}$. Crest width will be 20 ft with 2.5H:1V side slopes. A small toe berm will be necessary along the north dike extending into the abandoned river channel. Stability analyses were performed for various situations under steady seepage conditions with results shown on Sheets 4 and 5.

h. Bottom ash may be stockpiled within the general bottom ash recovery area, but should not be placed within 200 ft of the existing river bank to ensure stability along the channel. Stability analysis of the river channel was beyond the scope of this study. Conveyor foundations may be supported on spread footings sized for a design allowable soil bearing pressure of 2000 psf.

i. Compacted embankment fill, pipe backfill, subgrades, and slurry wall placement should be inspected by the Geotechnical Engineer to verify that the materials and installation meet the requirements described herein.

This study may be made available to prospective bidders for informational purposes. We would recommend that the project specifications contain the following statement:

> "A geotechnical engineering report has been prepared for this project by Schnabel Engineering Associates. This report is for informational purposes only and should not be considered part of the contract documents. The opinions expressed in this report are those of the Geotechnical Engineer and represent his interpretation of the subsoil conditions, tests, and the results of analyses which he has conducted. Should the data contained in this report not be adequate for the contractor's purposes, the contractor may make, prior to bidding, his own investigation, tests, and analyses. The report may be examined by bidders at the VEPCO office."

The test boring logs and location plan included in Appendix B should be included in the contract documents.

2. SITE DESCRIPTION, PROPOSED CONSTRUCTION AND SUBSURFACE CONDITIONS

Site Description

The proposed ash disposal pond is to be located within an abandoned sand and gravel excavation located on Farrar Island in Chesterfield County, Virginia. The site is enclosed by a former meander bend of the James River, immediately south of VEPCO's Chesterfield Power Station at Dutch Gap. Access to the site is by an abandoned haul road, which crosses the old river channel that bounds the north side of the site. The ash disposal pond currently in use is located adjacent to the northwest corner of the site.

The site proper is approximately 4000 x 1800 ft in areal extent, elongated in an east-west direction as indicated on Sheet 6. The majority of the site ranges from open to densely wooded and an unpaved road encircles the portion of the site contemplated for the ash pond. A lake created by past sand and gravel mining operations by Lone Star Industries, approximately 2400 x 700 ft in overall plan dimensions, occupies the western part of the site. Surface grades at the site are very irregular due to the previous mining, but one of the steepest grades occurs along the bank of the old river channel bounding the north side of the site. Topographic relief along the crest of the bank ranges from El 25 to El 35.5, with water surface elevations in the adjacent lake and abandoned river channel at approximately El 3 and 2, respectively. Topographic relief along the south side of the site is less pronounced but surface grades typically vary from El 8 to 23.

Information related to past use of the site was given to us by personnel from Lone Star Industries. The extreme northeast end of the site, once called Henricopolis, was settled in 1608. During the Civil War the original river meander enclosing the site was cut off by the Dutch Gap Channel in order to shorten the navigation route up river to Richmond. Prior to beginning sand and gravel mining operations in approximately 1946, Farrar Island was "plateau-like" in appearance. Sand and gravel operations conducted by Lone Star Industries over a 10 to 11 year period extended to maximum depths of 30 to 35 ft, reportedly terminating below the water table on silts and clays of the Patuxent Formation. Lake soundings conducted during this study indicate water depths range from 3 to 12 ft or El 0 to -9, as shown on Sheet 6.

Proposed Construction

The construction of an ash disposal facility for the Chesterfield Power Station is planned on this site. The proposed facility is to consist of an ash pond contained by an earthen embankment, which will correspond to the road which encircles the area. The crest height will be limited since the ash transmission and discharge facilities in existance will not allow for inflow above El 20$\pm$. The proposed embankment will incorporate the existing river banks and road embankment where possible. The balance of the embankment will be constructed of materials excavated within the proposed pond interior.

Subsurface Conditions

The subsurface exploration program for this project consisted of drilling 16 test borings, installation of five groundwater observation wells and two monitoring wells, drilling of six hand auger probes and performance of 22 lake bottom soundings. This work was completed in November, 1982. Test borings were drilled by Foundation Test Service, Bethesda, Maryland under our inspection. Hand augers and lake soundings were completed by engineering and geologist personnel from our office. All test boring and hand auger logs are included in Appendix B. Locations and sounding data are presented on the Test Boring Location Plans, Sheets 6 and 7, also included in Appendix B.

Of the 14 borings drilled in the proposed ash disposal area, 11 were drilled along the proposed embankment crest and three within the disposal area. Six hand augers were also drilled along the south edge of the lake. Two bulk samples were taken in Borings B-13 and B-14 to evaluate representative near surface material for use as borrow in construction of the embankments. One bulk sample of ash was also taken from the existing disposal area.

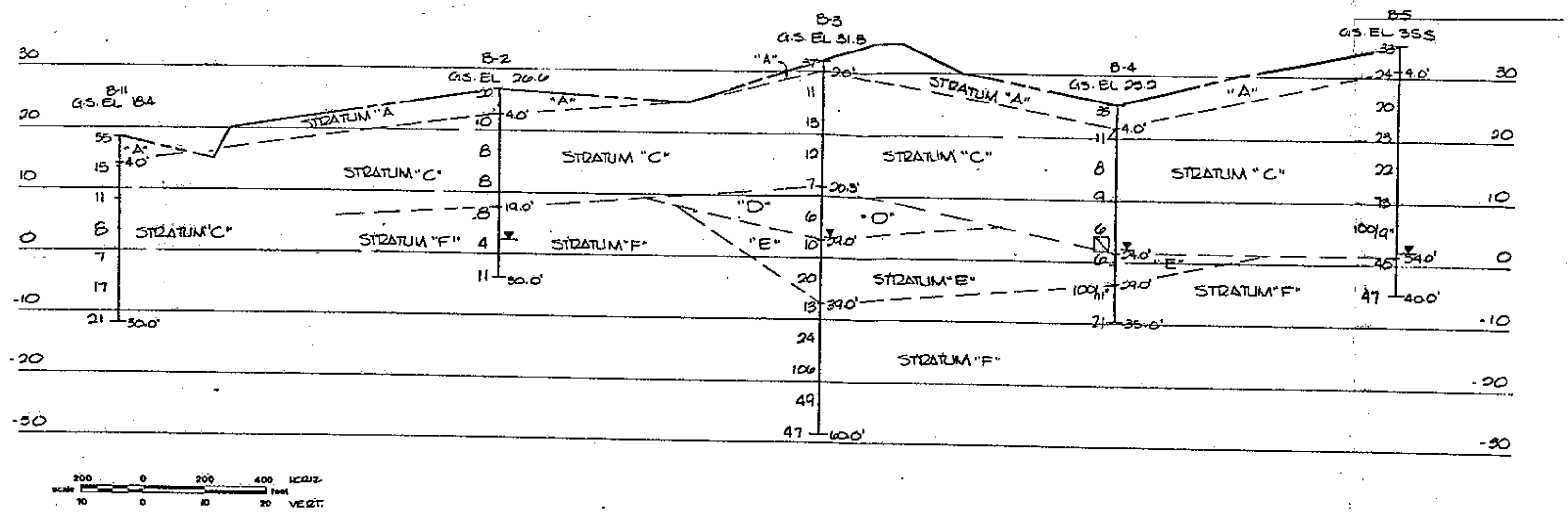
Borings B-15 and B-16 were drilled north of the proposed disposal area for the purpose of evaluating subsurface conditions in the vicinity of the bottom ash recovery area.

Estimated subsurface profiles are included on Sheets 1 through 3 at the end of this section. Based upon the test boring data, the following generalized strata underlie the site to the depths investigated:

Stratum A:	From the ground surface to depths of 2 to 14 ft	Brown to gray fine to coarse silty to clayey silty sand and sandy silt, FILL and PROBABLE FILL, trace gravel, with organic matter (SM to ML); loose to compact (N = 6 to 55)
Stratum B:	Below Stratum A in Boring B-1 to a depth of 54 ft	Brown to gray fine sandy SILT to silty SAND with clay lenses (ML to SM); fine to medium SAND, trace silt (SP); loose to firm (N = 4 to 10) and CLAY with sand lenses (CH); soft consistency (N = 2)

Stratum C:	Below Stratum A to depths of 7.5 to 34 ft	Brown to gray fine to coarse SAND with variable amounts of silt, clayey silt, silty clay and gravel, trace organic matter (SC, SM and SP): loose to very compact (N = 4 to 100+)
Stratum D:	Interbedded with and below Stratum C in Borings B-3 and B-10 to depths of 20 to 29 ft	Brown to gray fine sandy SILTY CLAY and CLAYEY SILT, trace gravel (CL to ML); medium consistency (N = 4 to 6)
Stratum E:	Below Strata C and D and interbedded with Stratum F to depths of 18.5 to 58.5 ft	Brown, gray to green SILTY CLAY, CLAYEY SILT and CLAY, with organic matter and mica (CL, ML and CH); medium to hard consistency (N = 5 to 40)
Stratum F:	Below Strata B and C and interbedded with Stratum E to depths of 18.5 ft to 60 ft, maximum depth of penetration	Brown, gray to tan, fine to coarse SAND with variable amounts of silt, clayey silt, silty clay and gravel (SM, SC); loose to very compact (N = 4 to 100+)

The above N values indicate the low and high Standard Penetration Test (SPT) resistances encountered in a particular layer as determined from the number of blows required to drive a 2 inch O.D. 1-3/8 inch I.D. sampling spoon one foot or fraction thereof, using a 140 pound hammer falling 30 inches. This test is conducted after seating the sampler six inches in the bottom of the hole according to ASTM D-1586. If the number of blows in the first six inches of drive exceeded 30, this value was used to determine the N value and was not considered a seating penetration.



SECTION A-A

GENERAL NOTES

1. NUMBERS TO THE LEFT OF THE BORING COLUMNS INDICATE NUMBER OF BLOWS REQUIRED TO DRIVE A 2 INCH O.D., 1 3/8 INCH SAMPLING SPOON 12 INCHES USING A 140 LB HAMMER FALLING 30 INCHES.
2. ESTIMATED GROUNDWATER LEVEL INDICATED BY ▼ : THESE LEVELS ARE ONLY ESTIMATES FROM AVAILABLE DATA AND MAY VARY WITH PRECIPITATION, POROSITY OF SOIL, SITE TOPOGRAPHY, ETC.
3. G. S. = GROUND SURFACE ⍂ = UNDISTURBED TUBE SAMPLE ▼ = LAKE LEVEL
4. TEST BORINGS DRILLED BY AYERS AND AYERS, INC., RICHMOND, VIRGINIA IN NOVEMBER, 1982.
5. THIS DRAWING CONTAINS INTERPRETATION OF TEST BORING DATA AND SHOULD NOT BE USED AS PART OF THE CONTRACT DOCUMENTS.
6. THESE PROFILES WERE DEVELOPED BY INTERPOLATION BETWEEN WIDELY SPACED BORINGS, ONLY AT THE BORING LOCATIONS SHOULD THEY BE CONSIDERED AS AN APPROXIMATELY ACCURATE REPRESENTATION AND THEN ONLY TO THE DEGREE IMPLIED BY THE NOTES ON THE BORING LOGS.
7. TEST BORINGS INSPECTED BY SCHNABEL ENGINEERING ASSOCIATES.

STRATA DESCRIPTIONS

STRATUM A: BROWN TO GRAY FINE TO COARSE SILTY TO CLAYEY SILTY SAND AND SANDY SILT, FILL AND PROBABLE FILL, TRACE FINE TO MEDIUM GRAVEL, WITH ORGANIC MATTER (SM TO ML); LOOSE TO COMPACT (N = 6 TO 55)

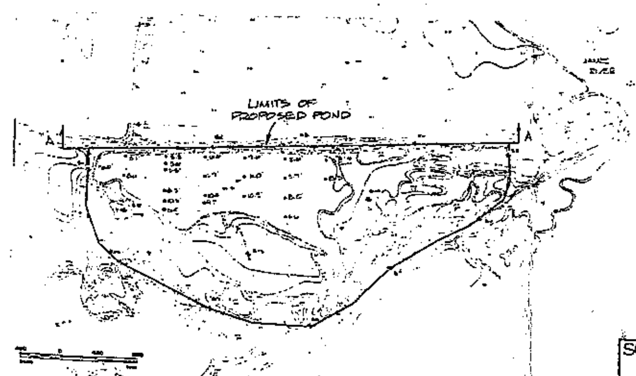
STRATUM B: BROWN TO GRAY FINE SANDY SILT TO SILTY SAND WITH CLAY LENSES (ML TO SM); FINE TO MEDIUM SAND, TRACE SILT (SP); LOOSE TO FIRM (N = 4 TO 10) AND CLAY WITH SAND LENSES (CH); SOFT CONSISTENCY (N = 2)

STRATUM C: BROWN TO GRAY FINE TO COARSE SAND WITH VARIABLE AMOUNTS OF SILT, CLAYEY SILT, SILTY CLAY AND GRAVEL, TRACE ORGANIC MATTER (SC, SM AND SP); LOOSE TO VERY COMPACT (N = 4 TO 100+)

STRATUM D: BROWN TO GRAY FINE SANDY SILTY CLAY AND CLAYEY SILT, TRACE GRAVEL (CL TO ML); MEDIUM CONSISTENCY (N = 4 TO 6)

STRATUM E: BROWN, GRAY TO GREEN SILTY CLAY, CLAYEY SILT AND CLAY, WITH VARIABLE AMOUNTS OF SAND AND GRAVEL, ORGANIC MATTER AND MICA (CL ML AND CH); MEDIUM TO HARD CONSISTENCY (N = 5 TO 40)

STRATUM F: BROWN, GRAY TO TAN, FINE TO COARSE SAND WITH VARIABLE AMOUNTS OF SILT, CLAYEY SILT, SILTY CLAY AND GRAVEL (SM, SC); LOOSE TO VERY COMPACT (N = 4 TO 100+)



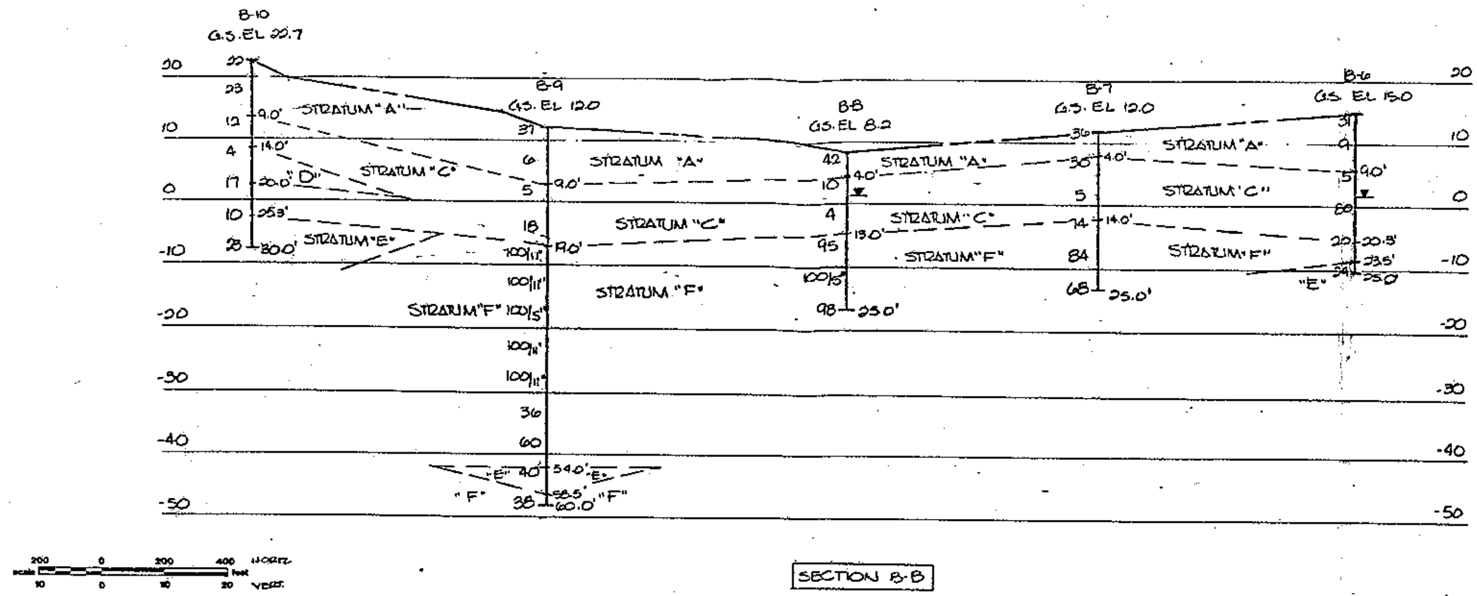
SECTION LOCATION PLAN

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS, SOIL MECHANICS AND FOUNDATIONS
BETHESDA, MARYLAND · RICHMOND, VIRGINIA

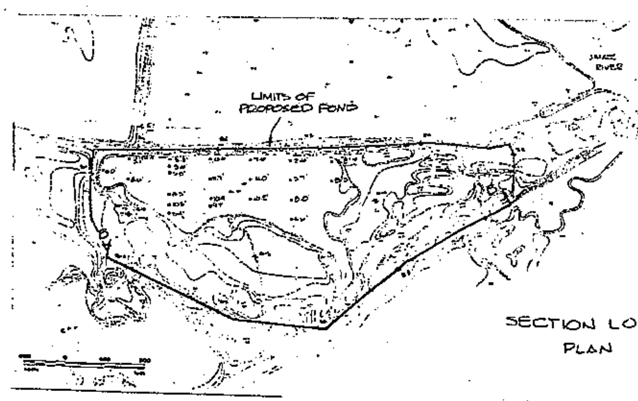
ASH DISPOSAL POND

CHESTERFIELD POWER STATION

ESTIMATED SUBSURFACE PROFILES	SCALE AS SHOWN	DATE 12-20-82
	DRAWN BY DPO	CKD BY JS
	DWG NO. VB2481	SHEET NO. 1



SECTION B-B

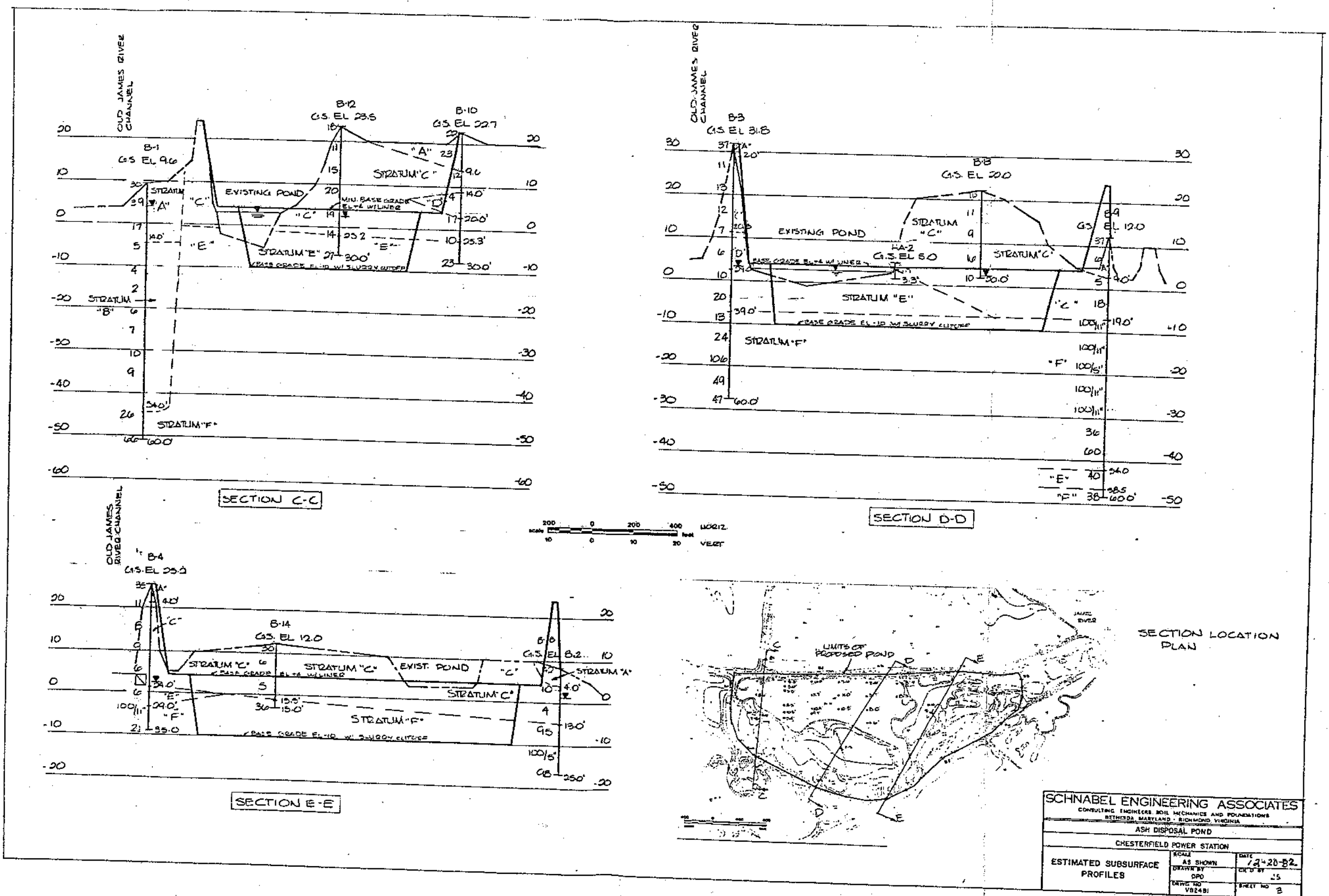


SECTION LOCATION
PLAN

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS, SOIL MECHANICS AND FOUNDATIONS
BETHESDA, MARYLAND • RICHMOND, VIRGINIA

ASH DISPOSAL POND
CHESTERFIELD POWER STATION

ESTIMATED SUBSURFACE PROFILES	SCALE: AS SHOWN	DATE: 12-20-82
	DRAWN BY: DPO	CK'D BY: JS
	DRWG. NO.: V82481	SHEET NO.: 2



OLD JAMES RIVER CHANNEL
B-1
G.S. EL 9.6
B-12
G.S. EL 23.5
B-10
G.S. EL 22.7
EXISTING POND
STRATUM "A"
STRATUM "C"
STRATUM "B"
STRATUM "E"
STRATUM "F"
MIN. BASE GRADE EL+4 W/LINER
GRADE EL -10 W/ SLURRY CUTOFF
SECTION C-C
B-3
G.S. EL 31.8
B-8
G.S. EL 20.0
HA-2
G.S. EL 5.0
B-4
G.S. EL 12.0
SECTION D-D
OLD JAMES RIVER CHANNEL
B-4
G.S. EL 25.0
B-14
G.S. EL 12.0
B-6
G.S. EL 8.2
EXIST. POND
STRATUM "A"
SECTION E-E
HORIZ.
VERT.
LIMITS OF PROPOSED POND
JAMES RIVER
SECTION LOCATION PLAN
SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS: SOIL MECHANICS AND FOUNDATIONS
BETHESDA, MARYLAND · RICHMOND, VIRGINIA
ASH DISPOSAL POND
CHESTERFIELD POWER STATION
ESTIMATED SUBSURFACE PROFILES
SCALE AS SHOWN
DATE 12-20-82
DRAWN BY DPO
CK'D BY JS
DWG. NO. V82481
SHEET NO. 3

3. GEOLOGIC CONDITIONS

Regional Geology

Subsurface conditions in the area just east of the Fall Line, which traverses Richmond, consist basically of unconsolidated sediments of variable thicknesses resting on the Petersburg Granite. These sediments range from Cretaceous age (135 to 65$\pm$ million years old) to Recent age, and were derived from source areas west of Richmond. Consequently, they are thinnest or absent along the Fall Line and thicken in an eastward direction, reaching thicknesses in excess of 3000 ft along the coast. These sediments consist of sand, silt, clay and gravel materials deposited in a variety of environments ranging from deltaic, beach, estuarine to shallow marine. The type of depositional environment has been influenced by the repeated transgression and regression of the Atlantic Ocean since Cretaceous time. This repetitious process has resulted in the erosion and reworking of existing sediments and deposition of new sediments.

The stratigraphic sequence along the James River in southeastern Chesterfield County generally consists of Cretaceous sediments (Patuxent Formation) resting on the Petersburg Granite. The Petersburg Granite is not exposed in the vicinity of the Chesterfield Power Station. The Patuxent Formation of Cretaceous age in turn is overlain by Pleistocene sand and gravel sediments and Recent James River alluvial deposits. We were unable to obtain information on the approximate thickness of the Patuxent Formation in the study area; however, we estimate it to be several hundred feet thick. Pleistocene sediments are generally less than 80 ft thick in the Richmond area. The Mattaponi Formation, Marlboro Clay, Nanjemoy Formation and Calvert Formation often separate the Patuxent from the overlying Pleistocene sediments in areas east of the James River. These formations were not encountered in the test borings, as they have been eroded away in the geologic past. Thus the Patuxent Formation of Cretaceous age is highly preconsolidated with respect to the present overburden, and this accounts for the very stiff consistency of the clay facies and very compact nature of the gravelly sands. Both the Cretaceous and Pleistocene deposits have been eroded by the James River and its tributaries. Recent alluvial sediments occur as channel fillings in existing and former river channels and as floodplain deposits. We understand that up to approximately 160 ft of channel sediments have been encountered in studies conducted in eastern Henrico County for the southeast portion of the Interstate 295 Beltway.

Both the Cretaceous and Pleistocene formations were deposited in deltaic environments. In such environments, granular materials are confined to stream channels and adjacent flood plains, while silt and clays settle out in the deeper water (bays) at the mouth of streams, in swamps and lakes. These deposits were initially homogeneous; however, with time these river systems changed their courses. The ultimate result was the deposition of sand, silt, clay and gravel materials of variable thickness and areal extent due to numerous cutting and filling processes.

Site Geology

Test borings and hand augers drilled at the site encountered fill, Recent James River deposits, and Pleistocene and Cretaceous sediments. Descriptions of the various strata encountered are presented below.

The soils designated Stratum A consist of fill and probable fill materials placed during sand and gravel mining operations on Farrar Island. Materials encountered in Boring B-1 represent fill soils placed during the construction of a roadway across the abandoned James River channel. Most of the remaining Stratum A soils described as Probable Fill, occur along the existing roadway and look very similar to the underlying Pleistocene soils of Strata C and D. However, based upon conversations with Lone Star Industries personnel, fill was placed periodically on the haul road to control rutting, etc. Consequently, we suspect that the upper three to four feet of the roads consist of fill.

Stratum B includes poorly sorted sequences of Recent James River sediments typically ranging from sands to clays with organic matter. Stratum B was encountered only in Boring B-1, which was drilled along the old river channel. This stratum extended to a depth of 54 ft below the existing ground surface.

Strata C and D represent the coarse-grained and fine-grained portions, respectively, of the Pleistocene age river terrace deposits. Although throughout the Richmond area these sediments have been known to be heterogeneous in composition, varying considerably in thickness and areal extent, the test borings indicate that they are relatively uniform in composition and occurrence within the site limits.

Soils of Strata E and F belong to the Patuxent Formation of Lower Cretaceous age. The fine-grained soils are designated Stratum E. The Patuxent consists of fine to coarse-grained feldspar-rich sands and gravels with interbedded silty clays and clays. The granular portion of the formation typically ranges from gray to tan in color, while the clays range from yellow, tan, light gray to green in color. Plant fragments occur locally within the Patuxent clays. The sediments of the Patuxent Formation were deposited in a deltaic environment consisting of prograding deltas with streams emptying into shallow bodies of water. The test borings indicate the Cretaceous sediments vary in composition, thickness and areal extent, as expected. The top of the Cretaceous age gravelly sands and clays range from El 7.6 to -12.

4. GROUNDWATER REGIME

Regional Groundwater Flow

East of the Fall Line, there are two major groundwater aquifers which occur stratigraphically above the Petersburg Granite. The uppermost aquifer consists of the water bearing strata present within the Pleistocene sediments, often termed the Water Table Aquifer. Groundwater flow within this aquifer generally conforms in a subdued manner to ground surface grades, and recharge occurs by the infiltration of precipitation and surface water. This aquifer is in general a dependable source of domestic groundwater supplies.

Groundwater within the underlying Patuxent Formation of Cretaceous age is the primary source of industrial and municipal groundwater supplies in the Coastal Plain region of Virginia. The highest yielding wells in the Richmond area generally occur within this Cretaceous aquifer. Groundwater is developed from the various sand and gravel strata present within the formation. Natural flow within the aquifer is east to southeast with a very low gradient of less than 1%. Recharge to the aquifer is by the leakage of groundwater from the overlying strata, and the direct infiltration of precipitation and surface water where the Patuxent is exposed at the ground surface.

Site Groundwater Conditions

Groundwater was encountered in all of the test borings during drilling. Observation wells consisting of slotted 1¼" pvc pipe were installed in Borings B-2, B-6, B-8, B-12 and B-13 upon completion of drilling for the purpose of obtaining long-term water level readings. Two inch monitoring wells were installed in B-1 and B-4 for obtaining water samples, if required, as well as water level readings.

Long-term water level readings indicate variations in the water table between El 1 and El 5 within the study area. The highest water level was measured in Boring B-1 located along the access road within the old river channel. This reading is probably influenced by the increased head conditions of the existing ash pond. Typically water levels were between El 2 and 3. Based on the water level measurements, groundwater flow beneath the site is in a southeastward direction. In the absence of any effects of the existing ash pond a very low gradient of approximately 0.01 ft/ft occurs.

As a result of the intense river meandering and enclosure of Farrar Island by the James River, groundwater levels as expected are essentially the same level as the river. The slightly higher water levels are probably the result of the inability of surface water to infiltrate and stabilize with the surrounding river level. Topographic data taken from an areal photograph indicated the water level of the interior dredged lake at El 3 at the time the photograph was taken. In the study area, the James River does experience daily changes in water level of approximately three feet as a result of tidal fluctuations. These tidal fluctuations also influence local groundwater levels; however, variations are estimated to be less than one foot. The project site is considered to be in a discharge area, with groundwater flow discharging to the James River. Under the increased head conditions associated with the proposed pond, seepage flow should discharge to the old river channel around the south and east perimeter of the site. No underflow and migration of pond water beyond this boundary is expected. The nearest possible downgradient domestic wells are about 1.5 miles away, along Route 10, and thus will not be effected by the new facility.

5. SOIL LABORATORY TESTING

Three bulk samples, one undisturbed tube sample, and numerous jar samples were tested in the soils laboratory with results included in the summary and graphs of Appendix A. All classifications are in accordance with ASTM D-2487.

Stratum A: Fine to medium sandy silt, fill (ML)

One jar sample of this stratum was tested. This sample contained 53% material by weight passing the No. 200 sieve. This soil had a Plasticity Index of 5 indicating a low plasticity soil. This material is derived from the on-site Pleistocene soils, Strata C and D.

Stratum C: Fine to medium sand, trace silt (SM-SP)

This coarse-grained material varies in the percentage of fines passing the No. 200 sieve with values of 10 to 35% recorded. These values effect the natural permeability as does the soil plasticity and in situ density. The plasticity of this stratum varied from non-plastic to a low plasticity of 8. We estimate natural permeabilities of this stratum based on the above data to be medium to high at about 10^{-4} to 10^{-3} cm/sec (0.3 to 3 ft/day).

Bulk samples of this material were compacted according to ASTM D-698, Standard Proctor, to determine its suitability as embankment and liner material. Relatively high maximum dry densities of 120.1 to 126.1 pcf were recorded for this material. A compacted permeability of 0.75×10^{-5} cm/sec (0.02 ft/day) was obtained. As expected, this compacted permeability is lower than the soil in situ, however, horizontal permeabilities are anticipated to be an order of magnitude higher. Thus, the Stratum C soils which comprise the majority of material to be excavated, are not considered suitable as liner material but may be used as embankment fill.

Shear strength parameters representative of these materials were obtained from triaxial shear tests performed on samples compacted to about 95% of maximum dry density per ASTM D-698. Drained angle of internal friction values of 34^{o} and 36^{o} resulted from these tests. An average value of $\phi' = 35^{o}$ was assumed in our analysis for compacted on site soils.

Stratum D: Clayey silt, some fine sand (MH)

Stratum D represents the discontinuous fine-grained portion of the Pleistocene sediments encountered at the site. The sample tested contained 80% material by weight passing the No. 200 sieve, with a Plasticity Index of 12 indicating medium plasticity. In situ permeabilities of this stratum are estimated to be less than 10^{-6} cm/sec (0.0038 ft/day) based on the grain size analyses.

Stratum E: Clay (CH), clayey silt (MH) and silty clayey sand (SC)

The predominately fine-grained soils of Stratum E contained between 46 and 91% material by weight passing the No. 200 sieve. Medium to high Plasticity Indices of 9 to 32 were recorded for the samples tested. A low natural dry density of 73.3 pcf was obtained for this stratum. A permeability test performed in the laboratory on an undisturbed sample resulted in a very low permeability of 9×10^{-7} cm/sec (0.0025 ft/day). This is typical for this highly preconsolidated clayey stratum.

One triaxial shear test was performed on Stratum E soils. Although the results of this one test, which are included in Appendix A, is insufficient data to establish strength parameters, a usable drained angle of internal friction may be obtained by combining this data with available correlations and previous experience in this type of soil. Angles of internal friction of between 27^{o} and 32^{o} were obtained in this manner. The lower bound of this range was assumed in this analysis.

Stratum F: Fine to coarse sand, some silt to silty clay (SM, SC)

The sample tested from this stratum contained 13% material passing the No. 200 sieve with a low Plasticity Index of 3. This soil is highly pre-consolidated and its high density is illustrated by the very high standard penetration test values obtained during drilling, usually in excess of 100 blows/ft. This high density and the percentage of fines gives this material a very low natural permeability. Based on the grain size curve we have estimated the permeability of this stratum to be on the order of 10^{-6} cm/sec (0.0038 ft/day). This can be further verified by in situ permeability testing. Previous pressuremeter testing within this stratum indicated an angle of internal friction $\emptyset = 43$ to 46°. A conservative value of $\emptyset = 40^{\circ}$ was used in our analysis.

Fly Ash

Fly ash was sampled from the existing ash pond. This sample contained only 32% material by weight passing the No. 200 sieve and thus probably contains some bottom ash. The sample was slurried in the lab and poured into the permeability mold. After a short waiting period, a constant head test was performed resulting in a moderate permeability of 3×10^{-4} cm/sec (1.0 ft/day). This permeability is considered slightly high and is probably the result of the short settling time used prior to testing.

6. GEOTECHNICAL AND GROUNDWATER HYDROLOGY ANALYSES

Several factors were considered in determining the most economical design for the proposed ash disposal facility. These included environmental controls, volumes of suitable and excess materials on site and stability considerations. The most significant cost factors will be disposal and use of excess material, and installation of any environmental controls necessary.

Earthwork Volumes

Although detailed quantity and cost estimates were beyond the scope of this study, we have made a preliminary estimate of the volume of materials within the site area to determine their order of magnitude. As such, the available material within the proposed pond area to El +4 is on the order of 1,000,000 cy, while the estimated volume of fill needed for dike construction is approximately 75,000 cy. The estimated volume within the existing lake is about 600,000 cy. The sandy Pleistocene soils of Stratum C within the site are suitable for dike construction. Since the dike embankment volume is only a fraction of the material to be excavated, it is clear that dike geometry is not critical in terms of overall cost.

Considerable cost savings may be realized by disposing of the excess material on site within a one mile haul distance. This material could then be sold as fill material or used for the same purpose at the station. We estimate the cost of on site disposal to be about \$2/cy compared to \$4/cy for off site disposal using truck hauling. These values were obtained from a local contractor. We recommend that consideration be given to disposal of material at the station north of and adjacent to the site. Station planning should consider the possible reclaiming of this area and its future uses so that an assessment of the type of placement and compaction of the fill can be made. Material stockpiled should be kept free of organic matter.

Environmental Controls

In order to determine the environmental controls necessary on this project, the contamination potential of the ash leachate was assessed. This entailed an evaluation of the rate of the leachate migration under the imposed head conditions, its migration through the existing geologic materials, and impact on groundwater and surface water in the area.

Results of U. S. EPA extraction procedure toxicity tests carried out on the Chesterfield Station fly ash and bottom ash during a previous study by others are shown in Table I. These results indicate that the ash leachate is not considered a hazardous material. However, several parameters will have concentrations above state drinking water standards.

Groundwater flow will be east to southeast towards the old James River channel, which is considered the discharge point for the site. No flow from the ash pond is expected beyond this boundary and thus, the contamination potential of the area groundwater supply is remote. Seepage flow to the river with elevated levels of constituents is possible, however, since attenuation of contaminents will not be significant within the Stratum C soils which underlie the ash pond and form portions of the natural embankments. Moderate seepage velocities of 0.5 ft/day are possible through the more highly permeable Stratum C soils under the imposed head conditions.

This seepage velocity will provide an estimated discharge of 0.15 mgd to the river. This is considered an upper limit since as the hydraulic head conditions within the pond become greater, layering of ash will take place tending to inhibit flow. This ability to inhibit flow is very dependent, however, on the manner in which the pond is filled, and should

RESULTS OF U. S. EPA EXTRACTION PROCEDURE TOXICITY TEST

TABLE I

Parameter	Unit	U. S. EPA EP Toxicity Test Chesterfield Power Station Bottom Ash	Fly Ash	U. S. EPA Hazardous Waste EP Toxicity Standards - Max. Level
Filterable Residue @ 180° C	mg/ℓ	144	799	-
Chloride	mg/ℓ	2.0	18	-
Fluoride	mg/ℓ	0.30	0.44	-
Nitrate	mg/ℓ N	0.1	0.3	-
Sulfate	mg/ℓ	1	430	-
Hardness	mg/ℓ $CaCO_3$	110	340	-
Metals:				
Aluminum	mg/ℓ	1.4	46	-
Antimony	mg/ℓ	<0.001	0.084	-
Arsenic	mg/ℓ	<0.001	0.057	5.0
Barium	mg/ℓ	0.46	0.24	100.0
Beryllium	mg/ℓ	0.003	0.030	-
Boron	mg/ℓ	<0.1	2.0	-
Cadmium	mg/ℓ	<0.01	<0.01	1.0
Calcium	mg/ℓ	28	90	-
Chromium	mg/ℓ	<0.01	<0.01	5.0
Copper	mg/ℓ	0.12	0.40	-
Iron	mg/ℓ	2.8	0.9	-
Lead	mg/ℓ	<0.01	<0.01	5.0
Magnesium	mg/ℓ	1.3	11	-
Manganese	mg/ℓ	0.7	0.7	-
Mercury	mg/ℓ	<0.0005	<0.0064	0.2
Nickel	mg/ℓ	0.2	0.4	-
Potassium	mg/ℓ	1.1	23	-
Selenium	mg/ℓ	<0.001	0.019	1.0
Silver	mg/ℓ	<0.001	<0.001	5.0
Sodium	mg/ℓ	0.8	9.9	-
Thallium	mg/ℓ	<0.001	0.006	-
Vanadium	mg/ℓ	<0.01	0.02	-
Zinc	mg/ℓ	0.38	1.9	-

not be counted on entirely as a deterent to leachate migration from the site. Based on the above considerations, we believe it would be prudent to provide an environmental control limiting migration from the facility. This control will be necessary to inhibit seepage through the natural Stratum C soils.

Two types of controls were considered in detail, a liner and a slurry wall cutoff. Liners considered were bentonite, synthetic, and compacted clay. Synthetic liners were not considered in detail due to their higher relative cost. The great majority of the on site soils to be excavated consist of the sandy Stratum C soils which are not of adequate quality to be used as an effective liner. Thus, a compacted clay liner would require the use of off site borrow. At an estimated unit price of \$6/cy, a minimum 1.0 ft thick liner would have a comparative cost of \$12,980/ac. A two inch bentonite liner applied at the rate of about 2 lb/ft^2 could also be used, but would also be expensive. Assuming a bentonite cost of \$275/ton, the comparative cost for a bentonite liner is \$11,980/ac. Other drawbacks to providing a liner include limiting the base grade of the pond to about El 4, thus losing the volume within the existing lake since filling above the watertable would be required in order to place liner materials. For these reasons, we believe a liner will not be the most economical environmental control on this project.

In general, as pond size becomes larger, cutoffs around the pond perimeter to low-permeable strata become increasingly cost competitive when compared to liners. We believe a slurry wall cutoff will cost about \$3/ft^2 and will be the most economical and effective environmental control for this project for the following reasons:

(1) Actual installation costs will be less than for a liner. We have estimated a comparative cost of \$8820/ac of pond.

(2) The present dredge lake volume can be utilized as pond volume.

(3) The pond base grade can be lowered to El-10, which will afford several additional years of pond service, depending on the ash generation rate.

(4) The cost of other seepage controls such as toe drains along the length of the pond dike will not be necessary.

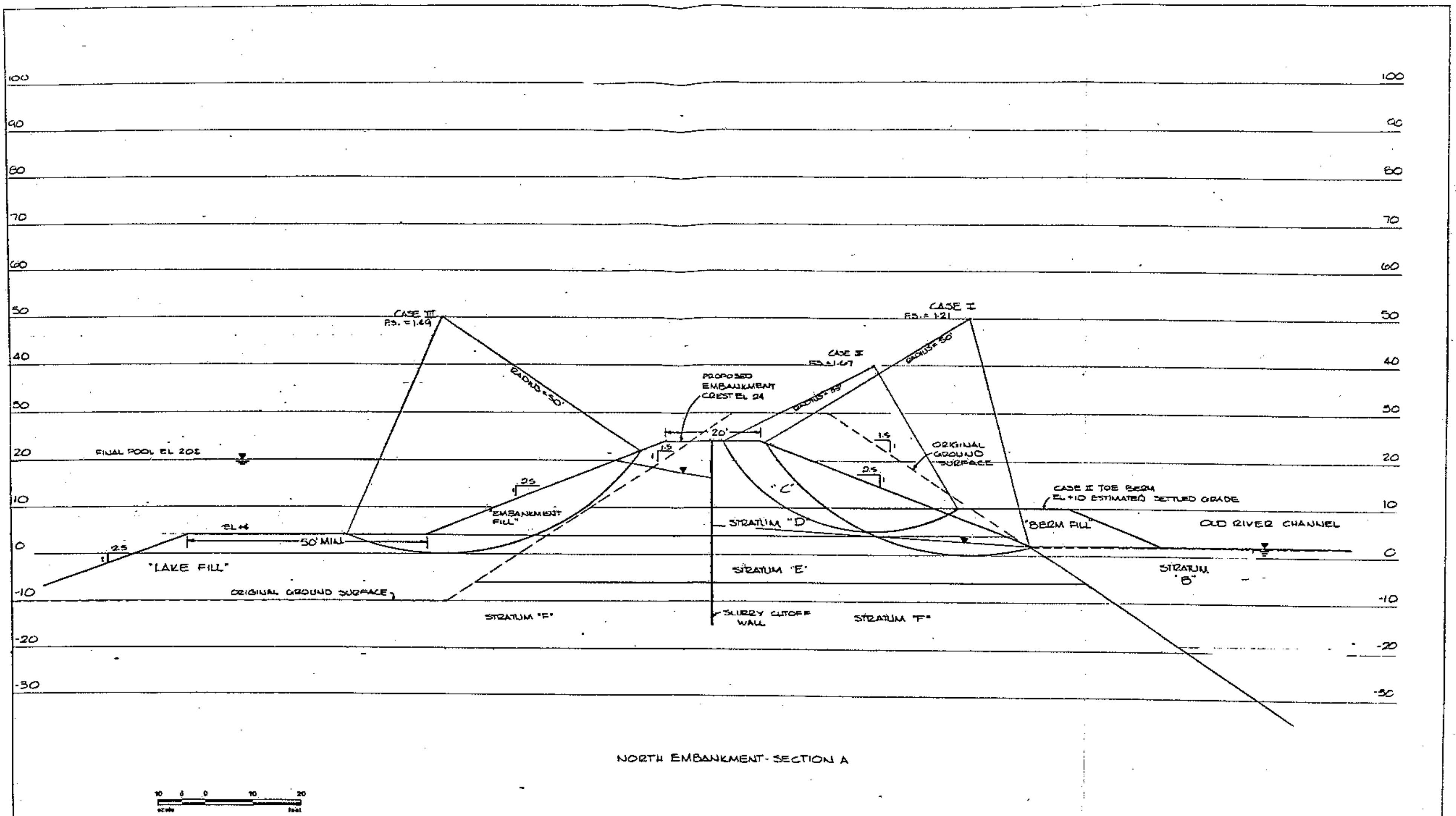
The slurry cutoff may consist of either soil bentonite or cement bentonite, and should extend to El-15. Since the slurry cutoff will be formed from the top of the surrounding dike at El 24, the slurry wall depth will be about 40 ft. The slurry wall should at all times extend into the very low permeable compact sands of Stratum F and silty clays of Stratum E.

Installation of the slurry wall will effectively reduce the seepage velocity from the site by several orders of magnitude, to an estimated 0.01 ft/day (3.7 ft/yr). The slurry wall may be constructed by excavating with a clamshell, conveyor-type ditcher, or vibrated beam. We recommend the latter of these methods due to the ease of installation at this site. Slurry wall permeabilities of about 1×10^{-7} cm/sec are expected.

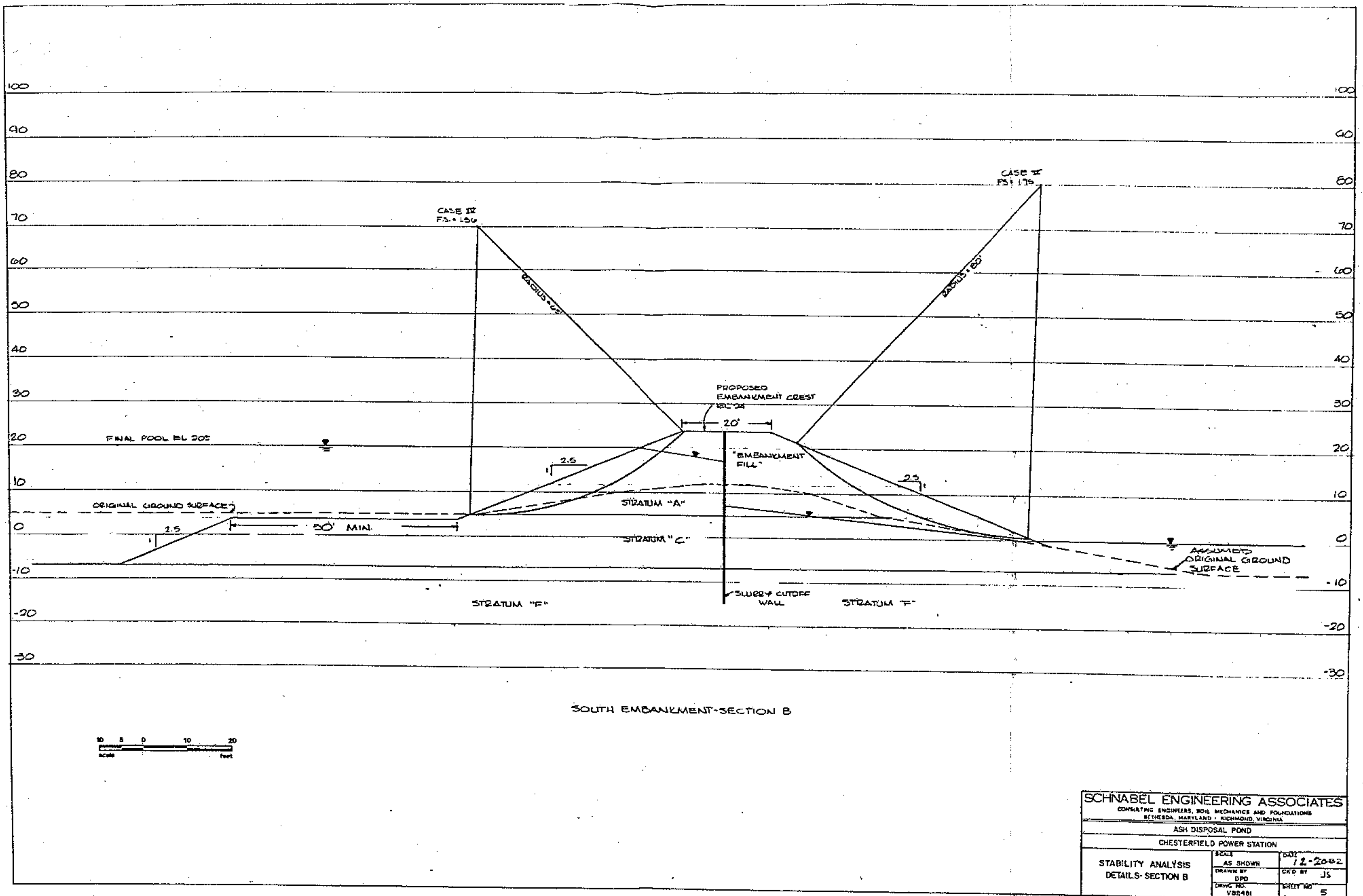
Stability Considerations

The proposed embankments are to be constructed by cutting existing ground surfaces as required, and placing compacted embankment fill to design grades. Embankment slopes of 2.5H:1V were assumed in this analysis. Two cross sections described below as Sections A and B are believed to be representative of proposed embankment configurations. These were analysed for stability.

Section A represents the north embankment, as detailed on Sheet 4. This embankment is to be constructed by grading the existing exterior slope and filling the interior slope to the design 2.5H:1V slopes. The



SCHNABEL ENGINEERING ASSOCIATES CONSULTING ENGINEERS, SOIL MECHANICS AND FOUNDATIONS BETHESDA, MARYLAND • RICHMOND, VIRGINIA	
ASH DISPOSAL POND	
CHESTERFIELD POWER STATION	
STABILITY ANALYSIS DETAILS-SECTION A	SCALE: AS SHOWN; DATE: 12-20-82; DRAWN BY: DPO; CK'D BY: JS; DWG NO.: V8248I; SHEET NO.: 4


CASE IV
F.S. = 1.56
CASE V
F.S. = 1.75
RADIUS = 65'
RADIUS = 80'
PROPOSED
EMBANKMENT CREST
EL. 24
20'
FINAL POOL EL 20.5
2.5
1
"EMBANKMENT
FILL"
ORIGINAL GROUND SURFACE
STRATUM "A"
STRATUM "C"
50' MIN.
ASSUMED
ORIGINAL GROUND
SURFACE
SLURRY CUTOFF
WALL
STRATUM "F"
STRATUM "F"
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
SOUTH EMBANKMENT-SECTION B
10 5 0 10 20
scale feet
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ASH DISPOSAL POND
CHESTERFIELD POWER STATION
STABILITY ANALYSIS
DETAILS- SECTION B
SCALE
AS SHOWN
DATE
12-2082
DRAWN BY
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CKD BY
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V82481
SHEET NO
5

crest will be 20 ft wide and is assumed to be cut to El 24 where higher river bank grades exist. The existing lake on the interior slope is to be filled to above the water level (El 4) a distance of 50 ft from the proposed toe, and then should be sloped at 2.5H:1V to the base grade. This material is assumed to be placed end dumped below the existing water level, and surface compacted with a 10 ton vibratory roller above water level. A 10 ft wave berm should be placed around the interior of the pond at normal pool for erosion protection.

Section B represents the proposed east, south, and west ash pond embankments. Construction of these embankments will require varying amounts of cut and fill to achieve the desired cross section. Crest elevation and width and slope grades are the same as for Section A. Representative details are indicated on Sheet 5. This section differs from that of Section A in that we assumed the presence of the clays of Strata D and E within the embankment.

The following soil parameters were developed for use in these analyses:

Stratum	Total Unit Weight (pcf)	Effective Strength Parameters $\bar{c}$ (psf)	Ø' (degrees)
A	125	0	32
B	100	100	0
C	130	0	35
D	110	0	20
E	110	0	27
F	140	0	40
Embankment Fill	130	0	35
Lake Fill	110	0	30
Toe Berm Fill	110	0	25

These parameters were developed based on test boring and laboratory test data.

The maximum water surface elevation for the proposed facility is El 20. Seepage control was assumed to be provided by a slurry cutoff wall installed through the center of the embankment. In this analysis, a head loss of 10 ft was assumed across the cutoff wall, resulting in assumed phreatic seepage surfaces as indicated on the representative sections. Stability analyses were performed using the Simplified Bishop method of slices.

Steady state seepage was considered in all analyses. This assumption is based on our understanding that rapid drawdown conditions are not possible in the proposed pond, and our belief that significant excess construction case pore pressures will not result since there are limited depths of additional fill required to reach design crest grade.

Stability analyses performed on the exterior slope of Section A resulted in a minimum factor of safety of 1.21. The assumed failure surface is indicated as Case I on Sheet 4. This factor of safety is less than the 1.5 value recommended by the U. S. Army Corps of Engineers, "Engineering and Design: Stability of Earth and Rock Fill Dams", EM 1110-2-1902. For steady seepage conditions, the stability of the proposed north embankment was considered marginal and required modification to meet the above criteria.

The modification will involve the constructionof an exterior toe berm on the existing river deposits located north of the proposed embankment, depicted as Case II on Sheet 4. A toe berm was considered over further flattening of slopes since fill material is readily available and the berm placed on the exterior slope will result in no loss of pond volume. The toe berm may be constructed of on site materials excavated from the pond interior. Because of the weak nature of the river deposits, it will be necessary to place this material loosely in a very thick lift, followed by surface compaction. A low strength value has, therefore, been assumed for this berm material. Top of berm elevation is assumed at El 10 and filling should be made to El 12 or 5 ft above existing ground surface to account for settlement of the underlying river deposits. The toe berm should extend horizontally 25 ft beyond the present slope, as shown on Sheet 4. This berm should also have a exterior slope of 2.5H:1V. The top of the berm should slope towards the old river channel at about a 2% grade.

Stability analyses performed on this modified section resulted in a minimum factor of safety of 1.67, meeting the above referenced criteria. The critical failure surface for this analysis is shown as Case II on Sheet 4.

A stability analysis was also performed on the proposed interior slope of Section A assuming a maximum water surface at El 20 and steady state seepage conditions. Material is assumed to remain in place below El 4 a distance of 50 ft inboard of the interior toe of slope. A factor of safety of 1.49 results, as indicated by Case III on Sheet 4. This factor of safety is considered adequate based on the above mentioned criteria.

Stability analyses performed on both the interior and exterior slopes of Section B resulted in minimum factors of safety of 1.56 and 1.75, respectively. The critical failure surfaces are shown as Cases IV and V on Sheet 5. These results also meet the Corp of Engineers requirements.

Based upon these results, we recommend that the ash pond embankments be constructed at 2.5H:1V slopes with 20 ft wide crest at El 24. Data in our files indicate that the underlying Cretaceous soils are highly preconsolidated. Based on the relatively small amounts of new embankment fill planned for this project, settlements of the embankment are expected to be very minor.

A slurry cutoff wall constructed through the embankment, extending into the very compact soils of the Patuxent Formation, Strata E and F, is recommended for controlling seepage. Should a pond liner be chosen for control of seepage, additional stability analyses should be performed. The choice of a liner over a slurry cutoff wall will result in a higher phreatic surface within the embankment, reducing embankment stability and resulting in the need for additional modifications, most notably the installation of exterior embankment toe drains.

Compacted fill for use in embankment construction should classify SC, SM, SP, or SW in accordance with the Unified Soil System, ASTM D-2487. This fill should be placed in loose lifts not exceeding 12 inches and be compacted to 95% of maximum dry density in accordance with ASTM D-698. On-site Stratum C and F soils excavated from the pond interior are expected to meet these criteria and may be used as compacted fill. The topsoil and organic matter should be stripped prior to fill placement and benching of existing slopes should be required as necessary to provide a level surface on which to compact new fill.

Inlet/Outlet Construction

Because inlet and outlet facilities require placement of pipes through the pond embankments, and because such construction requires a breach in the slurry wall, considerations pertaining to this installation are included. Based upon our experience with slurry construction, we recommend the pipes be installed after the cutoff wall, but prior to the slurry contractors departure from the site. This construction sequence would permit the contractor to pump slurry around the pipe during backfilling operations, and to reinstall the wall in the areas immediately above and

Geotechnical Engineering Study Long Term Ash Storage Pond Dike, 1996

Geotechnical Engineering Study
Long Term Ash Storage Pond Dike
Chesterfield County, Virginia

Project 963432A

April 22, 1996

Prepared for:

John Cima, P.E.
Virginia Power
5000 Dominion Boulevard
Glen Allen, Virginia 23060

Prepared by:

Schnabel Engineering Associates, Inc.
One West Cary Street
Richmond, Virginia 23220

Phone: (804) 649-7035
Fax: (804) 783-8023

COMMONWEALTH OF VIRGINIA
GILBERT T. SEESE, JR.
NO. 012625
PROFESSIONAL ENGINEER

Gilbert T. Seese, P.E.
Senior Associate

Raymond A. DeStephen, P.E.
Principal

EXECUTIVE SUMMARY

Between Stations 70+70 and 74+20 along the north dike, slope movements occurred during pond filling in 1992-93 and 1996. Movements are attributable to excess pore pressures and seepage forces associated with the increasing pond levels during dredging/filling.

We believe three different types of slope movements occurred in 1996: (1) a mud flow in the abutment at Sta 74+30, (2) a block movement of the geogrid-reinforced section from Sta 73+10 to 74+20 and, (3) a classical rotational displacement between about Sta 71+70 and Sta 73+10. These movements occurred when the pond level reached El 30 and above.

Pumping from relief wells was recommended during the 1996 dredge (which is still in progress) to arrest all three slope movements. Four relief wells were installed by Chesterfield County along the Henricus Access Road, and eight by Virginia Power along the crest of dike. To date relief well pumping has been successful in keeping the north dike embankment stable.

We considered long term remedial measures for the north dike embankment, that is for possible future dredges. We performed a back analyses for the stability under the failed conditions, and adjusted steady state strength parameters in the recent alluvial soils (Stratum B), accordingly. Future dredge conditions with pond at El 40 were analyzed with various seepage controls. Seepage controls consisted of various seepage cutoffs along the ~~vest~~ crest, a horizontal drain along the toe berm using directional drilling or one-step trenching, ~~and~~ use of existing relief wells together with placement of an interior compacted ash berm.

All of the long term remedial alternatives have various advantages and disadvantages as described herein. They all provide factors of safety of 1.3 or greater for the steady state case with pond at El 40. A slurry wall or grout curtain are the least expensive cutoffs, and are slightly higher in cost than the installation of a horizontal well. Installation of a compacted ash berm and use of existing recovery wells is expected to be the least expensive remedial measure, with most of the cost associated with construction of the ash berm.

Four slope inclinometers should be installed along the toe of the ash pond embankment as soon as possible. The inclinometers should be installed at Sta 69+50, 71+00, 72+50 and 74+00.

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LIST OF TABLES AND FIGURES

1. INTRODUCTION

The long term ash pond at the Chesterfield Power Station is filled by periodically hydraulically dredging ash at high volumes from the two year pond nearer the power plant. During the first dredging in the winter of 1992-93 and again this past winter (1996), slope movement occurred at the east end of the north dike embankment where it transitions into the natural abutment. This embankment includes a toe berm on which the Henricus Park Access Road was constructed. The road continues up a constructed reinforced slope at the abutment to the park.

1.1 Scope of Services

This project was divided into two task items. Task 1, Implementation of Short Term Remediation, includes the installation of relief wells and pumps in the area of instability, and field monitoring. Task 2, Long Term Remediation, includes the geotechnical engineering evaluation of the data recorded in Task 1, available geotechnical data, and slope stability to develop remedial alternatives for the long term stability of the ash pond dike slope. This Geotechnical Engineering Study includes:

- The evaluation of long term remedial measures and budget cost estimates for feasible alternatives.

- Discussion of construction considerations related to the implementation of the alternatives.

1.2 Site Description

The long term ash storage pond, constructed in the early 1980's, is located south of Coxendale Road and west of the James River at the Chesterfield Power Station complex in Chesterfield County, Virginia. This pond consists of a long dike enclosing an area previously quarried for sand and gravel. The ground surface in the quarry area was variable prior to being graded to El 2.5 for the ash pond. Crest grades for the constructed dike are about El 40 to 42. A toe berm with ground surface grades between about El 14 and 16 was constructed along the north dike. The toe berm includes a 3 ft wide toe drain to Sta 69+00, constructed with invert at about El +2. The toe drain runs to a deep sump where water is pumped back into the ash pond.

The north dike east abutment has a steeply eroded natural slope along the old river channel. This steep slope has continued to erode into the natural abutment. The old river channel area has silted in and is now designated as Aiken Swamp. Chesterfield County constructed a 3,600 ft two lane paved public access road from Coxendale Road to Henricus Park along the existing toe berm and up the abutment area from 1992 to 1994. A reinforced slope was constructed in the abutment area where the natural slope had previously eroded into the proposed right-of-way.

We obtained the information for this study from available topographic site plans, prepared by Resource Planners, Inc., previous geotechnical engineering reports prepared by Virginia Power and Schnabel Engineering Associates, Inc. and through our site visits.

1.3 1992-93 Dredging and Slope Movements

Hydraulic dredging of ash from a two-year pond into the long term pond was performed in the winter of 1992. Water levels in the long term ash pond were as high as about El 29 to 30 during the dredging. Chesterfield County began construction of the Henricus Park access road at that time.

In December, 1992, seepage through the north dike slope, above the Access road, was observed by Chesterfield County personnel and representatives from Virginia Power. This seepage generally occurred between stations 72+00 and 73+00. Seepage flow was clear, and estimated as less than about 20 gpm. Drainage trenches were excavated to the existing toe drain by Virginia Power Site Services so that seepage would discharge to the toe drain sump, from where it is pumped back into the ash pond, preventing unpermitted discharge into Aiken swamp.

In the general vicinity of the seepage, sloughing and cracking of both the roadway fill, and the dike slope in the roadway ditch had also occurred. Vertical displacements of up to 6 inches were observed in early December, 1992. The area of sloughing was where a 3 to 4 ft high cut was made into the dike by the County that was subsequently regraded. In late December, 1992, additional sloughing was observed in an area east of the initial slough. In this area, fill had been placed to grade the road up to the abutment. Vertical displacement of about 3 to 4 ft was observed, with no observed soil bulging at the toe of the slope. The approximate area of the 1992 escarpment is between station 71+50 and 73+50 as illustrated on Figure 1.

Sloughing of the natural abutment had gradually migrated into the park road alignment. This sloughing had created near vertical slopes about 12 to 15 ft high. The sloughing had increased due to the seepage from the ash pond through the abutment area. Seepage was observed in April and May, 1993 and had stopped by June 1993. The area of steeply eroded natural slopes is also illustrated on Figure 1.

1.4 1993 Schnabel Study

Work on the Access road was stopped between June and September, 1993. A geotechnical engineering study was performed by Schnabel Engineering to provide recommendations for reconstruction of the road along the transition section from the toe berm to the natural abutment. The recommendations included limiting grade changes to less than 2 ft during reconstruction of the road on the toe berm. Along the transition section from the toe berm to the abutment, a subdrain was required under the road. Lastly, geogrid-reinforced slope was recommended to replace the natural abutment area that had sloughed due to seepage from the pond.

1.5 1994 Road Re-Construction

In June 1994, work commenced in the area east of Station 69+00 and continued through the end of October, 1994. The water level in the pond was believed to be between about El 15 to 20 at the time this work proceeded.

A drainage blanket and subdrain were installed under the road from about Sta 69+00 to Sta 73+00, and the reinforced slope was completed from about Sta 72+50 to Sta 74+20 as shown on Figure 2. No additional sloughing or slope movement was observed during the road construction.

Following completion of the road reconstruction, a slope inclinometer was installed below the failure escarpment, and control readings were obtained in October, 1994 as well as October through November, 1995. The slope inclinometer data is discussed in Section 2.5

1.6 1996 Dredging and Slope Movements

Dredging of ash into the long term storage pond from the 2-year pond began in January 1996. On January 22, personnel from Chesterfield County Parks and Recreation notified Schnabel that a crack approximately 135 ft long and 1 to 2 inches wide had formed down the centerline of the road, west of about Sta 74+20. This crack formed just upslope of the geogrid-reinforced embankment section and is shown approximately on Figure 1. Vertical displacement along the crack was about 2 inches.

By January 25, the crack had lengthened to about 150 ft and settlement had increased to about 6 inches. The crack was filled with asphalt and covered with plastic. Snow and subfreezing temperatures occurred for the next two weeks. The water level in the pond had risen to about El 30 on January 22 and to El 31 by January 25. Slope inclinometer readings from January 22 to 25, 1996 indicated horizonal displacements of about 0.8 to 2.3 inches at 19 to 30 ft below ground surface (about El -10 to -20). Schnabel determined that slope displacements were due to high excess pore pressures within the slope caused by seepage from the ash pond, and to a lesser extent, a high water level in Aiken swamp. We recommended relief wells to control the phreatic surface within the slope. Four relief wells (RW-1 through RW-4) and two pumps were installed on January 26 and 27 as shown on Figure 1. This work was authorized by Chesterfield County. The night of January 31, the pump discharge lines froze and service of the pumps was lost. Significant movement of the slope was observed on February 1 during the installation of two additional pumps. The crack width increased to at least 12 inches and settlement of at least 18 inches was observed. Movements of 2.3 to 3.7 inches were recorded between El -10 and -20 in the slope inclinometer casing from January 25 through February 1. Two additional pumps were installed on February 1 and continued in service until February 8. Snow and ice had covered the road through February 7. Between February 1 and February 8, no significant movement was observed in the slope either in the length, width or settlement of the crack. The water level in the pond was continuing to rise and was at about El 34.5 on February 8.

During our site visit on February 8, we observed that the valves on the middle two pumps (RW-2 and RW-3) had been turned off. Again, we observed significant movement of the crack, widening to 24 to 26 inches, and settlement, increasing to about 24 to 46 inches. From the existing crack, an escarpment appeared westerly another 100 to 150 ft to about Sta 70+70. This escarpment crack continued on the dike above and west of the road as shown in Figure 1. In addition, seepage was observed upslope of the road (about El 26) near relief well RW-3. On February 11, a mud flow occurred at the steep slope of the natural abutment near Sta 74+30. This occurred at about El 13.

Eight additional relief wells (RW-5 through RW-12) were installed between February 15 and 19 at the approximate locations shown on Figure 1. These additional wells were authorized by Virginia Power following a meeting on February 12, 1996. In addition, Virginia Power began recirculating dredge water and discharging from the pond outlet structure to maintain or decrease the ash pond water level.

Since the installation of the additional relief wells and continued service of all 12 wells on February 19, we observed little to no additional movement of the distressed slope through mid-March. Sloughing at the area of the mud flow also stabilized by mid-March. However, seepage above the road behind RW-3, and from the toe of the reinforced slope, continues to flow.

Sometime prior to April 8, additional movement occurred. The escarpment extended westward and up the embankment slope to just below well RW-5 as shown on Figure 1.

Virginia Power is continuing to dredge ash into the pond. They are also pumping dredged water back to the two-year pond and discharging water from the outlet structure to the river. However, during this dredge, the pond level could rise again, possibly to a level above the previously recorded high, about El 35, as this is a function of the level the ash attains within the pond. If the current dredge schedule is maintained, dredging operations could be complete as early as mid-July.

Table 1 provides a summary of the crack development for the Henricus Road slope from January 22 through February 11. After February 11, monitoring points located on the embankment dike and the road slopes were established. Table 2 provides a summary of the data recorded from these points. The approximate locations of the monitoring points are indicated on Figure 1.

TABLE 1 HENRICUS ROAD SLOPE MOVEMENTS (JAN. 22 THROUGH FEB. 11)

DATE	CRACK DATA			SLOPE INCLINOMETER (SI-1) DEFLECTION (IN)	WATER LEVELS (ELEV)		
	TOTAL LENGTH (FT)	MAXIMUM WIDTH (IN)	MAXIMUM SETTLEMENT (IN)		POND ELEV	TEST WELL TW-B1	SI-1[2]
1/22	135	1-2	2	0.85	EL 30±	-	10.7+1.9'±
1/23	140+	1-2	4	1.15	EL 30±	-	10.7
1/24	140+	1-2	5	1.72	EL 30±	-	-
1/25	150+	3+	6	2.28	EL 31±	24.2	9.3
1/26	150+	CRACK COVERED	NO OBSERVATION	-	EL 31±	-	9.4
1/27	150+	CRACK COVERED	NO OBSERVATION	-	EL 31±	-	-
1/28	150+	CRACK COVERED	NO OBSERVATION	3.05	EL 31±	-	9.48
1/29	150	CRACK COVERED	NO OBSERVATION	2.96	EL 31.5	-	9.2
1/30	150	CRACK COVERED	NO OBSERVATION	2.56	EL 32	24.9	9.2
1/31	150'	CRACK COVERED	NO OBSERVATION	-	EL 32.2	-	9.1
2/1	PUMP LOST OVERNIGHT	12+	18	3.7	EL 32.9	25.8	9.1
2/2-2/6	ICE ON SLOPES AND ROAD	-	NO READINGS	-	EL 34	27.0	-
2/7	SNOW AND ICE STILL ON ROAD/SLOPES	-	NO SIGNIFICANT MOVEMENT NOTED	-	-	-	-
2/8	200-220[4] RW-2 AND RW-3 TURNED OFF BETWEEN 2/7 AND 2/8	24-36	36	-	EL 34.5	27.4	9.1
2/9	240+[4]	24-36	24 TO 46	-	-	-	-
2/10	240+[4]	24-36	24 TO 46	-	EL 34.5	27.6	9.1
2/11	240+[4]	24-36	24 TO 46	-	EL 34.9	27.9	9.1

1. GROUND SURFACE AT TOP OF DIKE EL 41.5
2. GROUND SURFACE AT SLOPE INCLINOMETER EL 8.8
3. SEE TABLES 2 AND 3 FOR DATA AFTER FEB. 11
4. INCLUDES ESCARPMENT ON SLOPE ABOVE ROAD FROM ABOUT STA 72 TO 73

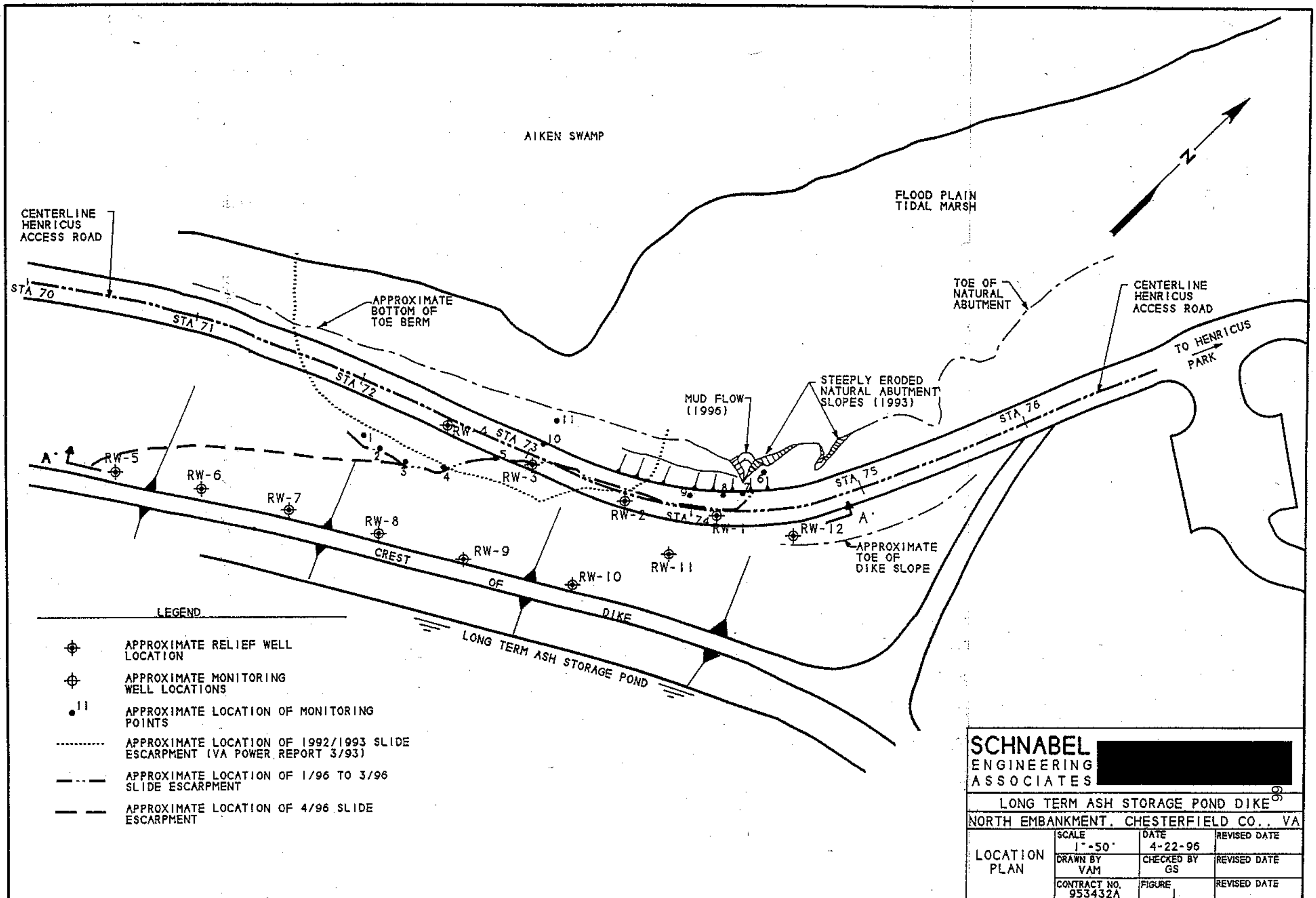
953432A

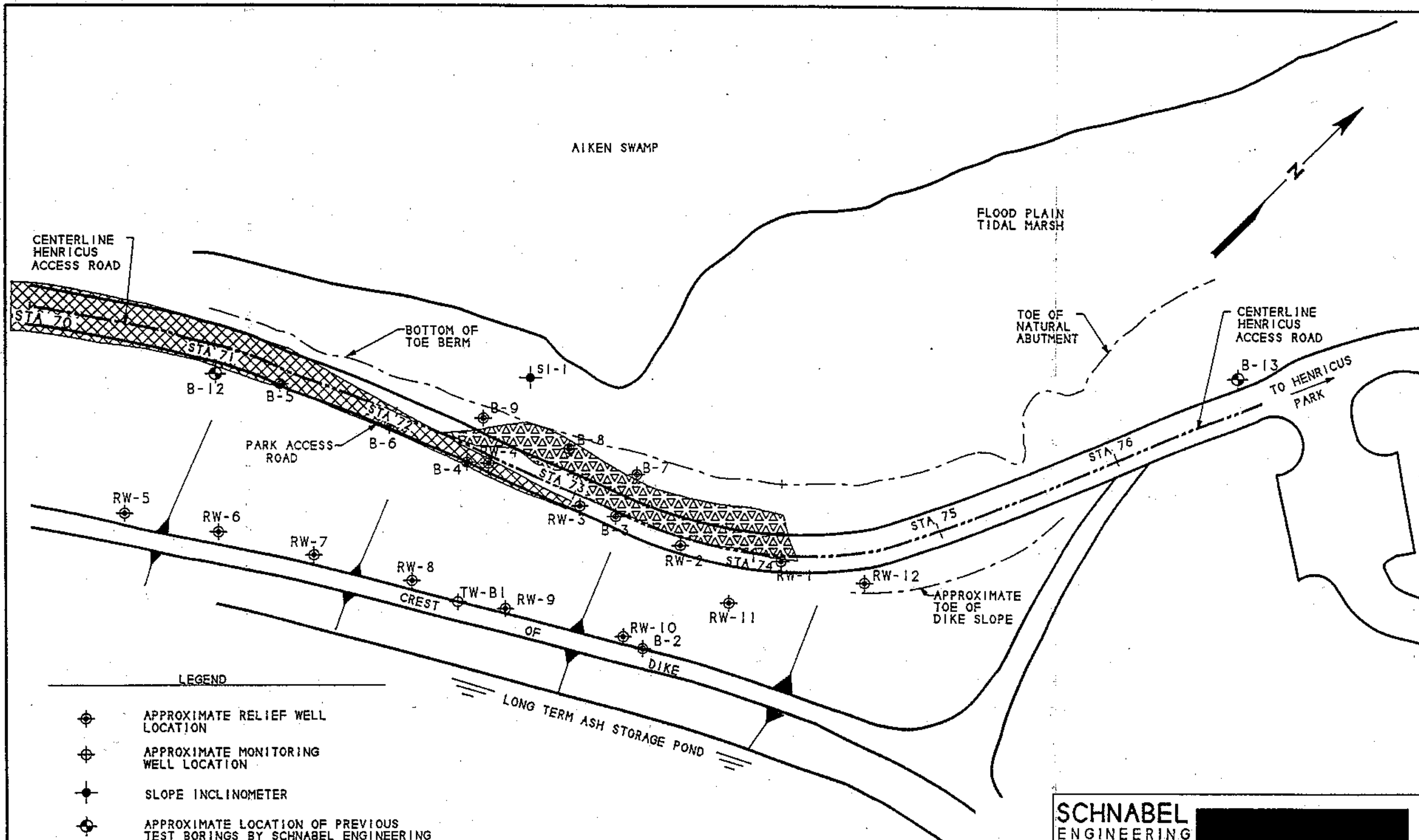
TABLE 2 - MONITORING POINT DATA

DATE	2/12 (INITIAL)	2/13	2/14	2/15	2/20	2/22	2/27	3/1	3/4	3/6	3/8	3/12	3/14	3/15	3/29
POINT NO.	Horizontal/vertical readings (ft)														
1	.02/0.4	-	-	-	-	-	-	-	-	-	-	-	-	-	-
2	.03/0.68	0.04/0.7	0.02/0.75	0.03/0.7	*.04/0.62	-/0.64	-/0.65	-/0.63	-/.61	-/0.64	-/0.62	-/.69	-/0.65	-/0.64	-/0.67
3	0.2/0.46	0.2/0.48	0.13/0.48	0.2/0.5	0.26/0.49	0.3/0.5	0.25/0.46	0.26/0.5	.3/.5	.3/.5	0.29/.5	.28/.5	0.5*/0.58	.5/.53	.5/.65
4	0.22/0.67	0.2/0.65	0.2/0.68	0.23/0.68	0.29/0.60	0.29/0.6	0.29/0.59	0.28/0.6	.28/.59	.28/.58	.28/.58	.25/.6	.75*/.68*	.74/.70	-/.7
5	0.50/0.75	.42/.67	0.5/0.68	0.5/0.64	0.56/0.62	0.49/0.62	0.48/0.60	.48/.64	0.4/0.6	-/.62	0.56/0.60	.56/.60	.58/.62	.6/.62	.6/.58
6		-/2.0	-/2.0	-/1.98	-	-/1.96	-/1.98	-/1.98	-/1.98	-/1.96	-/1.98	-/1.98	-/1.96	-/1.96	-/2.0
7	-	-	-	.93/1.92	-	0.9 to 2.0/-	1.0 to 2.0/-	1.0 to 32/-	1.0 to 1.0/-	1.0 to 2.0/-	1.0 to 2.0/-	1.0 to 2.2/-	2.0/-	2.0/-	1.9/-
8	-	-	- 7	-/3.83	-/3.66	3.0/-	4.5-6.2/-	3.5 to 4.5/-	3.4 to 4.4/-	3.75 to 4.4/-	4.5/-	4.5/-	4.45/-	4.4/-	4.4/-
9	-	-	-		-/*2.04	-/1.95	-/2.3	-/2.23	-/2.0	2.25V	-/2.4	-/2.4	-/2.3	-/2.28	-/1.95
10	-	-	-	-	3.2/2.98	-/2.9	-/3.2	-/3.25	-/3.18	3.92V	-/3.1	-/3.24	-/3.15	-/3.15	-/3.1
11	-	-	-	-	-	-	-	-	-/1.2	-/1.10V	-/1.2	-/1.2	-/1.0	-/1.1	-/1.0

*NEW BENCHMARK ESTABLISHED

1. SEE FIGURE 1 FOR LOCATIONS.
2. READINGS ARE MEASURED DEFLECTION AT THE POINT LOCATIONS.


AIKEN SWAMP
FLOOD PLAIN TIDAL MARSH
N
CENTERLINE HENRICUS ACCESS ROAD
STA 70
STA 71
STA 72
STA 73
STA 74
STA 75
STA 76
APPROXIMATE BOTTOM OF TOE BERM
TOE OF NATURAL ABUTMENT
CENTERLINE HENRICUS ACCESS ROAD
TO HENRICUS PARK
MUD FLOW (1996)
STEEPLY ERODED NATURAL ABUTMENT SLOPES (1993)
APPROXIMATE TOE OF DIKE SLOPE
A'
A
RW-1
RW-2
RW-3
RW-4
RW-5
RW-6
RW-7
RW-8
RW-9
RW-10
RW-11
RW-12
1
2
3
4
5
6
7
8
9
10
11
CREST OF DIKE
LONG TERM ASH STORAGE POND
LEGEND
APPROXIMATE RELIEF WELL LOCATION
APPROXIMATE MONITORING WELL LOCATIONS
APPROXIMATE LOCATION OF MONITORING POINTS
APPROXIMATE LOCATION OF 1992/1993 SLIDE ESCARPMENT (VA POWER REPORT 3/93)
APPROXIMATE LOCATION OF 1/96 TO 3/96 SLIDE ESCARPMENT
APPROXIMATE LOCATION OF 4/96 SLIDE ESCARPMENT
SCHNABEL ENGINEERING ASSOCIATES
[redacted]
66
LONG TERM ASH STORAGE POND DIKE
NORTH EMBANKMENT, CHESTERFIELD CO., VA
LOCATION PLAN
SCALE 1"=50'
DATE 4-22-96
REVISED DATE
DRAWN BY VAM
CHECKED BY GS
REVISED DATE
CONTRACT NO. 953432A
FIGURE 1
REVISED DATE



LEGEND

- APPROXIMATE RELIEF WELL LOCATION
- APPROXIMATE MONITORING WELL LOCATION
- SLOPE INCLINOMETER
- APPROXIMATE LOCATION OF PREVIOUS TEST BORINGS BY SCHNABEL ENGINEERING
- APPROXIMATE LOCATION OF PREVIOUS TEST BORINGS BY FROEHLING AND ROBERTSON, INC.
- DRAINAGE BLANKET/SUBDRAIN BENEATH PAVEMENT: CONNECTED TO EXISTING TOE DRAIN AT APPROXIMATELY STA 69+00.
- APPROXIMATE LOCATION OF GEOGRID REINFORCED SLOPE

SCHNABEL
ENGINEERING
ASSOCIATES
[redacted]

LONG TERM ASH STORAGE POND DIKE
NORTH EMBANKMENT, CHESTERFIELD CO., VA

DRAINAGE BLANKET/ SUBDRAIN AND GEOGRID SLOPE LOCATION PLAN	SCALE 1"=50'	DATE 4-22-96	REVISED DATE
	DRAWN BY VAM	CHECKED BY GS	REVISED DATE
	CONTRACT NO. 953432A	FIGURE 2	REVISED DATE 953432A

2. DATA COLLECTION AND ANALYSIS

2.1 Geology

The lowest geologic unit of interest in the ash pond area consists of Cretaceous age sediments occurring below about El 5. To the north of the ash pond, and into Aiken swamp, these sediments are believed to have been eroded by the old river channel. The Cretaceous age sediments consist of sand, silt, clay and gravel materials deposited in a variety of environments ranging from deltaic, beach, and estuarine to shallow marine. These sediments are highly preconsolidated with respect to the present overburden, accounting for the very stiff consistency of the clays and very compact densities of the gravelly sands. The Cretaceous sediments are overlain by Pleistocene age sediments. The Pleistocene age sediments are alluvial soils typically consisting of a poorly-sorted mixture of sand, clay and gravel exhibiting moderate strength and compressibility. These soils generally contain greater amounts of gravel with depth. Recent geologic age James River sediments typically overly the Pleistocene age sediments. The Recent James River sediments consist of sands, silts and clays with organic matter and exhibit relatively lower strengths and higher compressibilities.

In the immediate vicinity of the ash pond dike and north abutment, fill soils consisting of mixtures of sands, clays and gravels generally overlie the Pleistocene and Recent sediments. These fill soils were placed during sand and gravel excavation to maintain haul roads, for construction of the existing ash pond dike and toe berm, and for construction of the Henricus Park Road.

2.2 Relief Wells and Test Borings

Four relief wells were installed for Chesterfield County Parks and eight relief wells were installed for Virginia Power in January and February, 1996, respectively. These wells were installed under our observation and logs for the wells were recorded based on visual observations of the soils cuttings. Additional test borings were not performed for this study. However, previous Borings B-4 and B-5, and B-12 and B-13 performed under our observation during 1982 and 1988 studies, respectively, were used as reference for this study. In addition, Borings TW-B1, and B-2 through B-9 drilled at this site for the March 1993 Virginia Power study were also used. Logs for the relief wells and test borings are included in Appendix A. Approximate relief well and test boring locations are shown on Figure 2.

2.3 Generalized Subsurface Stratigraphy

We have characterized the following generalized subsurface soil stratigraphy for the ash pond dike and north abutment area based on available test boring and relief well data presented in Appendix A. Profiles along the dike crest and two cross-sections through the dike are illustrated on Figures 3, 4 and 5.

Stratum A: Stratum A consists of generally loose to compact density silty sand and clayey sand DIKE EMBANKMENT FILL containing varying amounts of gravel as well as lean clay lenses and layers. This occurs from the top of dike to depths of about 2.0 to 17.0 ft.

Stratum B: The Recent James River sediments are identified as Stratum B. These soils consist of very soft consistency SANDY SILT (ML) or loose density CLAYEY SAND (SC), SILTY SAND (SM) and POORLY GRADED SAND (SP) encountered below Stratum A to depths of about 14 to 19 ft.

Stratum C: Pleistocene age terrace deposits have been identified as Stratum C. The upper portion of this stratum consists of loose to very compact density SILTY SAND (SM), and CLAYEY SAND (SC) containing clay layers. The lower portion consists of POORLY GRADED SAND (SP) containing varying amounts of gravel and clay layers. These soils generally become more gravelly with depth and are therefore expected to exhibit higher permeabilities. Stratum C was encountered below Strata A and B to depths of about 22 to 52 ft.

Stratum D: These fine-grained Pleistocene geologic age soils were only encountered at the abutment in Boring B-13. These soils, encountered below Stratum A to a depth of 6 ft, consist of hard consistency SANDY LEAN CLAY (CL).

Stratum E and F: The Cretaceous age sediments are identified by these strata. Stratum E consists of generally firm to very compact density SILTY, CLAYEY and POORLY GRADED SANDS (SM, SC, SP) with varying amounts of gravel. Stratum F consists of generally stiff consistency SILT (ML) and FAT and LEAN CLAY (CH, CL) with varying amounts of sand and gravel. These strata were encountered below Stratum C to 60 ft, the maximum depth of penetration.

2.4 Water Levels

Water level readings which were obtained in the borings and relief wells during and after completion are noted on the logs. These levels show estimated hydrostatic water levels in the embankment at the time the borings and relief wells were drilled. A summary of the water level readings from January 22 through March 29, 1996 for the relief wells, slope inclinometer and monitoring well are provided on Tables 3 and 3A. Their locations are shown on Figure 1 at the end of Section 1.

Prior to installing pumps in Relief Wells RW-1 through RW-4 (January 25 to 27) along the Henricus Access road water levels in the wells were between El 19.7 and 22.3. Water levels representing the phreatic surface at the dike crest as recorded in RW-5 through RW-12 were El 25.6 to 29.4. The water level in the ash pond was between El 31 and 33 when these readings were recorded.

Throughout the pumping of relief wells, F&R-TW-B1 was used as a monitoring point for changes in the phreatic surface. As indicated by Tables 3 and 3A, even though pumping from relief wells has continued from February 9 through March the water level in F&R-B1 continued to vary in a subdued manner with the water level in the pond. The highest water level was recorded on February 14, 1996 at about El 28.1 in F&R-TW-B1 as the pond water level was at El 35. After this date, Virginia Power began pumping water back to the two-year pond and releasing water through the outlet structure to the river. The water level in F&R-TW-B1 has dropped to about El 25.6 and the pond water level has dropped to about the same elevation. These water levels were recorded on March 29, 1996.

Prior to the installation of relief wells, the phreatic surface was very high within the embankment. Excess pore pressure was demonstrated by the water level recorded in the slope inclinometer, which was about 1.9 ft above the ground surface at the toe of slope. The water level in the slope inclinometer reduced to about the ground surface grade following pumping from relief wells. This was similar for four shallow piezometers, installed at the toe of slope. All of the readings in the piezometers indicated water levels at or below the ground surface, indicating no excess seepage pressures in the shallow soils.

The phreatic surface through the embankment as measured in the slope inclinometer and relief wells (without pumping) is shown on Figure 6. The effects of relief well pumping

TABLE 3 - 1996 GROUND WATER ELEVATIONS

EVENT	JAN 22 CRACK FIRST NOTICED				JAN 26 FIRST RELIEF WELLS INSTALLED	JAN 27 RW-2 & -3 OFF 24 HRS. MAJOR CRACK MOVEMENT	WELL FUNCTIONING MOVEMENT STABILIZED						RW-2 AND RW-3 OFF; MAJOR MOVEMENTS OCCUR AT GEOGRID	NEW SCARP FORMED WEST OF GEOGRIDS				
LOCATION	**1/22**	**1/23**	**1/24**	**1/25**	**1/26**	**1/27**	**1/28**	**1/29**	**1/30**	**1/31**	**2/1**	**2/6**	**2/8**	**2/9**	**2/10**	**2/11**	**2/13**	**2/14**
POND	30	30	30	31	31	31	31	31.5	32	32.2	32.9	34	34.5	34.4	34.5	34.9	-	-
TW-B1	-	-	-	24.2	-	-	-	-	24.9	-	25.8	27	27.4	27.4	27.6	27.9	28.0	28.1
S-1 SLOPE INDICATOR	10.7 (+1.9')	10.7		9.3	9.4		9.5	9.2	9.2	9.1	9.1	9.1	9.1	9.1	9.1	9.1	9.1	9.1
RW-1 (-8)	-	-	-	-	20.1[1]	20.4[1]	21.1[1]	-5	-5	-5	-5	-5	3.6	5.7	-	7.0	-5	
RW-2 (-8)	-	-	-	-	22.3[1]	-	-5	-5	-5	- 5	-5	-5	24.3[1]	-2.5	-	-4	-5	
RW-3 (-8)	-	-	-	-	22.1[1]	21.7[1]	-5	-5	-5	-5	-5	-5	18.5[1]	-5.2	-	-5.1	-5	
RW-4 (-4.3)	-	-	-	-		14.4[1]	19.7[1]	-0.4	-0.4	-0.4	-0.4	-0.4	3.2	1.3	-	-0.4	0.4	
RW-5																		
RW-6																		
RW-7																		
RW-8																		
RW-9																		
RW-10																		
RW-11																		
RW-12																		

NOTES:

1. INITIAL READINGS ON READINGS WITH PUMPS TURNED OFF
2. PUMPS MALFUNCTED, SYDNOR HYDRODYNAMICS RESET PUMPS
3. PMP IS NOT WORKING

953432A

TABLE 3A - 1996 GROUND WATER ELEVATIONS (continued)

EVENT	VIRGINIA POWER BEGAN LOWING POND LEVEL; NEW RELIEF WELLS RW-5 THRU RW-12 RUNNING															
LOCATION	2/15	2/16	2/17	2/18	2/19	2/20	2/22	2/25	2/27	3/1	3/4	3/6	3/8	3/12	3/14	3/15	3/29
POND	-	33.6	-	-	32.9	32.9	33.05	32.28	29.91	29.64	30.52	31.3	30.8	29.3	28.6	28.4	24.9
TW-B1			27.8	27.1	27.0	27.0	26.95	26.75	26.49	26.31	26.54	26.84	26.68	26.7	26.6	26.5	25.6
S-1 SLOPE INDICATOR	9.1	9.1	9.1	9.1	9.1	8.98	9.22	9.13	9.08	9.04	8.96	9.00	8.89	-	8.9	8.9	9.0
RW-1 (-8)	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5
RW-2 (-8)	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5
RW-3 (-8)	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5	-5
RW-4 (-4.3)	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
RW-5			29.4[1]	-2.7	-2.7	-2.7	-2.7	-2.7	-2.7	-2.7	-2.7	-2.7	-2.7	-2.7	-2.7	-2.7	-2.7
RW-6			29.2[1]	-3.2	-3.2	-3.2	-3.2	-3.2	-3.2	-3.2	-3.2	-3.2	-3.2	-3.2	-3.2	-3.2	-3.2
RW-7			29.0[1]	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6	-3.6
RW-8			28.8[1]	26.8[1]	-3.5	-3.5	-3.5	-3.5	-3.5	-3.5	-3.5	-3.5	-3.5	25.5[2]	-3.5	-3.5	24.2[3]
RW-9			28.5[1]	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	10.8[2]	-5.6	-5.6	-5.6
RW-10			-	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6	-5.6
RW-11			26.1[1]	13.0	9.5	9.5	9.5	9.5	9.5	9.5	9.5	9.5	9.5	9.5	9.5	9.5	9.5
RW-12			25.6[1]	-3.7	-3.7	-3.7	-3.7	-3.7	-3.7	-3.7	-3.7	-3.7	-3.7	-3.7	-3.7	-3.7	-3.7

NOTES:

1. INITIAL READINGS ON READINGS WITH PUMPS TURNED OFF
2. PUMPS MALFUNCTED, SYDNOR HYDRODYNAMICS RESET PUMPS
3. PMP IS NOT WORKING

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are shown by Figure 7. The "drawn down" phreatic surface could only be estimated between pumping wells.

Based on the lack of draw down in well F&R-TW-B1, located between relief well RW-8 and -9, the radius of influence of the wells was assumed to be fairly small. This is reflected in Figure 7.

2.5 Slope Inclinometer

The slope inclinometer installed in August, 1994, indicated no movement in 1994 and 1995. When the crack in the road was reported on January 22, 1996, slope inclinometer readings were obtained that day and daily for the next two weeks.

Movements indicated by the inclinometer were generally in the lower 20 to 30 ft, from about El -10 to El -20. Horizontal movements steadily increased from about 1.0 inch to about 3.7 inches on February 1, 1996. The movement in the last 2 ft of the inclinometer casing restricted the travel of the inclinometer probe below the 29 ft depth after the February 1, 1996 reading. Due to the excessive deflection in the casing at the bottom of the slope inclinometer, movements recorded after February 1 may not represent actual deflections of the slope. Graphs of selected slope inclinometer readings up to February 1 are presented on Figure 8 at the end of this section.

2.6 Laboratory Testing and Soil Parameters

We conducted tests on selected samples of cuttings obtained in the relief wells, results are included in Appendix B. Previous testing of samples obtained in the earlier test borings aided in the classification of soils encountered, and provided data for use in the stability evaluations. The results of previous testing were provided in the earlier reports referenced in Section 2.2.

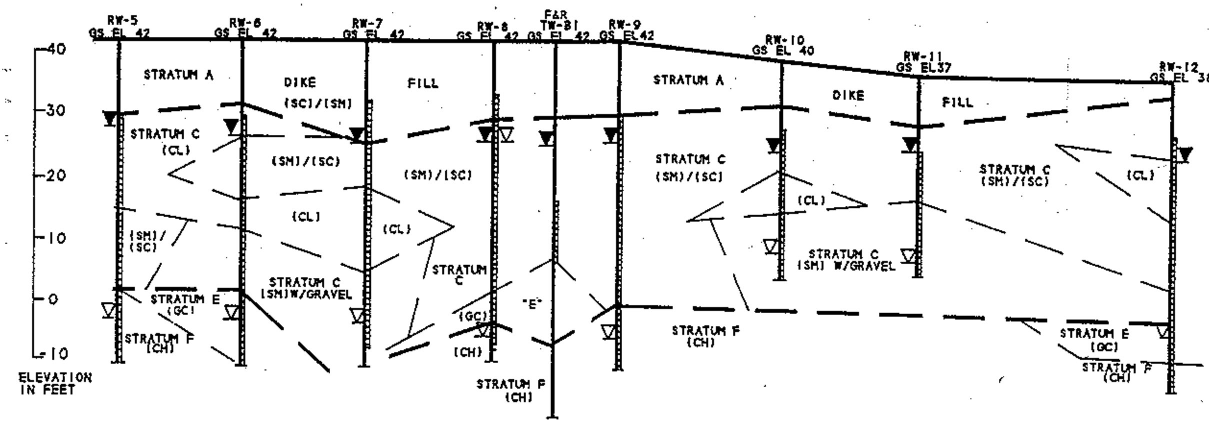
Gradation tests were performed on one sample of Stratum A and two samples of Stratum C soils obtained from auger cuttings. The sample tested from Stratum A classified as a clayey sand (SC) with about 41 percent material by weight finer than the No. 200 sieve. This sample is believed to represent the upper Pleistocene sands, with estimated permeability of 10^{-5} to 10^{-3} cm/sec. The two samples tested from Stratum C classified as silty sands (SM) with between 28.7 and 32.6 percent material by weight finer than the No. 200 sieve. Low to high permeabilities (10^{-5} to 10^{-1} cm/sec), are estimated for this stratum, and depend on the gravel and silt content.

The classification testing performed for this study, and strength testing performed for previous studies, were used for both stability analyses and modeling of flow through the embankment.

The soil parameters used for stability, assuming the steady state seepage case, are tabulated below. The strength parameters were generally the same as those we developed and used in previous analyses. However, based on a back analysis of the failed section of slope, we reduced, the strength parameters for the Stratum B, recent alluvium.

Table 4 - Soil Parameters

Stratum	Description	Unit Weights (pcf)		Cohesion (psf)	Friction Angle	Coefficient of Permeability (cm/sec)	
		Total	Saturated			Horizontal	Vertical
A	Embankment Fill	125	130	0	32	2×10^{-4}	1×10^{-4}
B	Alluvium	100	110	0	23	5×10^{-6}	5×10^{-6}
C	Upper Pleistocene Lower Pleistocene	130	135	0	35	5×10^{-3} 1×10^{-2}	5×10^{-4} 5×10^{-3}
E/F	Cretaceous	140	140	0	40	1×10^{-6}	5×10^{-7}
-	Compacted Ash	-	-	-	-	5×10^{-5}	1×10^{-5}



PROFILE A-A'

HORIZONTAL SCALE 0 50' 100'

VERTICAL SCALE 0 20' 40'

LEGEND

▼ WATER LEVEL IN WELL BEFORE PUMPS INSTALLED (24 HR READINGS)

▽ WATER LEVELS IN WELLS WITH PUMPING

SCREENED INTERVAL

NOTE: POND LEVEL BETWEEN EL 31 AND 33 AT TIME OF READINGS

STRATA GEOLOGY CLASSIFICATIONS

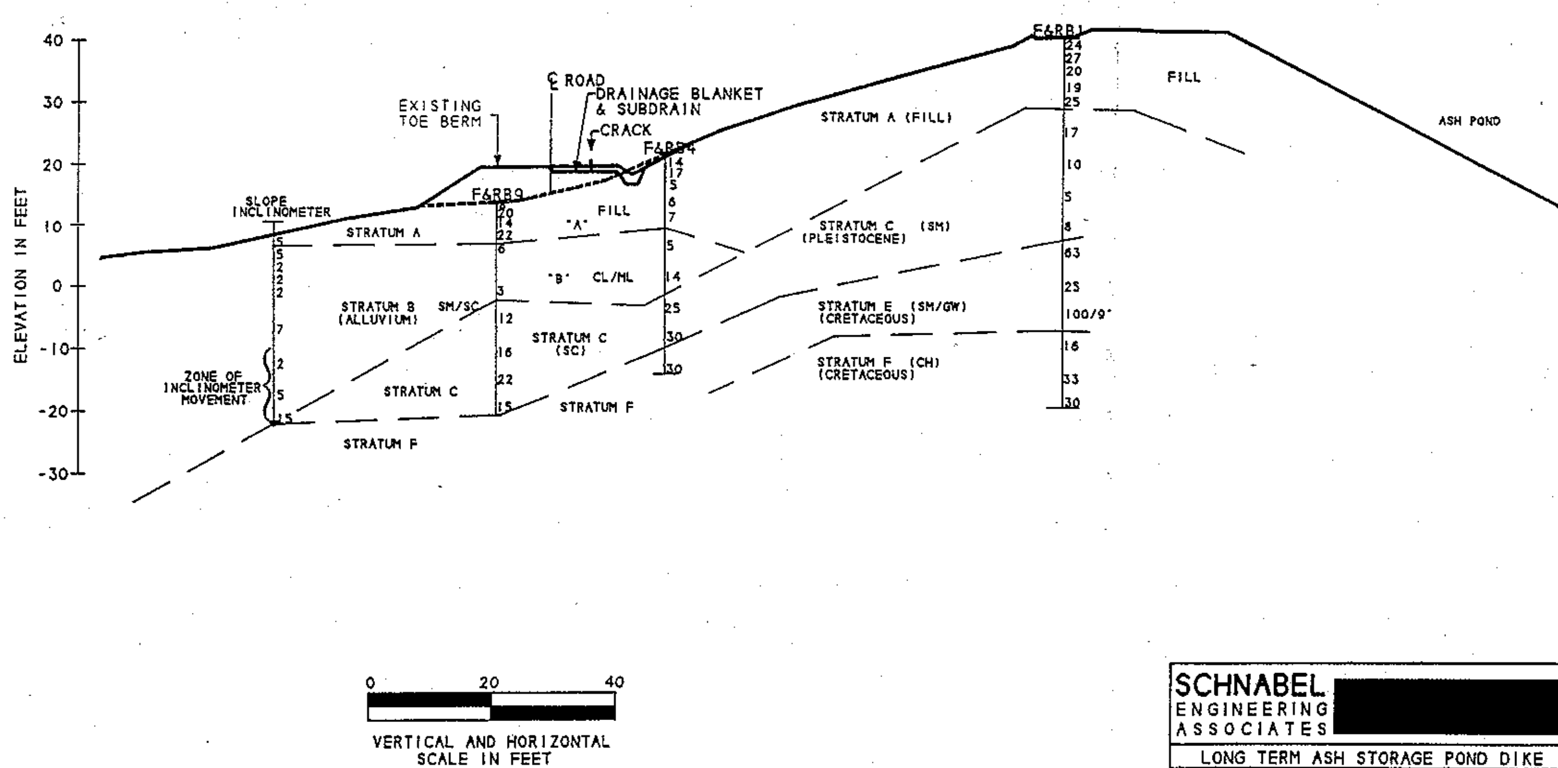
A	DIKE FILL	SC/SM
C	PLEISTOCENE	SC/SM. CL
E	CRETACEOUS	GC
F	CRETACEOUS	CH

SCHNABEL ENGINEERING ASSOCIATES [redacted]

LONG TERM ASH STORAGE POND DIKE 75

NORTH EMBANKMENT, CHESTERFIELD CO., VA

PROFILE ALONG NORTH EMBANKMENT CREST	SCALE AS SHOWN	DATE 4-22-96	REVISED DATE
	DRAWN BY VAM	CHECKED BY GS	REVISED DATE
	CONTRACT NO. 953432A	FIGURE 3	REVISED DATE

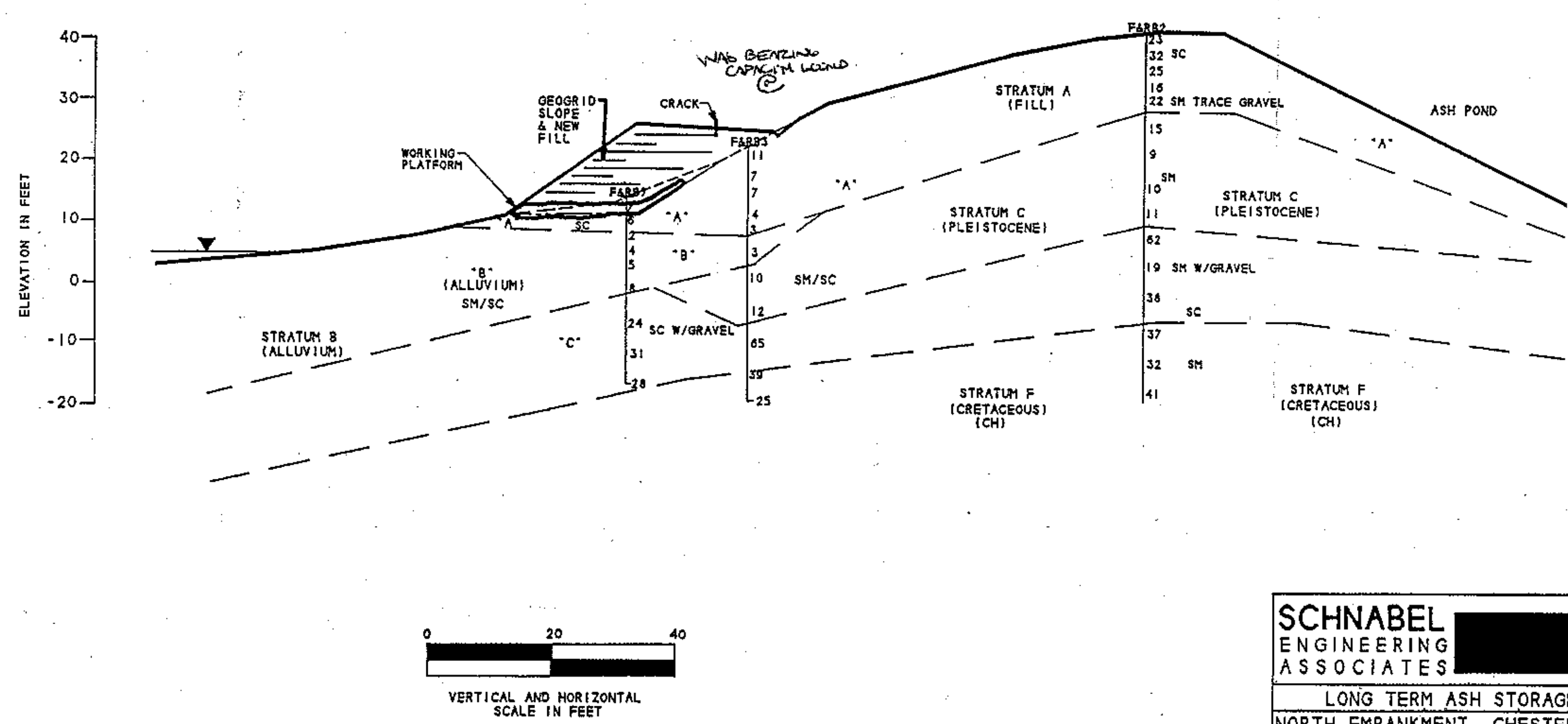


SCHNABEL ENGINEERING ASSOCIATES [redacted]

LONG TERM ASH STORAGE POND DIKE

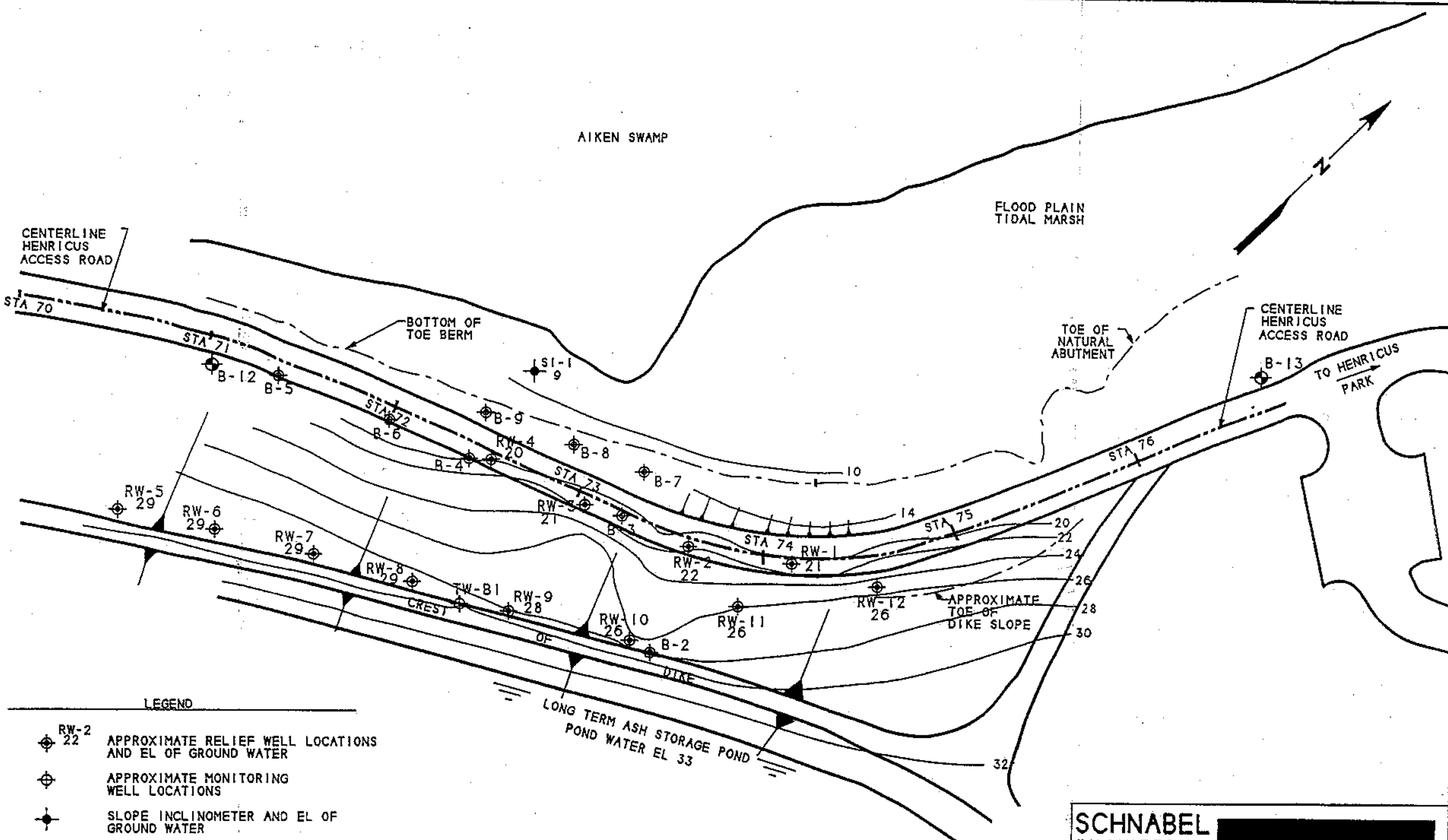
NORTH EMBANKMENT, CHESTERFIELD CO., VA

CROSS SECTION STA 72+50	SCALE 1"-20'	DATE 4-22-96	REVISED DATE
	DRAWN BY VAM	CHECKED BY GS	REVISED DATE
	CONTRACT NO. 953432A	FIGURE 4	REVISED DATE



SCHNABEL ENGINEERING ASSOCIATES [redacted]

LONG TERM ASH STORAGE POND DIKE 77			
NORTH EMBANKMENT, CHESTERFIELD CO., VA			
CROSS SECTION	SCALE 1" = 20'	DATE 4-22-96	REVISED DATE
	DRAWN BY VAM	CHECKED BY GS	REVISED DATE
STA 73+50	CONTRACT NO. 953432A	FIGURE 5	REVISED DATE



LEGEND

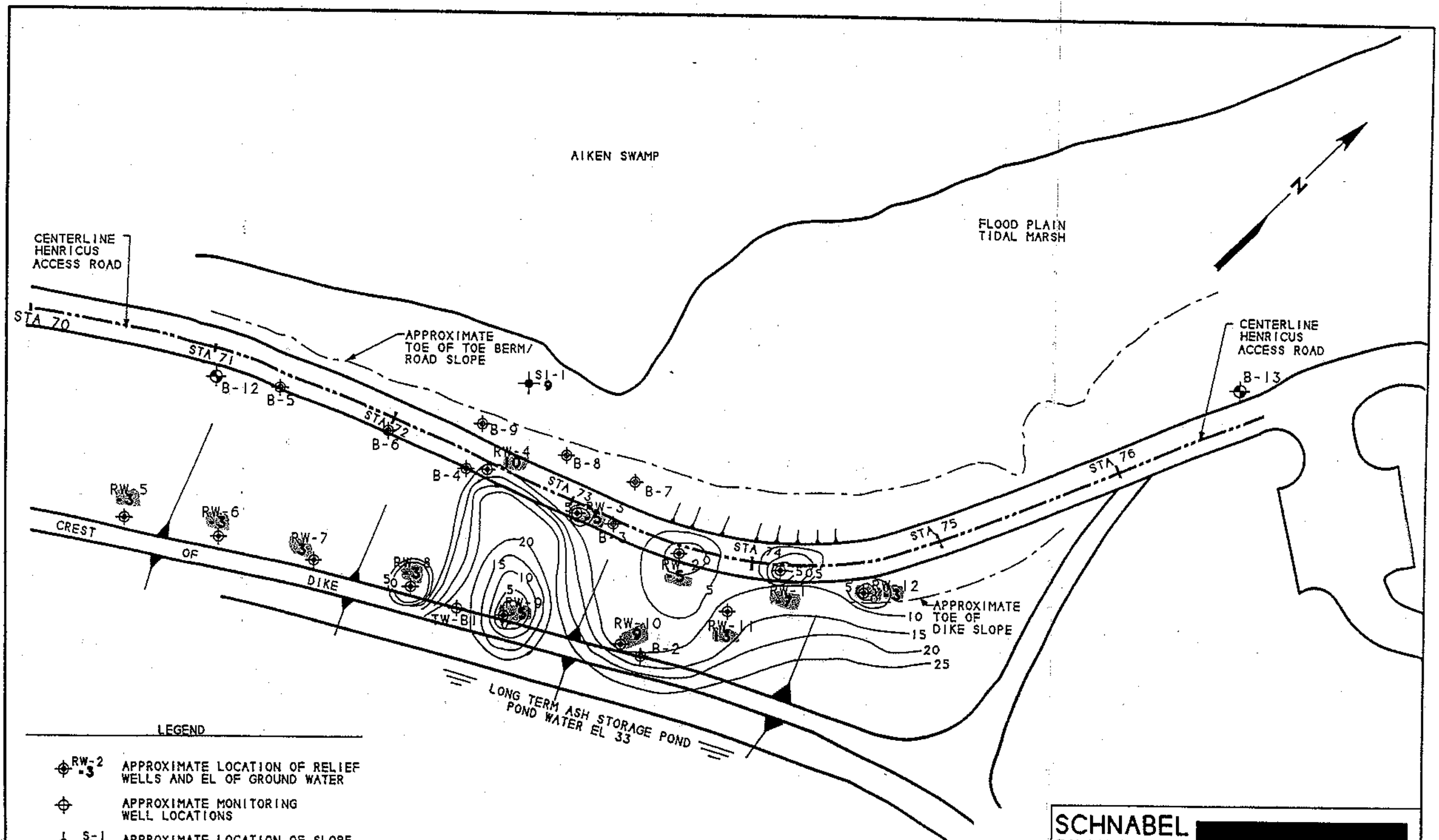
- RW-2 22 — APPROXIMATE RELIEF WELL LOCATIONS AND EL OF GROUND WATER
- APPROXIMATE MONITORING WELL LOCATIONS
- SLOPE INCLINOMETER AND EL OF GROUND WATER
- APPROXIMATE LOCATION OF PREVIOUS TEST BORINGS BY SCHNABEL ENGINEERING
- APPROXIMATE LOCATION OF PREVIOUS TEST BORINGS BY VIRGINIA POWER
- 10 — APPROXIMATE GROUND WATER CONTOURS

SCHNABEL ENGINEERING ASSOCIATES [redacted]

LONG TERM ASH STORAGE POND DIKE

NORTH EMBANKMENT, CHESTERFIELD CO., VA

PHREATIC SURFACE WITH POND AT EL 33	SCALE 1"=50'	DATE 4-22-96	REVISED DATE
	DRAWN BY VAM	CHECKED BY GS	REVISED DATE
	CONTRACT NO. 953432A	FIGURE 6	REVISED DATE 953432A



LEGEND

- RW-2 / 3 — APPROXIMATE LOCATION OF RELIEF WELLS AND EL OF GROUND WATER
- APPROXIMATE MONITORING WELL LOCATIONS
- S-1 / 9 — APPROXIMATE LOCATION OF SLOPE INCLINOMETER AND EL OF GROUND WATER
- APPROXIMATE LOCATION OF PREVIOUS TEST BORINGS BY SCHNABEL ENGINEERING
- APPROXIMATE LOCATION OF PREVIOUS TEST BORINGS BY VIRGINIA POWER
- 10 — APPROXIMATE GROUND WATER CONTOURS

SCHNABEL ENGINEERING ASSOCIATES [redacted]

LONG TERM ASH STORAGE POND DIKE

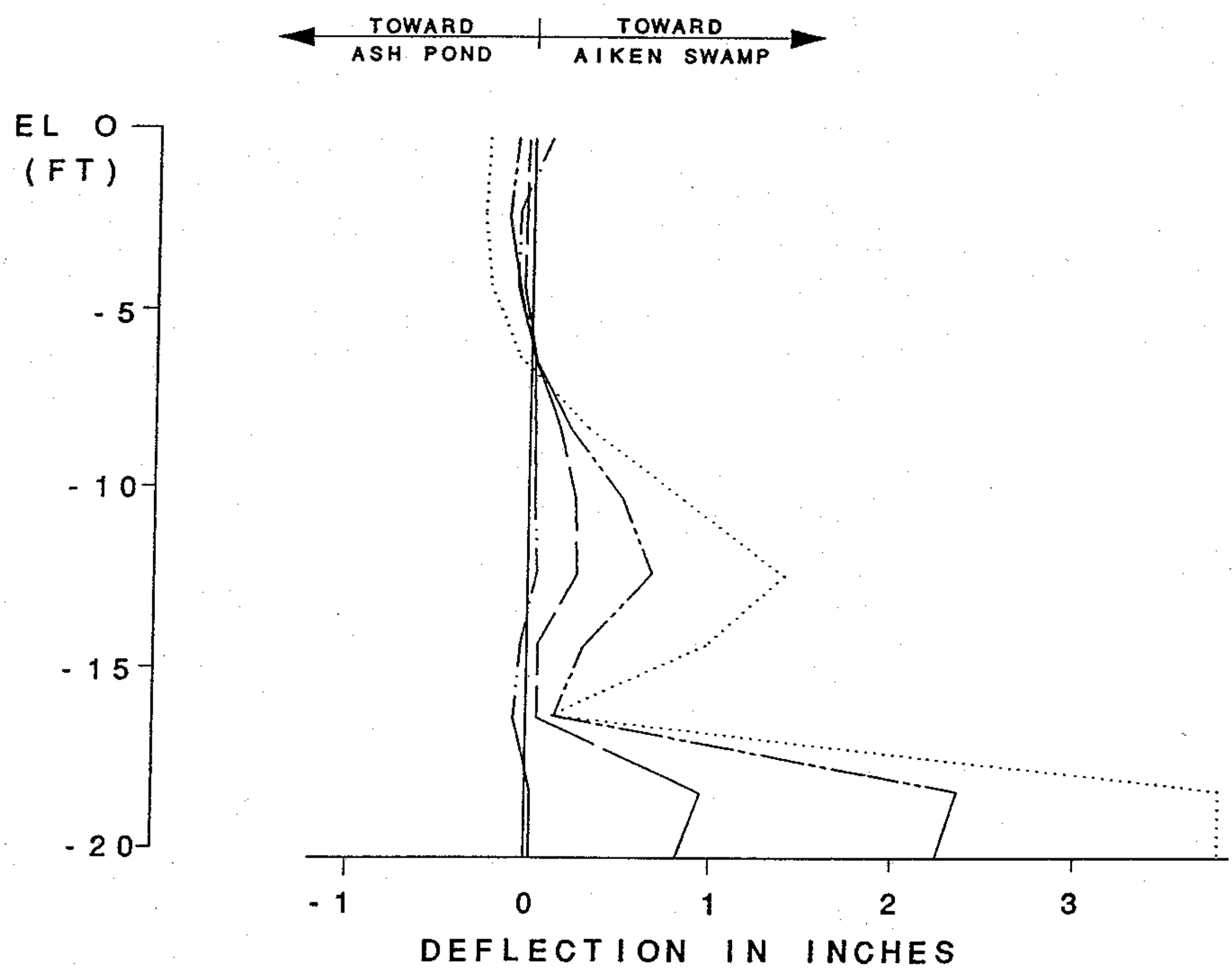
NORTH EMBANKMENT, CHESTERFIELD CO., VA

APPROXIMATE GROUND WATER WITH PUMPS	SCALE 1"=50'	DATE 4-22-96	REVISED DATE
	DRAWN BY VAM	CHECKED BY GS	REVISED DATE
	CONTRACT NO. 953432A	FIGURE 7	REVISED DATE

FIGURE 8

LONG TERM ASH STORAGE POND DIKE
NORTH EMBANKMENT
CHESTERFIELD COUNTY, VIRGINIA

SLOPE INCLINOMETER DEFLECTIONS



LEGEND

CONTROL READING OCT. 5, 1994

CONTROL READING OCT. 5, 1994

CRACK FIRST OBSERVED JAN. 22, 1995

CRACK OBSERVED JAN. 25, 1995

CRACK OBSERVED FEB. 1, 1995

NOTE: GROUND SURFACE AT ELEVATION ±8.9.

953432A

3. GEOTECHNICAL ANALYSIS AND RECOMMENDATIONS

3.1 Discussion

The embankment and natural abutment soils contain interbedded layers of high permeability sands and low permeability clays. High permeability sands comprise the lower portion of the Pleistocene terrace soils at the dike and abutment generally from about El 15 to El 0. The old James River Channel probably existed in Aiken Swamp parallel to the north dike embankment.

Seepage exiting high on the embankment slope (El 24 to 26) while relief wells are pumping, supports the assumption that the embankment and natural abutment soils contain interbedded layers of low and high permeability soils.

Prior to pumping, high seepage pressures were present in the underlying Strata B and C soils when the pond level approached El 30. The slope inclinometer indicated an excess hydrostatic head of 1.9 ft above ground surface at the toe of slope (El 10.7). The phreatic surface represented by this reading was not considered in the original design of the north dike embankment. Control of seepage pressures were identified as an important consideration during the original design, and were assumed to be controlled by a toe drain such that the hydrostatic head at the toe of slope would not exceed El +5.

We believe that three different types of slope movements have occurred. All three have been mobilized as a direct result of excess pore water pressure and seepage forces within the embankment and abutment. These include: 1) a mud flow in the abutment at about Sta 74+20 to 74+40, 2) a block movement of the geogrid-reinforced section from about 73+10 to 74+20, 3) a rotational movement of the embankment dike slope above the road from about Sta 71+70 to 73+10, passing through the slope inclinometer at about El -10 to -20, and an extension up this slide area to about Sta 70+70.

During the present hydraulic filling of the ash pond, relief wells are being used to control the phreatic surface and maintain stability. For future dredge filling operations, long-term remedial alternatives that would be effective in reducing or cutting off the amount of seepage through the embankment dike have been considered. We evaluated these alternatives based on seepage control, stability, cost, construction duration, disturbance to the embankment dike, future maintenance, and risk. The alternatives considered include:

- Pumping from relief wells and installing an internal ash berm,

- Installing a cutoff along the crest of dike (steel sheet pile, HDPE wall, slurry wall or grout curtain),

- Installing a horizontal drain that ties into the toe drain.

- Creating a new dike within the ash pond followed by excavating to ash adjacent to the existing embankment and pumping seepage back to the ash pond.

We have evaluated two sections of the embankment dike using a computer flow model to evaluate phreatic surfaces and stability. The modeling was performed to assist in evaluating the effectiveness of several of the remedial measures being considered for stabilizing the slope.

The existing slope inclinometer is not suitable for monitoring future slope movements due to the excessive deflections at the bottom of the inclinometer casing and the unknown response to the casing above the bottom.

3.2 Long-Term Remedial Alternatives

We have considered that another dredge of the ash pond will be forthcoming several years from now. However, we understand that Virginia Power has about 2.4 million cubic yards of space left in the ash pond. It is possible that they could move about 2 million cy during this dredging, which would preclude the need for another major dredging. However, they are also looking for markets to sell the ash. If successful, they would likely excavate dry ash from the long-term pond and at some point dredge again and sluice into this pond. This could happen more than once in the years to come. Any future dredges will likely result in pond water level of about El 40.

Long-term remedial alternatives that would be effective in reducing or cutting off the amount of seepage through the embankment dike are discussed below. Table 5 summarizes the advantages and disadvantages of the long-term alternatives. It should be noted that, depending on the current dredge schedule, it may be necessary to implement the remedial alternative during this dredge to reduce seepage if water levels exceed those previously attained during this dredge.

TABLE 5 LONG-TERM REMEDIAL ALTERNATIVES

REMEDIAL MEASURE	ADVANTAGES	DISADVANTAGES
Relief Wells/Compacted Ash Berm	Demonstrated effectiveness for pond levels to El 35 Does not disturb embankment Flexibility of adding new wells in other areas if needed Low cost, estimated $85,000 to $90,000 for compacted ash berm System is in place except for ash berm	System must be maintained until pond level below El 20 Monitoring and maintenance required Permanent ash berm reduces volume of saleable ash in pond Slope movements possible if system damaged or cut off
Steel Sheet Pile Cutoff	Level of effort can be well defined Known cost Effects observed shortly after installation Rigid structural section in dike Short construction time - 1 to 2 weeks	Vibrations during driving could effect slope stability High cost: $250,000 - $300,000 Some leakage expected Success of a fully penetrating cutoff questionable due to difficult installation in dense sands and gravels If cutoff not complete, will likely have to maintain wells
HDPE Panel Cutoff	Level of effort can be well defined Known cost Effects observed shortly after installation Essentially no leakage anticipated between panels	High cost: $315,000 to $350,000 Installation below 40 ft may require slurry wall construction, which would be cost prohibitive Introduces potential shear plain in embankment Construction time depends on method of installation
Grout Curtain	Increase in embankment strength Can install to desired cutoff depth through dense sands and gravels Moderate cost: $110,000 - $130,000 Effects observed during and shortly after installation Wells may be used to monitor grout program	Some seepage likely Actual cost depends on grout volumes May damage existing relief wells Construction time - 6 to 8 weeks Depending on controlled phreatic surface, may need to maintain wells during final dredging
Soil or Cement Bentonite Slurry Wall	Can install to desired cutoff depth through dense materials Moderate cost: $200,000 soil-bentonite, $275,000 cement-bentonite Will know effects quickly Cement-bentonite wall does not reduce overall strength	Soil-bentonite wall intorduces potential shear plain within embankment
Horizontal Well	Could be used indefinitely as it will be tied to toe drain Moderate cost: $100,000 to $120,000	If directional-drilled; would only be a supplement to existing relief wells Effectiveness limited to permeable zones intersected

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3.2.1 Relief Wells/Interior Ash Berm

The installation of an interior ash berm and use of the current relief well system for long-term stability of the embankment dike is considered feasible. The relief well system demonstrated its effectiveness for pond levels up to about El 35 until the recent slope movement in April. It generally does not disturb the embankment and it is in-place. Since it is in-place, the cost is low except that it will require maintenance and regular monitoring and additional evaluation will be required to assess the most recent movement. The system is flexible in that new wells can be added where or when necessary and this may be the solution to offset the most recent movement. It should remain in place after dredging has ceased until water levels in the pond are below about El 20. This system poses the most risk in that it depends on mechanical equipment that will require maintenance. It also depends on an uninterrupted power supply.

If the relief well system is maintained as the long-term alternative, a horizontal compacted ash berm at least 60 ft wide, graded on a 2.5 H to 1.0 V slope into the pond should be placed on the inside of the embankment between Station 70+50 and 76+00. The berm should be constructed after the current dredging has been completed and the water level in the pond is at the lowest level.

We recommend that the ash berm be placed in 1 ft lifts and compacted with vibrating equipment. Field permeability or 'slug' tests should be performed on the underlying dredged ash prior to construction of the ash berm to evaluate the permeability of the in-place ash. In addition, we recommend that ash permeabilities be evaluated in the laboratory for various densities of compacted ash to develop material and compaction requirements for the ash to be placed in the compacted ash berm.

A typical embankment section was modeled using Visual MODFLOW Version 1.5 (1995) by Waterloo Hydrogeologic Software. We modeled a water level in the pond at El 40 and a 60 ft compacted ash berm with no relief wells operating. Horizontal and vertical permeabilities of 5×10^{-5} and 1×10^{-5} cm/sec were assumed for the ash. The permeabilities for the soil strata are provided in Table 4, Section 2.6.

The ash berm is shown to be effective in reducing the resulting phrenetic surface to levels commensurate of those with a pond water level at about El 20. For this condition, the embankment is considered stable. The estimated phrenetic surface using the ash berm is illustrated on Figure 9 as Case 1. In comparison, the observed phrenetic surface for pond level El 20 is shown as Case 3.

The relative cost of this alternative is low. Except for the ash berm, this remedial alternative is in place. Construction of an ash berm is estimated to cost $85,000 to $90,000.

3.2.2 Steel or HDPE Sheet Pile Cutoff

A seepage cutoff made by installing steel sheet piles or HDPE panels is considered as a long-term alternative. This alternative would consist of installing sheet piles to the Cretaceous clays and sands of Strata E or F or to an estimated depth of 40 to 50 ft with the deeper zone between RW-5 to RW-9. The cutoff would extend from Sta 71+50 to 75+00. Some effects of these methods to cut off seepage would be observable shortly after their installation within the relief wells. The steel sheet pile cutoff would provide a rigid structural section in the embankment, however, some leakage would be expected between the sheets. The HDPE cutoff would have essentially no leakage between panels, but would introduce a potential shear plain in the embankment. The construction time for installation of the steel sheet piles would be on the order of 1 to 2 weeks. However, success of a fully penetrating cutoff into the Stratum E or F soils is questionable due to likely refusal in the dense sands and gravels of Stratum C. Therefore, it should be assumed that the relief wells must remain operational as a backup to the steel sheetpile alternative. Vibrations during driving may also adversely impact the slope.

Construction time for the HDPE cutoff would depend on the method of installation. Installation of the HDPE cutoff below about 40 ft would require slurry wall type construction, which would be cost prohibitive. Estimated cost of the steel sheet pile cutoff would be in the range of $250,000 to $300,000. The estimated cost of the HDPE panel wall would be in the range of $315,000 to $350,000 or higher, if slurry construction is necessary.

To evaluate the effects of the cutoffs, a typical embankment section was modeled using Visual MODFLOW. We considered the water level in the pond at El 40 and a fully penetrating cutoff wall to the Strata E and F soils. Horizontal and vertical permeabilities used were presented in Table 4 Section 2.6. The resulting phrenetic surface is illustrated on Figure 9 as Case 2. Again, it is about the same as that for a pond water level at El 20, at which point the embankment is stable.

3.2.3 Grout Curtain

Pressure grouting consists of injecting a cement grout under pressure from grout pipes as they are withdrawn from their jetted or driven depth (estimated about El 0). The grout is injected in an alternating pattern of widely spaced holes with intermediate holes grouted as needed. We estimate that about 350 linear ft of embankment section would be grouted to a depth of about 40 ft. The grouting would be performed along the crest of the embankment dike.

Chemical grouts, having a shorter gel time, may be needed in areas where high permeability soils are encountered. The use of chemical grouts would be on an as necessary basis as they are more expensive. Pressure grouting will provide a seepage cutoff and an increase in the embankment strength in the grouted zone. A fully penetrating grout curtain should be possible through the dense Stratum C sands.

The effects of the grouting would be observed during and shortly after the program is completed. The existing relief wells would provide monitoring points for evaluating the effectiveness of the grouting program. However, relief wells along the top of the dike may be damaged and need to be replaced following the grouting program. The estimated time for performing the pressure grouting program is about 6 to 8 weeks with an estimated cost of $130,000 to $150,000.

The effects of the grout curtain alternative are considered the same as for a sheet pile cutoff. Because the likelihood of leakage through the grout curtain is higher, we recommend that the relief wells be maintained to provide a backup system.

3.2.4 Slurry Cutoff Wall

A soil-bentonite or cement-bentonite slurry cutoff wall may be installed as a long-term alternative. The wall could be excavated to the desired cutoff, about El -2 to-8, through the dense soils of Stratum C. The wall length would be about 350 ft. The cost of the slurry wall would be moderate and slightly higher than the pressure grouting. Like the HDPE cutoff, the soil-bentonite slurry cutoff wall would introduce a potential shear plain in the embankment. However, the cement-bentonite cutoff would not reduce the overall strength. Modeling the slurry cutoff wall alternative was considered the same as modeling other cutoff alternatives, with about the same effect in lowering the phreatic surface. Costs of the soil-bentonite and cement-bentonite slurry cutoffs are estimated to be about $200,000 and $275,000, respectively.

3.2.5 Horizontal Drain

This type of drain may be installed by directional drilling methods or by using narrow width trenching. It would consist of a horizontal, slotted, 6-inch diameter HDPE pipe surrounded with a graded sand filter. Where trenching is used, a one-step process is employed that allows the entire trench to be backfilled with sand or gravel while excavating. This method provides a vertical interceptor to control seepage through the embankment above the drain.

The horizontal drain would connect to the existing embankment toe drain at about Sta 69+00. We estimate a horizontal drain installed by the trench method would be located along the Park access road so it could be tied to the existing toe drain. The horizontal drain would be installed to a depth of about 15 to 20 ft in the abutment area and 5 ft where it intercepts the toe drain. Assuming a drain length of about 600 ft, the estimated cost for this alternative would be about $100,000 to $110,000.

Trenching may have some depth limitations but this would depend somewhat on the location of the horizontal drain. This alternative could be used indefinitely as it would be tied to the existing toe drain. However, this alternative would depend on mechanical systems to continuously remove seepage. The capacity of the toe drain pumps may also have to be increased to handle the additional flow. We would expect the existing relief wells would be maintained as a backup for the horizontal drain.

If the directional drilling technology is used to install the drain, its effectiveness would be limited to the permeable zones intersected during drilling. This alternative should only be considered as a supplement to the existing relief well system. A moderate cost of about $120,000 for drilling the horizontal well would be expected.

3.2.6 New Dike within the Ash Pond

A new dike could be constructed on top of the ash presently in the pond adjacent to the embankment. The steps would involve 1) lowering the water in the pond to the top of the present ash, 2) constructing a fly ash embankment to El 42 with an internal drainage system consisting of a bottom ash drainage system and 3) instructing a sump and pump to collect seepage and return it to the pond.

The advantage of the internal berm is the seepage head would be reduced to about El 15 to 20 on the upstream side of the existing embankment. A more stable condition would most likely exist in the area of concern based on the stability evaluations discussed below, since the phreatic surface through the area would be lower than with other cutoff alternatives.

The disadvantage is the loss of ash pond storage and long term pumping costs. We estimate the embankment construction cost at about $500,000. This assumes a 800 ft long structure and material obtained from the present pond.

The dike should be constructed with the center line about 100 ft from the existing embankment ash toe at about El 15 with side slopes of about 3.5H or 4.0H to 1V. A typical section is shown on Figure 10.

3.3 Slope Inclinometer

Four slope inclinometers should be installed along the toe of the ash pond embankment as soon as possible. They should be installed with the lower section of casing into the Strata E or F Cretaceous soils (estimated El -30 to -40). The inclinometers should be installed at Stations 69+50, 71+00, 72+50, and Sta 74+00. Initial readings should be obtained after installation. A monitoring program should be determined based on present and future dredging activity and water levels in the ash pond.

3.4 Stability Evaluation

A block movement of the geogrid-reinforced slope, from about Sta 73+10 to 74+20 was evaluated. Basically the geogrid-reinforced section experienced a shear displacement more akin to a bearing capacity failure. This is suggested by the observed movement of the failed section relative to the toe of slope (see Figure 11). Furthermore, the magnitude of the displacement (3.5 ft horizontal and 4.5 ft vertical) is not compatible with the strains observed for the deeper soils in the slope inclinometer. A sketch of this failed section is shown on Figure 11. The computer generated surface is included in Appendix C. The slope movement in this section was a failure resulting from a loss of supporting strength in the underlying soils due to excess seepage pressures.

Considering the relationship of the escarpment west of the geogrid-reinforced section of the slope and the depth of greatest displacement in the inclinometer, a traditional rotational displacement was modeled for this area. Lower factors of safety for shallower failures, generally representative of sloughing, were indicated but not considered since the primary failure surface, as indicated by the slope inclinometer was deeper. A sketch of this section is shown on Figure 12 and the computer generated surfaces are shown in Appendix C.

Taking the observed pond level and phrenetic surface at the time of failure, and assuming a factor of safety of 1.0, we performed a back analysis and adjusted the strength parameters of Stratum B accordingly. Using the modified Stratum B parameters (c=0, ⌀=23°), we performed stability evaluations on embankment sections at Sta 72+50 and 73+50 using PCSTABL6H.

Stability was evaluated for a future pond water level at El 40. Two scenarios were considered, a seepage cutoff wall and the addition of an ash berm. The phrenetic surface resulting from these engineering controls as estimated from two-dimensional computer modeling, were used in the analyses. The results of these stability analyses are tabulated below.

Alternative	Pond Water Surface EL	Factors of Safety
Seepage Cutoff Wall	EL 40	1.2-1.3
Ash Berm	EL 40	1.3-1.4

A stability evaluation of the slope was performed using the modified Stratum B parameters and a pond water level at El 20. This elevation represents the highest verified pond level where the north dike transition area was stable. A factor of safety of 1.35 was estimated. Based on this evaluation, we believe that once the pond water level is below El 20, monitoring of the slope inclinometers and operation of the relief wells (if in service) can be discontinued.

3.4.1 Long-term Alternatives

For any of the remedial alternatives, we believe the ash pond embankment dike would remain stable assuming a maximum pond water level at El 40. The factors of safety are lower than 1.5 which would typically be considered for steady state seepage conditions. Therefore, it is important that the slope inclinometers be installed and monitored regularly during dredging whenever the pond water level is above El 30. In addition, since relief wells are a backup to the long-term alternatives, they should remain in service and be inspected on a daily basis as long as the pond water level is above El 30.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS AUGER PROBE LOG	**Project:** Slope Henricus Park Road Chesterfield County, Virginia	**Contract Number:** 953432 **Boring Number:** RW-1 **Sheet:** 1 of 1

Boring Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia

Boring Foreman: M. Young

Drilling Method: 10" Hollow Stem Auger

Drilling Equipment: CME-55

SEA Representative: D. Snyder

Dates Started: 1/26/96 **Finished:** 1/26/96

Location: See Boring Location Plan, Figure A1

Ground Surface Elevation: 32.4±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	1/26	10:30	13.5	--	--
Completion	1/26	12:38	13.5	--	--
Casing Pulled	1/26	--	--	--	--
	1/27	--	12.0	--	--
	1/28	--	12.7	--	--

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
	Asphalt	FILL						EMBANKMENT
	Basecourse							FILL
	Fine to medium clayey sand FILL, moist - brown							
6.0	Fine to medium silty sand FILL, moist - brown	FILL	26.4	A				Hard drilling
13.0	Clayey sand, moist - brown	SC	19.4					PLEISTOCENE
16.0	Fine to coarse silty sand, wet - brown	SM	16.4	C				
29.0	Fat clay with sand, moist - dark gray	CH	3.4	F				Cretaceous Hard drilling
42.0	Auger probe terminated at 42.0 ft		-9.6					

Comments:
1) 6" dia. PVC relief well installed to 42 ft. Screen from El +10 to -10. Pump installed at El -8.
2) Soil classification based on observation of cuttings and soil on augers.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS AUGER PROBE LOG	**Project:** Slope Henricus Park Road Chesterfield County, Virginia	**Contract Number:** 953432 **Boring Number:** RW-2 **Sheet:** 1 of 1

Boring Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia

Boring Foreman: G. Richards

Drilling Method: 10" Hollow Stem Auger

Drilling Equipment: ATV-45

SEA Representative: D. Snyder

Dates Started: 1/26/96 **Finished:** 1/26/96

Location: See Boring Location Plan, Figure AI

Ground Surface Elevation: 30.2±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	1/26	12:45	7.9	--	--
Completion	1/26	--	7.9	--	--
Casing Pulled	1/26	--	--	--	--

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
	Asphalt Basecourse Fine to coarse silty sand FILL, moist - brown do, wet - tan	FILL		A				EMBANKMENT FILL Hard drilling
12.0	Fine to coarse clayey sand, moist - brown and tan do, wet	SC	18.2	C				PLEISTOCENE
28.0	Fat clay with gravel, moist - dark gray do, no gravel	CH	2.2	F				Cretaceous
40.0	Auger probe terminated at 40.0 ft		-9.8					

Comments:
1) 6" dia. PVC relief well installed to 40 ft. Screen from El +10 to -10. Pump installed at El -8.
2) Soil classification based on observation of cuttings and soil on augers.
3) Note: No cuttings during drilling of Calvert soils.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS AUGER PROBE LOG	Project: Slope Henricus Park Road Chesterfield County, Virginia	Contract Number: 953432 Boring Number: RW-3 Sheet: 1 of 1

Boring Contractor: Fishburne Drilling, Inc. Chesapeake, Virginia
Boring Foreman: G. Richards
Drilling Method: 10" Hollow Stem Auger
Drilling Equipment: ATV-45
SEA Representative: D. Snyder
Dates Started: 1/26/96 **Finished:** 1/26/96
Location: See Boring Location Plan, Figure A1

Ground Surface Elevation: 25.9±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	1/26	10:30	3.8	--	--
Completion	1/26	--	3.1	--	--
Casing Pulled	1/26	--	--	--	--
	1/27	8:32	4.2	--	--

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
	Asphalt Basecourse							EMBANKMENT FILL
4.0	Fine to medium clayey sand FILL, moist - brown	FILL	21.9	A				
17.0	Fine to coarse clayey sand, wet - tan do, gravel	SC	8.9	C				PLEISTOCENE Hard drilling
30.0	Fat clay with sand, moist - dark gray	CH	-4.1	F				Cretaceous
36.0	Auger probe terminated at 36.0 ft		-10.1					

Comments:
1) 6" dia. PVC relief well installed to 42 ft. Screen from El +10 to -10. Pump installed at El -8.
2) Soil classification based on observation of cuttings and soil on augers.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS AUGER PROBE LOG	**Project:** Slope Henricus Park Road Chesterfield County, Virginia	**Contract Number:** 953432 **Boring Number:** RW-4 **Sheet:** 1 of 1

Boring Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia

Boring Foreman: M. Young

Drilling Method: 10" Hollow Stem Auger

Drilling Equipment: CME-55

SEA Representative: D. Snyder

Dates Started: 1/26/96 **Finished:** 1/27/96

Location: See Boring Location Plan, Figure A1

Ground Surface Elevation: 21.7 ±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	1/26	--	9.5	--	--
Completion	1/27	--	7.9	--	--
Casing Pulled	1/27	--	7.3	--	--

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
	Asphalt Basecourse Fine to medium clayey sand FILL, moist - brown	FILL		A				EMBANKMENT FILL
8.0	Fine to medium sandy lean clay, wet - brown	CL	13.7	B				ALLUVIUM
14.0	Fine to medium clayey sand, wet - tan do, gravel	SC	7.7	C				PLEISTOCENE Augers hitting gravel
30.0	Auger probe terminated at 30.0 ft		-8.3					

Comments:
1) 6" dia. PVC relief well installed to 30 ft. Screen from El +10 to -10. Pump installed at El -4.3.
2) Soil classification based on observation of cuttings and soil on augers.
3) Rods/hammer hung up on augers after contractor knocked out plug; augers were removed from hole and redrilled back into hole.
4) Contractor required several attempts to install well casing.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS AUGER PROBE LOG	Project: Long Term Ash Storage Pond Dike North Embankment Chesterfield County, Virginia	Contract Number: 953432A Boring Number: RW-5 Sheet: 1 of 1

Boring Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia
Boring Foreman: G. Richards
Drilling Method: 10" Hollow Stem Auger
Drilling Equipment: CME-55
SEA Representative: D. Snyder
Dates Started: 2/15/96 **Finished:** 2/17/96
Location: See Boring Location Plan, Figure A1

Ground Surface Elevation: 42.0±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	2/15	--	14.5	--	--
Completion	2/15	--	13.0	--	--
Casing Pulled	--	--	--	--	--
RW-5	2/17	--	14.4	--	--

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
	Fine to medium silty sand FILL, moist - brown do, with gravel	FILL		A				Hard drilling EMBANKMENT FILL Hard drilling
10.0	Fine to medium clayey sand FILL with gravel	FILL	32.0					
12.0	Fine to coarse sandy lean clay with gravel, moist - dark brown	CL	30.0	C				PLEISTOCENE Soft
26.0	Fine to medium clayey sand, wet - brown	SC	16.0					
27.0	Fine to coarse silty sand, wet - tan	SM	15.0					Hitting gravel Hard drilling
40.0	Fat clay with gravel do, trace gravel	CH	2.0	F				CRETACEOUS
52.0	Auger probe terminated at 52.0 ft		-10.0					

Comments:
1) 6" dia. PVC relief well installed to El -10. Screen from El 30 to -10. Pump installed at El -8.
2) Soil classification based on observation of auger cutting and soil on augers.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS AUGER PROBE LOG	**Project:** Long Term Ash Storage Pond Dike North Embankment Chesterfield County, Virginia	**Contract Number:** 953432A **Boring Number:** RW-6 **Sheet:** 1 of 1

Boring Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia
Boring Foreman: M. Scott
Drilling Method: 10" Hollow Stem Auger
Drilling Equipment: CME-55
SEA Representative: D. Snyder
Dates Started: 2/14/96 **Finished:** 2/14/96
Location: See Boring Location Plan, Figure A1

Ground Surface Elevation: 42.0±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	2/14	--	22.0	--	--
Completion	2/14	--	18.0	--	--
Casing Pulled	2/17	--	15.4	--	--

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
.0	Fine to coarse clayey sand with gravel FILL, moist - brown	FILL	42.0	A				EMBANKMENT FILL Hard drilling
10.0	Fine to coarse sandy lean clay with gravel, moist - brown	CL	32.0	C				PLEISTOCENE
15.0	Fine to medium silty sand, moist - brown	SM	27.0					
18.0	Fine to medium clayey sand, moist - brown	SC	24.0					Easier drilling
25.0	Fine to medium sandy lean clay, moist - brown	CL	17.0					
30.0	Fine to coarse silty sand with gravel, wet - brown	SM	12.0					
40.0	Gravel with fat clay pockets	GC	2.0	E				CRETACEOUS
52.0	Auger probe terminated at 52.0 ft		-10.0					

Comments:
1) 6" dia. PVC relief well installed to El -9. Screen from El 30 to -9. Pump installed at El -7.
2) Soil classification based on observation of auger cutting and soil on augers.
3) Contractor had problems with running sand during well installation.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS AUGER PROBE LOG	**Project:** Long Term Ash Storage Pond Dike North Embankment Chesterfield County, Virginia	**Contract Number:** 953432A **Boring Number:** RW-7 **Sheet:** 1 of 1

Boring Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia
Boring Foreman: M. Scott
Drilling Method: 10" Hollow Stem Auger
Drilling Equipment: CME-55
SEA Representative: D. Snyder
Dates Started: 2/13/96 **Finished:** 2/14/96
Location: See Boring Location Plan, Figure A1

Ground Surface Elevation: 42.0±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	2/14	--	16.0	--	--
Completion	2/14	--	18.0	--	--
Casing Pulled	--	--	--	--	--
RW-7	2/17	--	15.7	--	--

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
	Root mat Fine to coarse silty sand with gravel FILL, moist - brown	FILL		A				EMBANKMENT FILL Hard drilling
9.0	Fine to coarse clayey sand FILL with gravel, moist - brown	FILL	33.0					
16.0	Fine to coarse clayey sand, moist - brown	SC	26.0	C				PLEISTOCENE Soft
23.0	Fine to medium sandy lean clay, moist to wet - brown	CL	19.0					
37.0	Fine to medium silty sand with gravel, wet - brown to gray	SM	5.0					Hard drilling
52.0	Auger probe terminated at 52.0 ft		-10.0					

Comments:
1) 6" dia. PVC relief well installed to El 7. Screen from El 33 to 7. Pump installed at El 5.
2) Soil classification based on observation of auger cutting and soil on augers.
3) No auger cuttings from 36 to 42 ft.
4) Wooden plug broken in augers. Contractor had to pull augers before installing well.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS AUGER PROBE LOG	Project: Long Term Ash Storage Pond Dike North Embankment Chesterfield County, Virginia	Contract Number: 953432A Boring Number: RW-8 Sheet: 1 of 1

Boring Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia
Boring Foreman: M. Scott
Drilling Method: 10" Hollow Stem Auger
Drilling Equipment: CME-55
SEA Representative: D. Snyder
Dates Started: 2/15/96 Finished: 2/17/96
Location: See Boring Location Plan, Figure A1

Ground Surface Elevation: 42.0±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	2/15	--	18.0	--	--
Completion	2/16	--	18.0	--	--
Casing Pulled	--	--	--	--	--
RW-8	2/17	--	15.2	--	--

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
0	Fine to medium clayey sand with gravel FILL, moist - brown	FILL	42.0	A				EMBANKMENT FILL Hard drilling
12.0	Fine to medium silty sand, trace gravel, moist - brown	SM	30.0	C				PLEISTOCENE
16.0	Fine to medium clayey sand, wet - brown	SC	26.0					
19.0	Fine to coarse silty sand, wet - tan	SM	23.0					
40.0	Gravel with fat clay, moist - gray	GC	2.0	E				CRETACEOUS
45.0	Fat clay with gravel, moist - dark gray	CH	-3.0	F				
51.0	Auger probe terminated at 51.0 ft		-9.0					

Comments:
1) 6" dia. PVC relief well installed to El -9. Screen from El 34 to -6. Pump installed at El -4.
2) Soil classification based on observation of auger cutting and soil on augers.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS AUGER PROBE LOG	**Project:** Long Term Ash Storage Pond Dike North Embankment Chesterfield County, Virginia	**Contract Number:** 953432A **Boring Number:** RW-9 **Sheet:** 1 of 1

Boring Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia
Boring Foreman: G. Richards
Drilling Method: 10" Hollow Stem Auger
Drilling Equipment: CME-55
SEA Representative: D. Snyder
Dates **Started:** 2/15/96 **Finished:** 2/17/96
Location: See Boring Location Plan, Figure A1

Ground Surface Elevation: 42.0±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	2/15	--	-19.0	--	--
Completion	2/16	--	--	--	--
Casing Pulled	2/17	--	-15.5	--	--

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
	Fine to coarse clayey sand with gravel FILL, moist - brown	FILL		A				EMBANKMENT FILL Hard drilling
11.0	Fine to coarse clayey sand with gravel, moist - brown	SC	31.0	C				PLEISTOCENE
18.0	Fine to coarse clayey sand, moist - brown	SC	24.0					
29.0	Fine to medium silty sand, wet - tan	SM	13.0					
42.0	Fat clay with gravel, moist - dark gray	CH	.0	F				CRETACEOUS
50.0	Fat clay with sand, moist - dark gray	CH	-8.0					
52.0	Auger probe terminated at 52.0 ft		-10.0					

Comments:
1) 6" dia. PVC relief well installed to El -9.5. Screen from El 30 to -9.5. Pump installed at El -7.5.
2) Soil classification based on observation of auger cutting and soil on augers.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS AUGER PROBE LOG	**Project:** Long Term Ash Storage Pond Dike North Embankment Chesterfield County, Virginia	**Contract Number:** 953432A **Boring Number:** RW-10 **Sheet:** 1 of 1

Boring Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia

Boring Foreman: G. Richards
Drilling Method: 10" Hollow Stem Auger
Drilling Equipment: CME-55
SEA Representative: D. Snyder
Dates Started: 2/19/96 **Finished:** 2/19/96
Location: See Boring Location Plan, Figure A1

Ground Surface Elevation: 40.0±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	2/19	--	-23.5	--	--
Completion	2/19	--	--	--	--
Casing Pulled	--	--	--	--	--
RW-10	2/19	--	23.0	--	--

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
	Fine to coarse silty sand FILL with gravel, moist - brown	FILL		A				EMBANKMENT FILL
	Fine to coarse sandy lean clay FILL with gravel, moist - brown							
7.0	Fine to coarse silty sand, trace gravel, moist - brown	SM	33.0	C				Hard drilling
18.0	Fine to medium sandy lean clay, moist - brown	CL	22.0					PLEISTOCENE
24.0	Fine to coarse silty sand, trace gravel, wet - brown	SM	16.0					
35.0	Auger probe terminated at 35.0 ft		5.0					

Comments:
1) 6" dia. PVC relief well installed to El 5.5. Screen from El 29 to 5.5. Pump installed at El 7.
2) Soil classification based on observation of auger cutting and soil on augers.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS AUGER PROBE LOG	**Project:** Long Term Ash Storage Pond Dike North Embankment Chesterfield County, Virginia	**Contract Number:** 953432A **Boring Number:** RW-11 **Sheet:** 1 of 1

Boring Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia
Boring Foreman: G. Richards
Drilling Method: 10" Hollow Stem Auger
Drilling Equipment: CME-55
SEA Representative: D. Snyder
Dates **Started:** 2/15/96 **Finished:** 2/15/96
Location: See Boring Location Plan, Figure A1

Ground Surface Elevation: 37.0±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	2/15	--	9.0	--	--
Completion	2/15	--	12.0	--	--
Casing Pulled	--	--	--	--	--
RW-11	2/17	--	9.9	--	--

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
	Fine to medium sandy lean clay FILL, trace gravel, moist - brown	FILL		A				EMBANKMENT FILL
8.0	Fine to coarse clayey sand, trace gravel, moist - brown do, wet	SC	29.0					
20.0	Fine to coarse silty sand, wet - brown and tan do, with gravel	SM	17.0	C				PLEISTOCENE
32.0	Auger probe terminated at 32.0 ft		5.0					

Comments:
1) 6" dia. PVC relief well installed to El 6. Screen from El 26 to 6. Pump installed at El 8.
2) Soil classification based on observation of auger cutting and soil on augers.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS AUGER PROBE LOG	**Project:** Long Term Ash Storage Pond Dike North Embankment Chesterfield County, Virginia	**Contract Number:** 953432A **Boring Number:** RW-12 **Sheet:** 1 of 1

Boring Contractor: Fishburne Drilling, Inc.
Chesapeake, Virginia
Boring Foreman: G. Richards
Drilling Method: 10" Hollow Stem Auger
Drilling Equipment: CME-55
SEA Representative: D. Snyder
Dates Started: 2/15/96 **Finished:** 2/17/96
Location: See Boring Location Plan, Figure A1

Ground Surface Elevation: 38.0±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	2/15	--	14.0	--	--
Completion	2/15	--	--	--	--
Casing Pulled	--	--	--	--	--
RW-12	2/17	--	12.4	--	--

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
2.5	Fine to medium silty sand FILL, trace gravel, moist - brown	FILL	35.5	A				EMBANKMENT FILL
13.0	Fine to coarse clayey sand, moist - brown	SC	25.0	C				PLEISTOCENE
23.0	Fine to medium sandy lean clay, moist - brown	CL	15.0	C				
40.0	Fine to coarse silty sand, wet - tan do, with gravel	SM	-2.0	C				Hard drilling
45.0	Gravel with fat clay pockets, moist - dark gray	GC	-7.0	E				CRETACEOUS
50.0	Fat clay, trace gravel with sand, moist - dark gray	CH	-12.0	F				
	Auger probe terminated at 50.0 ft							

Comments:
1) 6" dia. PVC relief well installed to El -11. Screen from El -11 to 29. Pump installed at El -9.
2) Wood plug broke in augers, contractor flushed augers before installing well.
3) Soil classification based on observation of auger cutting and soil on augers.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING ENGINEERS	TEST BORING LOG	BORING NO.: B-4

PROJECT: ASH DISPOSAL POND; CHESTERFIELD POWER STATION
CLIENT: VEPCO
BORING CONTRACTOR: FOUNDATION TEST SERVICE — DRILL: CME-55

SHEET NO. 1 OF 1	JOB NO.: V82481	ELEVATION 25.2±	CASING SIZE: 2¼"		
DATE START: 11/3/82	DATE FINISHED: 11/3/82	DRILLER: B. SPIERENBURG	INSPECTOR: S. COPLEY		

WATER LEVEL DATA	DATE	TIME	DEPTH	CAVED
ENCOUNTERED	11/3	12:11	29.0'	-
AFTER CASING PULLED				
– HR. READING	SEE TABLE BELOW			

DRIVE	SAMPLER
TYPE	S.S.
DIA.	2" OD
WT.	140 #
FALL	30"

STRATUM	DEPTH FT.	ELEV. 25.2±	BLOWS ON SAMPLE SPOON PER 6"	SYMBOL	IDENTIFICATION	REMARKS
A			11+17+18	S	FINE TO MEDIUM SILTY SAND, PROBABLE FILL, WITH SILTY CLAY LENSES, TRACE FINE GRAVEL, MOIST - BROWN (SM)	PROBABLE FILL
	4.0	20	5+5+6	S	FINE SILTY SAND WITH SILTY CLAY LENSES, MOIST - BROWN (SM)	
			3+4+4	S		
C		10	4+4+5	S	do, CLAYEY SILTY	PLEISTOCENE SEDIMENTS
			3+3+3	S	do, WITH GRAY LENSES, TRACE ORGANIC MATTER	
[illegible]	24.0	0	1+3+3	S	CLAYEY SILT, TRACE FINE SAND AND ORGANIC MATTER, MOIST - GRAY (ML)	Tube pressed 24" alternate hole Recovery=24"
[illegible]	29.0		76+24/5"	S	FINE TO COARSE SAND, TRACE SILT, WITH FINE TO MEDIUM GRAVEL, WET - BROWN (SM TO GM)	CRETACEOUS SEDIMENTS
	33.5					
	35.0		11+11+10	S	FINE TO COARSE SAND, SOME SILTY CLAY WITH FINE GRAVEL AND ORGANIC LENSES, MOIST - GRAY (SC)	

BORING TERMINATED AT 35.0 FT

Water Observation Well Installed to 34.2 ft

Date	Time	Elev. W.L.	Remarks
11/3	-	-	2" pvc
11/5	7:39 a.m.	1.6	-
11/10	9:19 a.m.	2.1	7 day reading

SCHNABEL ENGINEERING ASSOCIATES CONSULTING ENGINEERS	TEST BORING LOG	BORING NO.: B-5

PROJECT ASH DISPOSAL POND; CHESTERFIELD POWER STATION
CLIENT VEPCO
BORING CONTRACTOR FOUNDATION TEST SERVICE DRILL CME-55

WATER LEVEL DATA	DATE	TIME	DEPTH	CAVED
ENCOUNTERED	11/3	-	DRY	-
AFTER CASING PULLED	11/3	11:03	30.0'	32.0'
22 HR. READING	11/4	8:50	33.2'	34.2'

	DRIVE SAMPLER
TYPE	S.S.
DIA.	2" OD
WT.	140 #
FALL	30"

SHEET NO. 1 OF 1
JOB NO. V82431
ELEVATION: 35.5±
CASING SIZE: 2½"
DATE START: 11/3/82
DATE FINISHED: 11/3/82
DRILLER: B. SPIERENBURG
INSPECTOR: S. COPLEY

STRATUM	DEPTH FT.	ELEV. 35.5±	BLOWS ON SAMPLE SPOON, PER 6"	SYMBOL	IDENTIFICATION	REMARKS
A			16+19+14	S	FINE SILTY SAND, PROBABLE FILL, TRACE ORGANIC MATTER, DRY - BROWN (SM)	PROBABLE FILL
	4.0	30	8+11+13	S	FINE SILTY SAND, WITH MICA, TRACE ORGANIC MATTER, MOIST - BROWN (SM)	
			6+9+11	S	do, TRACE SILT	PLEISTOCENE SEDIMENTS
		20	7+11+12	S	do, TRACE SILT	
C			8+11+11	S	do, SOME SILT	
		10	20+33+40	S	do, WITH FINE TO MEDIUM GRAVEL	
	29.0		77+23/3"	S	FINE TO MEDIUM SAND, TRACE SILT WITH FINE TO MEDIUM GRAVEL, DRY - BROWN (SP)	
	34.0	0	33+24+24	S	FINE TO COARSE SILTY CLAYEY SAND WITH FINE TO MEDIUM GRAVEL, MOIST - TAN TO BROWN (SC)	CRETACEOUS SEDIMENTS
F	40.0		34+30+17	S	do, LIGHT GRAY	
					BORING TERMINATED AT 40.0 FT	Caved and dry at 8.7' on 11/10/92

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS

TEST BORING LOG

BORING NO: B-12 (104)
SHEET NO: 1 OF 1
JOB NO: V830894
ELEVATION: 15.0±
CASING SIZE: 2-1/4"
DATE START: 9-16-88
DATE FINISHED: 9-16-88
DRILLER: F. ELGIN
INSPECTOR: R. HARRIS

PROJECT: HENRICUS HISTORICAL PARK ACCESS RD., CHESTERFIELD CO., VA
CLIENT: COUNTY OF CHESTERFIELD, VIRGINIA
BORING CONTRACTOR: AYERS AND AYERS, INC. DRILL: CME-45

WATER LEVEL DATA	DATE	TIME	DEPTH	CAVED
ENCOUNTERED	9-16	-	DRY	-
AFTER CASING PULLED	9-16	10:03	DRY	3.5
HR. READING	BACKFILLED UPON COMPLETION			

DRIVE SAMPLER	
TYPE	S.S.
DIA.	2" O.D.
WT.	140#
FALL	30"

STRATUM	DEPTH FT.	ELEV.	BLOWS ON SAMPLE SPOON PER 6"	SYMBOL	IDENTIFICATION	REMARKS
		15.0			2" TOPSOIL	
			12+13+23	S	FINE TO MEDIUM SILTY SAND FILL, CONTAINS ROOT FRAGMENTS, MOIST - BROWN	
			18+19+16	S		
		10	3+4+4	S	do, CONTAINS MICA	FILL
A			5+4+5	S		
			1+2+3	S		
	13.5				FINE TO MEDIUM SANDY LEAN CLAY (CL), CONTAINS MICA, MOIST - GRAY	
E	14.0				FINE TO MEDIUM SILTY SAND (SM), CONTAINS MICA,	CRETACEOUS AGE SEDIMENTS
	15.0	0	1+2+11	S	MOIST - GRAY	

BORING TERMINATED AT 15.0 FT

SCHNABEL ENGINEERING ASSOCIATES
CONSULTING ENGINEERS

TEST BORING LOG

BORING NO: B-13

PROJECT: HENRICUS HISTORICAL PARK ACCESS RD., CHESTERFIELD CO., VA
CLIENT: COUNTY OF CHESTERFIELD, VIRGINIA
BORING CONTRACTOR AYERS AND AYERS, INC.
DRILL: CME-45

SHEET NO: 1 OF 1
JOB NO: V880894
ELEVATION: 36.0±
CASING SIZE: 2-1/4"
DATE START: 9-19-83
DATE FINISHED: 9-19-88
DRILLER: F. ELGIN
INSPECTOR: J. ANDERSON

WATER LEVEL DATA	DATE	TIME	DEPTH	CAVED
ENCOUNTERED	9-19	–	DRY	–
AFTER CASING PULLED	9-19	9:35	DRY	6.0
__ HR. READING	BACKFILLED UPON COMPLETION			

DRIVE SAMPLER	
TYPE	S.S.
DIA.	2" O.D.
WT.	140#
FALL	30"

STRATUM	DEPTH FT.	ELEV.	BLOWS ON SAMPLE SPOON PER 6"	SYMBOL	IDENTIFICATION	REMARKS
		36.0			6" ROOT MAT AND TOPSOIL	
			12+15+19	S	FINE TO MEDIUM SILTY SAND FILL, TRACE GRAVEL, CONTAINS CRUSHED STONE, DRY - BROWN	FILL
A	2.0					
			14+21+32	S	FINE TO MEDIUM SANDY LEAN CLAY (CL), MOIST - BROWN	
D						TERRACE DEPOSITS
			15+28+34	S	do, TRACE GRAVEL, DRY	
	6.0	30				
			7+12+15	S	FINE TO MEDIUM SILTY SAND (SM), DRY - BROWN	
C			7+11+14	S		
	15.0		11+14+19	S		

BORING TERMINATED AT 15.0 FT

BORING LOG

SINCE 1881 F&R **FROEHLING & ROBERTSON, INC.**
FULL SERVICE LABORATORIES · ENGINEERS & CHEMISTS
"OVER ONE HUNDRED YEARS OF SERVICE"

DATE: March 19, 1993

Report No.: T-55-166

Client: Virginia Power

Project: Chesterfield Ash Pond - North Dike

Boring No.: B-1 (1 of 2) | Total Depth 60.0' | Elev: 42.03' | Location: N380851.062, E2329947.570

Type of Boring: Hollow Stem Auger | Started: 1/25/93 | Completed: 1/25/93 | Driller: England

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (Feet)		REMARKS
		Medium Dense, Reddish Brown Clayey Fine SAND, Trace Gravel and Crushed Stone - Moist (DIKE FILL)	7/12/12	0.0		Groundwater was Encountered at 39'8" During Drilling
			14/12/15	1.5		
			8/10/10	3.0		
				4.5		
37.03	5.0	Medium Dense, Brown to Reddish Brown Clayey to Silty Fine to Medium SAND, Trace Fine to Medium Gravel - Moist (DIKE FILL)		6.0		Groundwater was Observed at 18'3" Upon Removal of Auger
			10/10/9	7.5		
			12/13/12	8.5		Groundwater was Observed at: 18'6" on 1/26/93 17'6" on 1/27/93 18'3" on 1/28/93 19'5" on 2/4/93
				10.0		
30.03	12.0	Medium Dense to Loose, Brown Silty Very Fine SAND, Little Clay, Trace Mica - Wet (SM)	6/9/8	13.5		
				15.0		Set Piezometer at 35'0"
						SEE ATTACHED PIEZOMETER DIAGRAM B-1
			5/5/5	18.5		
				20.0		
			2/2/3	23.5		
				25.0		
			2/3/5	28.5		
				30.0		
			7/35/28	33.5		
8.03	34.0	Very Dense, Brown Fine to Medium SAND with Fine Gravel - Wet (SM)		35.0		NOTE (1) Medium Dense, Light Gray Clayey Fine to Medium SAND - Wet (SC)
4.03	38.0	SEE NOTE (1)	5/10/15	38.5		
				40.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the last two increments of penetration is termed the standard penetration resistance, N.

BORING LOG

FROEHLING & ROBERTSON, INC.
FULL SERVICE LABORATORIES · ENGINEERS & CHEMISTS
"OVER ONE HUNDRED YEARS OF SERVICE"
SINCE 1881

DATE: March 19, 1993

Report No.: T-55-166

Client: Virginia Power

Project: Chesterfield Ash Pond - North Dike

Boring No.: B-1 (2 of 2) Total Depth 60.0' Elev: 42.03' Location: N380851.062, E2329947.570

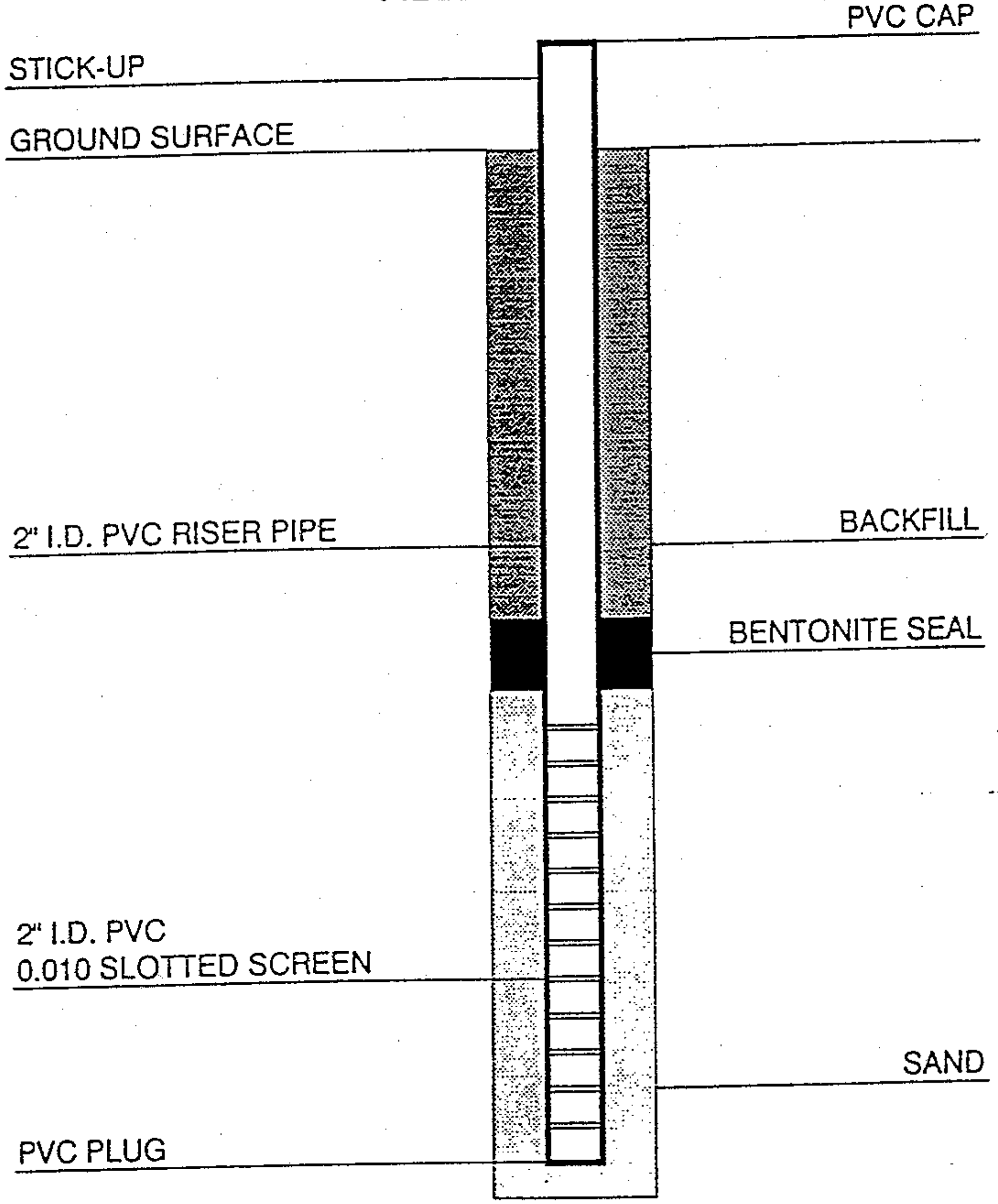
Type of Boring: Hollow Stem Auger Started: 1/25/93 Completed: 1/25/93 Driller: England

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (Feet)		REMARKS
		Medium Dense, Light Gray Clayey Fine to Medium SAND - Wet (SC)				
0.03	42.0			43.5		
		Very Dense, Tan and Gray Clayey and Silty Fine Coarse SAND and GRAVEL - (SC-SM)	21/45/*	44.8		*50/0.3'
-5.97	48.0			48.5		
		Medium Dense to Dense, Gray Silty Fine SAND - Dry (Preconsolidated) (SM)	20/7/9	50.0		
				53.5		
			10/15/18	55.0		
				58.5		
-17.97	60.0		10/12/18	60.0		
		Boring Terminated at 60.0'				

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the last two increments of penetration is termed the standard penetration resistance, N.

PIEZOMETER DIAGRAM

PIEZOMETER B-1



BACKFILL	12.0 ft.	CASING TOP EL	45.03
BENTONITE	2.0 ft.	CASING	25.0 ft.
SAND	21.0 ft.	SCREEN	10.0 ft.
TOTAL DEPTH	35.0 ft.	STICK-UP	3.0 ft.

SINCE 1881

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"OVER ONE HUNDRED YEARS OF SERVICE"

DATE: March, 1993
SCALE: NONE
DRWN: DAK | T-55-166

PIEZOMETER DIAGRAM
CHESTERFIELD ASH POND - NORTH DIKE
CHESTERFIELD COUNTY, VA VIRGINIA POWER

DWG. NO. B-1

Form No 103

BORING LOG

FROEHLING & ROBERTSON, INC.
FULL SERVICE LABORATORIES · ENGINEERS & CHEMISTS
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SINCE 1881

DATE: March 19, 1993

Report No.: T-55-166

Client: Virginia Power

Project: Chesterfield Ash Pond - North Dike

Boring No.: B-2 (1 of 2) | Total Depth 60.0' | Elev: 41.83' | Location: N380860.614, E2330043.740

Type of Boring: Hollow Stem Auger | Started: 1/26/93 | Completed: 1/26/93 | Driller: England

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (Feet)		REMARKS
		Medium Dense, Reddish Brown Fine Sandy CLAY, Trace Gravel and Crushed Stone - Moist (DIKE FILL)	10/10/13	0.0		
40.33	1.5		12/14/18	1.5		Groundwater was Encountered at 40'2" During Drilling
				3.0		
		Dense to Medium Dense, Reddish Brown Clayey Fine SAND, Trace Gravel - Moist (DIKE FILL)	7/10/15	3.5		
				5.0		Groundwater was Observed at 20'2" Upon Removal of Auger
			9/8/10	6.0		
				7.5		
33.83	8.0	Very Stiff, Reddish Brown Fine Sandy CLAY, Trace Mica - Moist (DIKE FILL)	8/10/12	8.5		Groundwater was Observed at: 18'8" at +1 Hr. 20'1" on 1/27/93 20'11" on 1/28/93 18'0" on 2/4/93
				10.0		
29.83	12.0	Medium Dense to Loose, Brown Very Silty to Silty Fine SAND, Trace Clay, Trace Mica - Moist to Wet (SM)	4/6/9	13.5		Cave-in Depth at 29'0"
				15.0		
			3/4/5	18.5		
				20.0		
			3/4/6	23.5		
				25.0		
			2/4/7	28.5		
				30.0		
9.83	32.0	Very Dense to Medium Dense, Brown Silty Fine to Medium SAND and Fine GRAVEL - Wet (SM)	14/29/33	33.5		
				35.0		
			8/13/6	38.5		
				40.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the last two increments of penetration is termed the standard penetration resistance, N.

BORING LOG

FROEHLING & ROBERTSON, INC.
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"OVER ONE HUNDRED YEARS OF SERVICE"
SINCE 1881

Report No.: T-55-166

DATE: March 19, 1993

Client: Virginia Power

Project: Chesterfield Ash Pond - North Dike

Boring No.: B-2 (2 of 2) | Total Depth 60.0' | Elev: 41.83' | Location: N380860.614, E2330043.740

Type of Boring: Hollow Stem Auger | Started: 1/26/93 | Completed: 1/26/93 | Driller: England

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (Feet)		REMARKS
		Very Dense to Medium Dense, Brown Silty Fine to Medium SAND and Fine GRAVEL - Wet (SM)	9/15/23	43.5 / 45.0		
-5.17	47.0					
		Dense, Dark Gray Silty Fine SAND - Dry (Preconsolidated) (SM)	16/18/19	48.5 / 50.0		
			8/14/18	53.5 / 55.0		
			16/20/21	58.5 / 60.0		
-18.17	60.0					
		Boring Terminated at 60.0'				

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the last two increments of penetration is termed the standard penetration resistance, N.

BORING LOG

SINCE 1881 FROEHLING & ROBERTSON, INC.
FULL SERVICE LABORATORIES · ENGINEERS & CHEMISTS
"OVER ONE HUNDRED YEARS OF SERVICE"

DATE: March 19, 1993

Report No.: T-55-166

Client: Virginia Power

Project: Chesterfield Ash Pond - North Dike

Boring No.: B-3 (1 of 1) | Total Depth 40.0' | Elev: 21.86' | Location: N380921.943, E2330002.470

Type of Boring: Hollow Stem Auger | Started: 1/27/93 | Completed: 1/27/93 | Driller: England

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)
		Medium Dense to Very Loose, Reddish Brown Clayey Fine SAND, Trace Mica and Roots to 4.5' - Moist (ROAD FILL)
9.86	12.0	Very Loose to Medium Dense, Brown Very Silty Fine SAND, Trace Mica - Wet (SM)
0.36	21.5	Medium Dense, Tan and Gray Clayey Fine to Medium SAND - Wet (SC)
-5.14	27.0	Very Dense, Brown Silty Fine to Medium SAND and GRAVEL - Wet (SM)
-9.64	31.5	Dense to Medium Dense, Dark Gray Silty Fine SAND - Dry (SM)
-18.14	40.0	Boring Terminated at 40.0'

* Sample Blows	Sample Depth (Feet)
5/5/6	0.0 - 1.5
4/4/3	1.5 - 3.0
	3.0 - 3.5
3/3/4	3.5 - 5.0
	5.0 - 6.0
3/3/1	6.0 - 7.5
	7.5 - 8.5
2/2/1	8.5 - 10.0
2/2/1	13.5 - 15.0
2/5/5	18.5 - 20.0
4/5/7	23.5 - 25.0
27/42/23	28.5 - 30.0
10/16/23	33.5 - 35.0
10/11/14	38.5 - 40.0

REMARKS

Groundwater was Encountered at 26'3" During Drilling

Groundwater was Observed at 13'3" Upon Removal of Auger

Groundwater was Observed at:
7'6" on 1/28/93
8'2" on 2/4/93

Set Piezometer at 39'0"

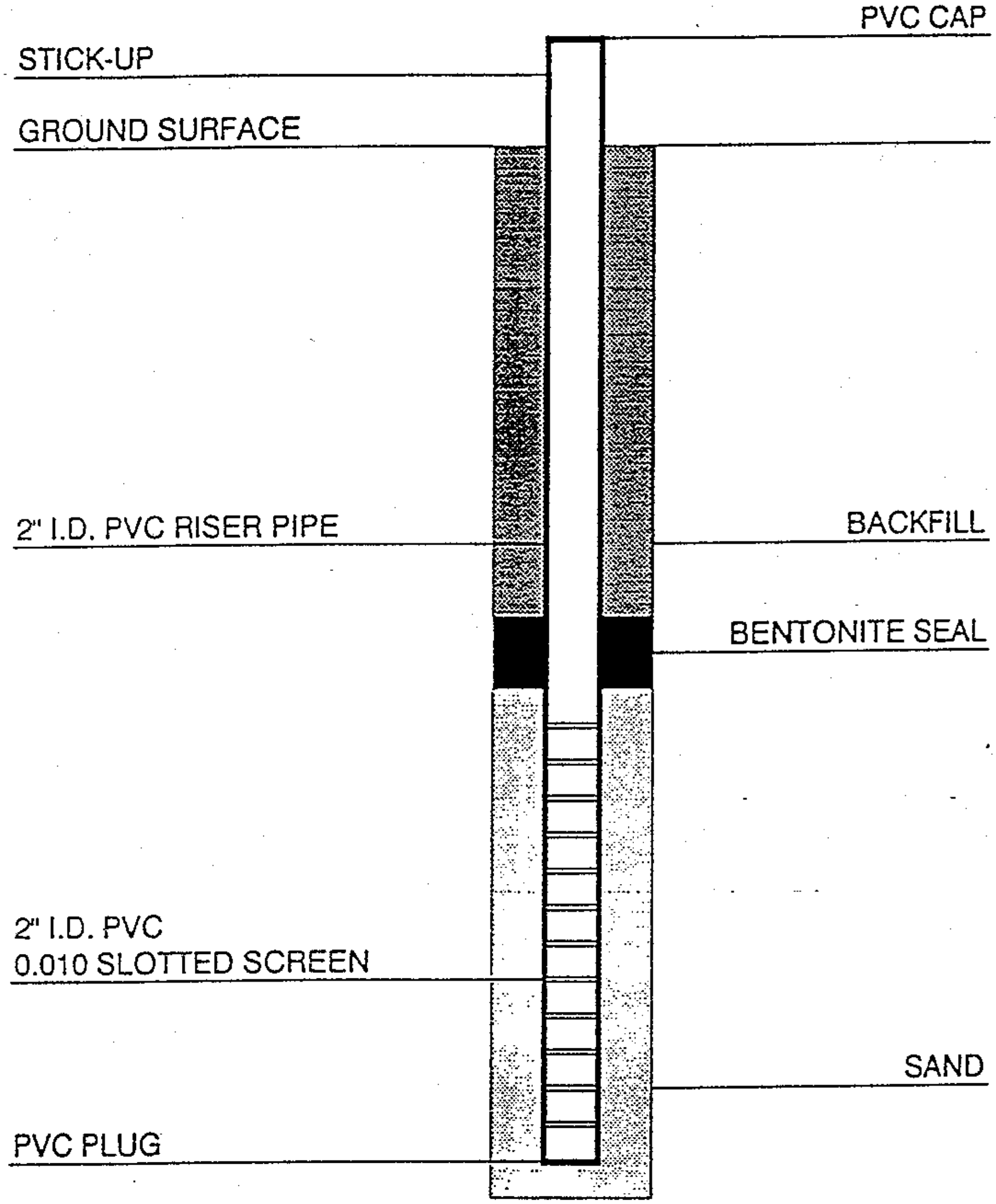
SEE ATTACHED PIEZOMETER DIAGRAM B-3

NOTE:
Undisturbed Samples Taken at Offset Boring at
4.0' to 6.0'
6.5' to 8.5'

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the last two increments of penetration is termed the standard penetration resistance, N.

PIEZOMETER DIAGRAM

PIEZOMETER B-3



BACKFILL	23.5 ft.	CASING TOP EL	22.86
BENTONITE	2.5 ft.	CASING	29.0 ft.
SAND	13.0 ft.	SCREEN	10.0 ft.
TOTAL DEPTH	39.0 ft.	STICK-UP	1.0 ft.

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DATE: March, 1993
SCALE: NONE
DRWN: DAK | T-55-166

PIEZOMETER DIAGRAM
CHESTERFIELD ASH POND - NORTH DIKE
CHESTERFIELD COUNTY, VA VIRGINIA POWER

DWG. NO. B-3

Form No 103

BORING LOG

FROEHLING & ROBERTSON, INC.
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SINCE 1881

DATE: March 19, 1993

Report No.: T-55-166

Client: Virginia Power

Project: Chesterfield Ash Pond - North Dike

Boring No.: B-4 (1 of 1) | Total Depth 35.0' | Elev: 17.92' | Location: N380923.428, E2329932.520

Type of Boring: Hollow Stem Auger | Started: 1/27/93 | Completed: 1/27/93 | Driller: England

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (Feet)	REMARKS
		Medium Dense to Loose, Reddish Brown Silty Fine SAND, Trace Mica - Moist (ROAD FILL)	7/6/8	0.0 - 1.5	Groundwater was Encountered at 13'6" During Drilling
			6/8/9	3.0	
			4/2/3	4.5	Groundwater was Observed at 6'0" Upon Removal of Auger
				6.0	
			4/2/4	7.5	
				8.5	Groundwater was Observed at: 5'6" on 1/28/93; 5'11" on 2/4/93
			2/3/4	10.0	
5.92	12.0				
		Firm to Stiff, Brown Fine Sandy Silty CLAY - Wet (CL-ML)		13.5	Cave-in Depth at 26'0"
			2/2/3	15.0	NOTE: Undisturbed Samples Taken at Offset Boring at: 6.0' to 8.0' - No Rec.; 8.0' to 10.0'; 11.0' to 13.0'; 15.0' to 17.0'
				18.5	
			3/6/8	20.0	
-4.08	22.0				
		Medium Dense, Gray to Tan and Gray Clayey Fine to Medium SAND with Fine Gravel at 28.0' - Wet (SC)		23.5	
			18/13/12	25.0	
				28.5	
			23/18/12	30.0	
				33.5	
-16.08	34.0		23/18/12	35.0	
-17.08	35.0	Medium Dense, Dark Gray Silty Fine SAND - Dry (SM)			
		Boring Terminated at 35.0'			

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the last two increments of penetration is termed the standard penetration resistance, N.

BORING LOG

FROEHLING & ROBERTSON, INC.
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SINCE 1881

Report No.: T-55-166

DATE: March 19, 1993

Client: Virginia Power

Project: Chesterfield Ash Pond - North Dike

Boring No.: B-5 (1 of 1) Total Depth 30.0' Elev: 16.10' Location: N380928.258, E2329817.690

Type of Boring: Hollow Stem Auger Started: 1/26/93 Completed: 1/26/93 Driller: England

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (Feet)	REMARKS
		Medium Dense to Very Loose, Reddish Brown Clayey Fine SAND Trace Roots at 1.5' Trace Mica at 4.5' Trace Roots at 7.5' - Dry to Wet (SC)	5/6/6 4/6/6 5/5/7 5/4/3 1/2/2	0.0 1.5 3.0 3.5 5.0 6.0 7.5 8.5 10.0	Groundwater was Encountered at 17'3" During Drilling Groundwater was Observed at 3'3" Upon Removal of Auger Groundwater was Observed at: 1'0" on 1/27/93 1'0" on 1/28/93 1'8" on 2/4/93
4.10	12.0	Soft, Gray Fine Sandy CLAY - Wet (CL)	1/2/1	13.5 15.0 18.5	Cave-in Depth at 22'4" NOTE: Undisturbed Samples Taken at Offset Boring at: 13.0' to 15.0' 18.0' to 20.0'
-2.90	19.0	Medium Dense to Dense, Tan and Gray Clayey Fine to Medium SAND and GRAVEL - Wet (SC)	2/9/15 9/21/16	20.0 23.5 25.0	
-10.90	27.0	Dense, Dark Gray Silty Fine SAND - Dry (SM)	24/15/17	28.5 30.0	
-13.90	30.0	Boring Terminated at 30.0'			

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the last two increments of penetration is termed the standard penetration resistance, N.

BORING LOG

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"OVER ONE HUNDRED YEARS OF SERVICE"

Report No.: T-55-166

DATE: March 19, 1993

Client: Virginia Power

Project: Chesterfield Ash Pond - North Dike

Boring No.: B-6 (1 of 1) | Total Depth 35.0' | Elev: 19.01' | Location: N380927.585, E2329879.450

Type of Boring: Hollow Stem Auger | Started: 1/26/93 | Completed: 1/26/93 | Driller: England

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)
7.01	12.0	Medium Dense to Very Loose, Reddish Brown Clayey Fine SAND, Trace Mica - Wet (SC)
-2.99	22.0	Very Loose to Medium Dense, Gray Silty SAND with Gravel - Wet (SM)
-14.99	34.0	Very Loose to Medium Dense, Brown to Tan and Gray Silty to Clayey Fine to Medium SAND with Fine Gravel - Wet (SM/SC)
-15.99	35.0	Dense, Dark Gray Silty Fine SAND - Dry (SM)

Boring Terminated at 35.0'

* Sample Blows	Sample Depth (Feet)
4/5/7	0.0 – 1.5
3/4/6	1.5 – 3.0
	3.0 – 3.5
6/8/7	3.5 – 5.0
	5.0 – 6.0
5/6/10	6.0 – 7.5
	7.5 – 8.5
2/2/2	8.5 – 10.0
	10.0 – 13.5
1/1/1	13.5 – 15.0
	15.0 – 18.5
3/6/6	18.5 – 20.0
	20.0 – 23.5
2/1/2	23.5 – 25.0
	25.0 – 28.5
5/7/11	28.5 – 30.0
	30.0 – 33.5
24/21/16	33.5 – 35.0

REMARKS

Groundwater was Encountered at 8'3" During Drilling

Groundwater was Observed at 16'4" Upon Removal of Auger

Groundwater was Observed at Ground Surface on 1/28/93 and on 2/4/93

Set Piezometer at 34'0"

NOTE: Undisturbed Samples Taken at Offset Boring at:
2.5' to 4.5'
10.0' to 12.0'
13.0' to 15.0'

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the last two increments of penetration is termed the standard penetration resistance, N.

BORING LOG

SINCE 1881
FROEHLING & ROBERTSON, INC.
FULL SERVICE LABORATORIES • ENGINEERS & CHEMISTS
"OVER ONE HUNDRED YEARS OF SERVICE"

Report No.: T-55-166

DATE: March 19, 1993

Client: Virginia Power

Project: Chesterfield Ash Pond - North Dike

Boring No.: B-7 (1 of 1) | Total Depth 30.0' | Elev: 16.93' | Location: N380946.652, E2330012.270

Type of Boring: Hollow Stem Auger | Started: 2/2/93 | Completed: 2/2/93 | Driller: England

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)
11.93	5.0	Firm to Soft, Brown Sandy CLAY with Roots and Organics - Moist (FILL)
4.93	12.0	Very Loose to Loose, Brown Silty Clayey SAND with Organics - Wet (SC-SM)
-5.57	22.5	Loose to Medium Dense, Tan and Gray Silty to Clayey SAND and Fine GRAVEL - Wet (SM/SC)
-13.07	30.0	Dense to Medium Dense, Dark Gray Silty Fine SAND - Dry (SM)

Boring Terminated at 30.0'

* Sample Blows	Sample Depth (Feet)
2/3/4	0.0 – 1.5
4/3/3	1.5 – 3.0
1/1/1	3.5 – 5.0
2/2/2	6.0 – 7.5
2/2/3	8.5 – 10.0
3/4/4	13.5 – 15.0
7/12/12	18.5 – 20.0
10/14/17	23.5 – 25.0
9/12/16	28.5 – 30.0

REMARKS

Groundwater was Encountered at 16'3" During Drilling

Groundwater was Observed at 3'4" Upon Removal of Auger

Groundwater was Observed at: 1'2" on 2/4/93

Cave-in Depth at 18'3"

NOTE:
Undisturbed Samples Taken at Offset Boring at:
4.0' to 6.0'
6.0' to 8.0'

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the last two increments of penetration is termed the standard penetration resistance, N.

BORING LOG

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Report No.: T-55-166

DATE: March 19, 1993

Client: Virginia Power

Project: Chesterfield Ash Pond - North Dike

Boring No.: B-8 (1 of 1) | Total Depth 25.0' | Elev: 14.82' | Location: N380947.557, E2329970.970

Type of Boring: Hollow Stem Auger | Started: 2/2/93 | Completed: 2/2/93 | Driller: England

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (Feet)	REMARKS
		Firm to Stiff, Clayey SILT with Roots, Organics, and Wood - Moist to Wet (FILL)	2/3/4	0.0	Groundwater was Encountered at 4'3" During Drilling
			5/6/6	1.5	
11.82	3.0			3.0	
		Medium Dense to Loose, Reddish Brown Silty to Clayey Fine SAND - Wet (SM/SC)	8/8/8	3.5	
				5.0	Groundwater was Observed at 0'6" Upon Removal of Auger
			2/4/4	6.0	
				7.5	
			4/3/3	8.5	Groundwater was Observed at 2'1" on 2/4/93
				10.0	
2.82	12.0				Cave-in Depth at 16'4"
		Loose, Brown to Gray Silty Fine to Medium SAND, Trace Gravel - Wet (SM)	3/3/6	13.5	
				15.0	NOTE: Undisturbed Sample Taken at Offset Boring at 7.0' to 9.0'
			3/2/6	18.5	
				20.0	
-6.18	21.0				
		Medium Dense, Dark Gray Silty Fine SAND - Dry (SM)	9/6/7	23.5	
-10.18	25.0			25.0	

Boring Terminated at 25.0'

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the last two increments of penetration is termed the standard penetration resistance, N.

BORING LOG

FROEHLING & ROBERTSON, INC.
SINCE 1881
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"OVER ONE HUNDRED YEARS OF SERVICE"

Report No.: T-55-166

DATE: March 19, 1993

Client: Virginia Power			
Project: Chesterfield Ash Pond - North Dike			
Boring No.: B-9 (1 of 1)	Total Depth 35.0'	Elev: 13.68'	Location: N380948.077, E2329922.570
Type of Boring: Hollow Stem Auger	Started: 2/2/93	Completed: 2/2/93	Driller: England

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (Feet)		REMARKS
		Loose to Medium Dense, Reddish Brown Clayey Fine SAND, Trace Mica - Wet (Trace Wood at 7.5') (FILL)	2/3/5 4/7/13 6/8/6 6/18/4	0.0 1.5 3.0 3.5 5.0 6.0 7.5		Groundwater was Encountered at 12'6" During Drilling Groundwater was Observed at Ground Surface Upon Removal of Auger
5.68	8.0	Loose, Brown Very Silty Fine SAND, Trace Mica - Wet (SM)	3/3/3	8.5 10.0		Groundwater was Observed at Ground Surface on 2/4/93
2.68	11.0	Stiff, Brown Silty CLAY, Trace Mica - Wet (CL)	1/1/12	13.5 15.0		Cave-in Depth at 14'0" NOTE: Undisturbed Sample Taken at Offset Boring at: 17.0' to 19.0'
-3.32	17.0	Medium Dense, Brown to Tan and Gray Clayey Fine to Medium SAND and Fine GRAVEL - Wet (SC)	4/7/5 9/8/8 7/12/10 2/5/10	18.5 20.0 23.5 25.0 28.5 30.0 33.5 35.0		
-20.32	34.0	Medium Dense, Dark Gray Silty Fine SAND - Dry (SM)				
-21.32	35.0	Boring Terminated at 35.0'				

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the last two increments of penetration is termed the standard penetration resistance, N.

SCHNABEL ENGINEERING ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS TEST BORING LOG	Project: HENRICUS PARK ACCESS ROAD COXENDALE ROAD CHESTERFIELD COUNTY, VIRGINIA	Contract Number: 933234 Boring Number: SI-1 Sheet: 1 Of 1

Boring Contractor: AYERS AND AYERS, INC.
POWHATAN, VIRGINIA
Boring Foreman: F. ELGIN
Drilling Method: 3¼" HOLLOW STEM AUGER
Drilling Equipment: CME-45B (ATV)

SEA Representative: T. WOODALL

Dates Started: 08/19/94 Completed: 08/19/94

Location: SEE TEST BORING LOCATION PLAN, FIGURE A1

Ground Surface Elevation: 9.3 ±

Groundwater Observations

	Date	Time	Depth	Casing	Caved
Encountered	8-19	9:32	9.0	-	-
Completion	8-19	10:45	-	-	-
Casing Pulled	8-19	11:30	-	-	-

DEPTH (FT.)	STRATA DESCRIPTION	CLASS.	ELEV. (FT.)	STRATUM	SAMPLING DEPTH	SAMPLING DATA	W (%)	REMARKS
2.0	FINE TO MEDIUM POORLY GRADED SAND WITH SILT FILL, MOIST - BROWN	SP-SM	7.3	A		2+2+3		FILL
7.0	FINE TO MEDIUM SILTY SAND FILL, CONTAINS ROOT FRAGMENTS, MOIST - BROWN	SM	2.3		5	2+3+2 1+1+1		
14.0	FINE TO MEDIUM SILTY SAND, CONTAINS WOOD FRAGMENTS, MOIST - GRAY do, WET BELOW 9.0 FT do, TRACE GRAVEL BELOW 14.0 FT	SM	-4.7	B	10	1+1+1 1+1+1		RECENT ALLUVIUM
29.0	FINE TO MEDIUM SILTY SAND, TRACE GRAVEL, WET - BROWN AND GRAY	SM	-19.7	C	15 20 25	2+4+3 1+1+1 WOH+2+3		PLEISTOCENE
29.5	FINE TO MEDIUM CLAYEY SAND, MOIST - GRAY	SC	-20.2	E	30	4+7+8		CRETACEOUS
30.5	FINE SANDY LEAN CLAY, MOIST - GRAY	CL	-21.2	F				
	BOTTOM OF BORING @ 30.5 FT.							

Comments:
1) SLOPE INCLINOMETER INSTALLED TO 30.5 FT