

INITIAL SAFETY FACTOR ASSESSMENT

40 CFR 257.73 (e)

Bottom Ash Pond Complex

Kanawha River Site

Glasgow, WV

May, 2026

Prepared for: Appalachian Power Company

Prepared by: American Electric Power Service Corporation

1 Riverside Plaza

Columbus, OH 43215

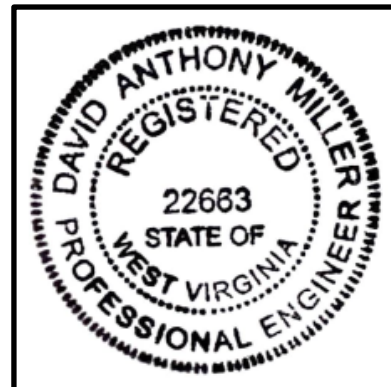


Kanawha River Site
Bottom Ash Pond Complex
Initial Safety Factor Assessment

PREPARED BY _____ DATE _____
Dan Murphy, P.E.

REVIEWED BY _____ DATE _____
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APPROVED BY David Anthony Miller DATE 04.23.2026
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Director- Ash Management Services



I certify to the best of my knowledge, information, and belief that the information contained in this safety factor assessment meets the requirements of 40 CFR § 257.73(e)

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1.0 OBJECTIVE

The “Hazardous and Solid Waste Management System: Disposal of Coal Combustion Residuals From Electric Utilities; Legacy CCR Surface Impoundments”, 89 Fed. Reg. 38950 (May 8, 2024) (amending 40 C.F.R. §257) requires owners and operators of facilities with a legacy coal combustion residual (CCR) surface impoundment to prepare an initial safety factor assessment document for each legacy CCR surface impoundment at the facility.

The Bottom Ash Pond Complex at the Kanawha River Site is subjected to this rule.

2.0 DESCRIPTION OF THE CCR UNIT

The Kanawha River Site is located approximately 0.5 miles southeast of Glasgow, West Virginia. The latitude/longitude of the Bottom Ash Pond Complex is: 38° 12' 33.34"N / 81° 25' 25.52"W. Construction of the Kanawha River Plant began in October of 1950 and the plant reached commercial operation in August of 1953. The Kanawha River Plant ceased operating in May 2015.

The BAP consists of the North Bottom Ash Pond, the South Bottom Ash Ponds, and the Clearwater Pond. The site is bounded by the Kanawha River on the west, the town of Glasgow on the North, and the railroad on the east.

The BAP is generally an incised surface impoundment with only the west portion of the Clearwater Pond contained by an 8-foot-tall dike built up above original grade. The water level of the Clearwater Pond is operated and maintained below the exterior toe. The exterior slope of the western Clearwater Pond dike is 2 Horizontal to 1 Vertical (2H: 1V) with interior slopes of 2.5H: 1V. The remainder of the pond complex is incised, with a surrounding ground elevation of 623 ft-msl. This grading scheme is a function of the powerplant grades being elevated for flood protection to elevation 623 ft-msl and the BAP area being a main borrow area, and the proximity of the western property line to the Clearwater Pond.

The BAP discharges through a concrete riser in the northwest corner of the Clearwater Pond (Outfall 001) that ties into a stormwater sewer which flows under the BAP.

3.0 SAFETY FACTOR ASSESSMENT 257.73(e)

The Initial Safety Factor Assessment was prepared by WSP USA, Inc. and is included as Attachment A.

The most critical failure surfaces of the perimeter dike of the Kanawha River Bottom Ash Pond Complex meet the required FS values. Therefore, it is concluded that the Kanawha River Bottom Ash Pond complex are stable and meet the stability FS required by 40 CFR §257.73(e).

ATTACHMENT A

Initial Safety Factor Assessment Report



REPORT

Initial Safety Factor Assessment

Bottom Ash Pond, Former Kanawha River Power Plant, Glasgow, West Virginia

Submitted to:

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April 3, 2026



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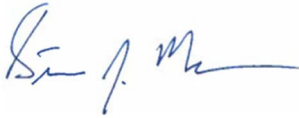
Slope Stability Analyses Outputs

CERTIFICATION

Professional Engineer Certification Statement [40 CFR 257.73(e)]

I hereby certify that, having reviewed the attached documentation and being familiar with the provisions of Title 40 of the Code of Federal Regulations Section 257.73 (40 CFR Part 257.73(e)), I attest that this Safety Factor Assessment Report is accurate and has been prepared in accordance with good engineering practices, including the consideration of applicable industry standards, and with the requirements of 40 CFR Part 257.73(e) - Initial and Periodic Safety Factor Assessments.

WSP USA Inc.



Signature

April 3, 2026

Date of Report Certification

Steven J. Moeller, PE (AL, GA, NC, SC, TN, WV)

Name

27354

Professional Engineering Certification Number



1 INTRODUCTION

WSP USA Inc. (WSP) was retained by Appalachian Power Company, a subsidiary of American Electric Power Company, Inc. (AEP), to perform this initial Safety Factor Assessment for the Bottom Ash Pond (BAP) complex, an inactive Coal Combustion Residuals (CCR) surface impoundment at the former Kanawha River Power Station. This assessment has been prepared in accordance with the requirements of 40 Code of Federal Regulations (CFR) Part §257, Subpart D (CCR Rule), specifically §257.73(e).

1.1 Purpose and Regulatory Basis

The objective of this assessment is to evaluate the stability of the Kanawha River BAP complex under various loading conditions and confirm compliance with the minimum safety factors required by the CCR Rule. Per 40 CFR §257.73(e), owners and operators of CCR surface impoundments must demonstrate that the unit meets specified safety factors for static, seismic, and liquefaction conditions.

1.2 Site Description and History

The former Kanawha River Power Station, owned and operated by the Appalachian Power Company, is located along West Virginia State Route 60, adjacent to the Kanawha River (Figure 1). Construction of the Kanawha River Plant began in October 1950, and the plant reached commercial operation in August 1953. The Kanawha River Plant ceased operating in May 2015. The BAP complex consists of the North BAP, the South BAP, and the Clearwater Pond. The site covers approximately 8 acres and is bounded by a former coal pile on the west, the town of Glasgow on the north, and the railroad on the east.

1.3 Scope of Assessment

Previous field investigations were performed to characterize the subsurface conditions at the former Kanawha River Power Station, including areas adjacent to the BAP complex. The results of the field programs are presented in the Geotechnical Data Report (GDR) (WSP 2026).

This safety assessment report provides:

- Field data from previous investigations,
- Slope stability analyses under multiple loading conditions, and
- Model outputs and calculated safety factors for the critical cross section of the BAP complex.

2 BACKGROUND

The former Kanawha River Power Station utilized multiple surface impoundments, the Eastern Fly Ash (EFA) Pond and the BAP complex, to manage sluiced ash. This Safety Factor Assessment evaluates the stability of the BAP complex, which is classified as a legacy CCR impoundment under the United States Environmental Protection Agency's (USEPA) Legacy CCR Final Rule (May 28, 2024). This rule established regulatory requirements for legacy CCR surface impoundments and CCR management units.

The area of the BAP complex is approximately 8 acres and is bounded by a former coal pile on the west, the town of Glasgow on the North, and the railroad on the east. The BAP complex consists of the North BAP, the South BAP, and the Clearwater Pond (see Figure 2). The BAP complex is generally a below-grade facility with only the west portion of the Clearwater Pond contained by a dike. The water level of the complex is operated and maintained below adjacent natural ground levels.

Based on the historical drawings (APPENDIX A), the exterior slope of the western Clearwater Pond dike is approximately 2 Horizontal to 1 Vertical (2H: 1V) with interior slopes of 2.5H: 1V. The remainder of the pond complex is incised, with a surrounding ground elevation of 623 feet above mean sea level (feet MSL). The BAP complex was originally constructed around 1951. The pond operated as a single pool of water, without any splitter dikes or treatment cells, for approximately 16 years. Around 1977, the BAP complex was modified from its original configuration to the present configuration of 3 treatment cells - the North BAP, the South BAP, and the Clearwater Pond. As part of this modification, the pond bottom was raised from 593 feet MSL to 602 feet MSL using bottom ash material. The North and South BAPs apparently had a polyvinyl chloride (PVC) liner installed at an elevation of 597 feet MSL. The interior splitter dike is approximately 960 feet long and constructed of bottom ash, with a top elevation ranging from 620 to 622 feet MSL. The western dike of the Clearwater Pond is approximately 230 feet long with a top elevation of 623 feet MSL and appears to be constructed of cohesive material from the original plant excavation. The BAP complex discharges through concrete risers located within the North BAP and Clearwater Pond that convey water to the South BAP riser structure, which discharges through a 36-inch concrete pipe into the Kanawha River.

3 SAFETY FACTOR ASSESSMENT REQUIREMENTS

In accordance with § 257.73(e)(1), the owner or operator of a CCR surface impoundment must conduct initial and periodic safety factor assessments and document whether the calculated factors of safety (FoS) achieve the minimum safety factors specified for the critical cross-section of the embankment. The critical cross-section is the cross-section anticipated to be the most susceptible of all cross-sections to structural failure based on appropriate engineering considerations, including loading conditions. The safety factor assessments must be supported by appropriate engineering calculations. The minimum safety factors specified in §257.73(e)(1)(i) through(iv) include:

- The calculated static FoS under the long-term, maximum storage pool loading condition must equal or exceed 1.50;
- The calculated static FoS under the maximum surcharge pool loading condition must equal or exceed 1.40;
- The calculated seismic FoS must equal or exceed 1.00; and
- For dikes constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20.

The owner or operator of the CCR unit must obtain a certification from a qualified professional engineer stating that the initial assessment specified in §257.73(e)(1) meets the requirements of this section.

4 SAFETY FACTOR ASSESSMENT

Slope stability analyses of the BAP complex were conducted to determine whether the calculated FoS for the critical cross-section of the perimeter dike meets or exceeds the minimum safety factors specified in Section 3.

4.1 Methodology

The stability assessment was conducted using the General Limit Equilibrium (GLE) method implemented in the computer program SLIDE2 (Version 9.026). Specifically, the Morgenstern-Price method (1965) was applied to evaluate potential failure surfaces associated with the selected cross-section. The FoS is calculated by dividing the resisting forces by the driving forces along the critical slip surface.

The stability analyses were performed for one cross-section (Section A-A) of the BAP complex dike, as shown in Figure 2. This cross-section was selected as the most susceptible to structural failure based on appropriate engineering considerations, including height and loading conditions. Material properties of the pond dike were developed from the GDR, and the slope geometry was developed based on the historical drawings (APPENDIX A) and the United States Geological Survey (USGS) topographic survey performed in 2018 (Figure 2). The submerged slope below the normal pool elevation of 613 feet MSL was developed based on a review of the available historical drawings (APPENDIX A). Previous subsurface investigations, analyses, and reports included in the GDR were used to determine the material strength properties, presented in Table 1, for the subsurface stratum at the BAP complex. The calculation package presenting the selection of geotechnical material strength properties for the slope stability analyses is provided as APPENDIX B.

The existing dike primarily consists of recompacted Earthfill comprised of sandy and silty alluvial soils. The exterior slopes of the dike are generally uniform across the site, with the slopes of the more critical section at approximately 2.3H:1V based on the available topography. The maximum height of the dike is approximately 7 feet. The four loading scenarios required by the CCR rule are discussed in the following section.

4.2 Loading Scenarios

In accordance with 40 CFR §257.73(e), slope stability analyses were performed for the following four loading conditions required by the CCR Rule:

- 1) Long-Term Maximum Storage Pool Condition: Represents the steady-state condition under the maximum water level historically maintained in the impoundment.
- 2) Maximum Surcharge Pool Condition: Simulates the temporary rise in water level due to short-term events such as storm inflow or operational surcharging.
- 3) Seismic Condition: Evaluates stability under earthquake loading using the site-specific peak ground acceleration (PGA) in accordance with regulatory requirements.
- 4) Liquefaction Condition: Assesses the potential for loss of strength in susceptible soils under seismic loading, considering effective stress and cyclic resistance.

4.2.1 Long-Term Maximum Storage Pool Condition

The long-term maximum storage pool condition represents the steady-state condition corresponding to the highest water surface elevation historically maintained within the BAP complex during normal operations. Based on a review of available historical drawings (APPENDIX A), the normal operating pool elevation for the BAP complex is at an approximate elevation of 613 feet MSL, which is the elevation at which it has been historically maintained.

For the purposes of the long-term static stability analyses, the ponded water level within the BAP complex was assumed to be at an elevation of 613 feet MSL. This pool elevation was used to define the submerged slope geometry and internal water loading conditions acting on the dike under steady-state operation. The assumed pool level is below the crest of the perimeter dike and below the surrounding natural ground surface, and it represents the maximum sustained water level expected under normal operating conditions, exclusive of temporary surcharge events.

The groundwater conditions beneath and adjacent to the BAP complex were defined independently of the surface pool elevation. The long-term groundwater table was established based on the site-specific potentiometric surface

map, which indicates groundwater at an approximate elevation of 590 feet MSL in the vicinity of the dike. This potentiometric surface was used to define the steady-state phreatic surface within the embankment and foundation soils for the long-term stability analyses.

Accordingly, the long-term maximum storage pool condition evaluated herein reflects a maintained pond water elevation of 613 feet MSL combined with a steady-state groundwater condition corresponding to a potentiometric surface at approximately 590 feet MSL, consistent with historical operation of the BAP complex and observed site groundwater conditions.

4.2.2 Maximum Surge Pool Condition

The maximum surge pool condition represents a temporary, short-term increase in surface water elevation above the normal operating pool level due to storm inflow or other transient conditions. For the BAP complex, the normal operating pool elevation is maintained at an elevation of 613 feet MSL as described in Section 4.2.1.

For purposes of evaluating the maximum surge pool condition, a temporary surge pool elevation of 615.8 feet MSL was selected. This surge elevation was evaluated in the context of an extreme precipitation event. A 100-year rainfall event of 5.17 inches over 24 hours at the BAP complex was conceptually assessed to estimate the potential short-term increase in pond water elevation. The volume of inflow into the BAP complex associated with the 100-year rainfall event from direct precipitation into the ponds and from runoff from the surrounding area corresponds to an equivalent increase in pond water elevation of approximately 2.8 feet.

Under this loading condition, the groundwater conditions within the embankment and foundation soils were assumed to remain consistent with the long-term steady-state groundwater conditions defined by the site-specific potentiometric surface at an approximate elevation of 590 feet MSL. This assumption is appropriate given the transient nature of the surge condition and the limited time available for groundwater levels to equilibrate with the elevated pond water surface.

4.2.3 Seismic Condition (Pseudo-static Stability Analysis)

The pseudo-static slope stability analysis was performed using the Hynes-Griffin and Franklin (1984) procedure, which recommends applying a horizontal seismic coefficient equal to 50% of the expected Peak Ground Acceleration (PGA). This approach accounts for the fact that the full PGA is not sustained during an earthquake and provides a conservative estimate of seismic loading for stability evaluations.

For this site, the design earthquake PGA was determined to be 0.153 g based on the site-specific seismic hazard analysis. Applying the Hynes-Griffin and Franklin recommendation:

$$k_h = 0.5 \times PGA = 0.5 \times 0.153 \text{ g} = 0.765 \approx 0.077 \text{ g}$$

Thus, the pseudo-static condition was modelled using a k_h of 0.077g, representing half the PGA. This value is consistent with industry practice and regulatory guidance for CCR impoundments and provides a reasonable balance between conservatism and realistic seismic loadings.

4.2.4 Liquefaction Condition (Post-Earthquake Liquefaction)

Liquefaction susceptibility of the dike materials was evaluated using the procedure outlined by Youd et al. (2001), based on Standard Penetration Test (SPT) data from the borings adjacent to the BAP complex, obtained during previous geotechnical exploration programs (GDR). FoS against liquefaction was calculated for all borings. The analysis incorporated earthquake magnitude and peak horizontal acceleration as input parameters to characterize the intensity and duration of ground shaking.

The applicable ground motion parameter, peak horizontal acceleration at bedrock, was taken as $a_{max} = 0.153g$, consistent with the pseudo-static stability assessment. The earthquake magnitude was determined using the Unified Hazard Tool provided by the USGS Earthquake Hazards Program (USGS, 2023). An event with a moment magnitude of $M_w = 5.96$, corresponding to a 2% probability of exceedance in 50 years (2,475-year return period), was selected for the liquefaction assessment. Details of the calculated Cyclic Stress Ratio (CSR) and Cyclic Resistance Ratio (CRR) values are provided in APPENDIX C.

Based on the evaluation, the dike materials are not susceptible to liquefaction under the design seismic conditions. Consequently, post-liquefaction stability analyses were not required and were not performed.

5 SLOPE STABILITY ASSESSMENT RESULTS

Table 2 summarizes the results of the slope stability analyses for the dike section evaluated in accordance with 40 CFR §257.73(e)(1)(i)–(iv) of the CCR Rule. Stability analyses were conducted for the long-term maximum storage pool condition, the maximum surcharge pool condition, and the pseudo-static seismic loading condition.

For the long-term static condition, the calculated FoS for both local and global failure mechanisms exceed the CCR Rule minimum requirement of $FoS \geq 1.50$. Under the maximum surcharge pool condition, which assumes a conservative 2.8-foot temporary increase in pool elevation due to a 100-year rainfall event, the calculated FoS exceeds the CCR Rule minimum requirement of $FoS \geq 1.40$.

For the pseudo-static seismic loading condition, the calculated FoS exceeds the CCR Rule minimum requirement of $FoS \geq 1.00$ for both local and global failure surfaces. Detailed stability figures for each loading condition are provided in APPENDIX D.

Based on the analysis presented in this report, the dike for the BAP complex at the former Kanawha River Power Plant meets the required factors of safety as required by 40 CFR §257.73(e)(1)(i) through (iv) for all load cases considered.

6 REFERENCES

Hynes-Griffin, M. E., & Franklin, A. G. (1984). Rationalizing the seismic coefficient method. U.S. Army Corps of Engineers, Waterways Experiment Station.

Morgenstern, N. R., & Price, V. E. (1965). The analysis of the stability of general slip surfaces. *Géotechnique*, 15(1), 79–93.

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WSP USA Inc. (WSP). (2026). Geotechnical Data Report. Kanawha River Ash Ponds (BAP and EFA), Former Kanawha River Power Plant, Glasgow, West Virginia. January 28, 2026

Youd, T. L., Idriss, I. M., Andrus, R. D., et al. (2001). Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 127(10), 817–833.

Signature Page

WSP USA Inc.



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Senior Vice President

NG/SM/ng

Tables

Table 1: Summary of Geotechnical Material Properties

Material	Unit Weight (pcf)	Drained Strength		Undrained Strength	
		Effective Internal Friction ϕ' (°)	Effective Cohesion c' (psf)	Total Internal Friction ϕ' (°)	Total Cohesion c' (psf)
Embankment Materials					
Earthfill	125	32	100	-	-
Coal Combustion Residuals					
CCR-Impounded	100	28	0	-	-
Alluvium					
Alluvium-1	120	32	100	0	26
Alluvium-2	115	32	0	0	26

Table 2: Summary of Slope Stability Assessment Results

Material	Section	Long-term static (CCR Rule Threshold FoS ≥ 1.50)	Maximum Surcharge Pool Condition (CCR Rule Threshold FoS ≥ 1.40)	Pseudo-static seismic loading (CCR Rule Threshold FoS ≥ 1.00)
Local¹	A-A	3.04	3.04	2.49
Global²	A-A	4.84	4.84	3.63

Notes:

1. For Local Slope Failure Surfaces, the minimum depth is set to 5 feet to capture shallow failures near the crest or slope face without including very thin surface slides.

2. For Global Slope Failure Surfaces, the minimum depth is set to 15 feet to focus on deeper, more critical failure mechanisms that could impact overall embankment stability

Figures



LEGEND

- EXISTING CONTOURS
- EXISTING ROADS
- EXISTING TREE LINE
- APPROXIMATE BOTTOM ASH POND BOUNDARY
- AEP MONITORING WELL (MW-XX), 1988, 1989, 1995
- EHS MONITORING WELL (MW-16XX), 2016
- EHS MONITORING WELL (MW-25XX), 2025
- AEP BORING (FA-B-XX), 1988/1989
- AEP BORING (FA-B-XX), 2005
- AEP BORING (B-14-XX), 2014
- AEP BORING (B-XX)
- AEP BORING (FR-B-XX)
- AEP CPT SOUNDING (FA-B-14), 2014



DATE	NO.	DESCRIPTION	APPD.

REVISIONS

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APPALACHIAN POWER COMPANY
BOTTOM ASH POND
FORMER KANAWHA RIVER POWER PLANT
 GLASGOW WEST VIRGINIA

SLOPE STABILITY SECTION
LOCATION PLAN

UNIT: FEET	DRAWING NUMBER: 2 OF 2	REV: -
SCALE: 1" = 200'		
DR: CCP		APPROVED BY
CH: NG		
SUP: NG		
ENG: SJM		
DATE: 01/05/2026		
		AEP SERVICE CORP. 1 RIVERSIDE PLAZA COLUMBUS, OH 43215

- NOTES**
- SOME OF THE MONITORING WELLS (FROM 1988/1989/1995) MAY HAVE BEEN ABANDONED.

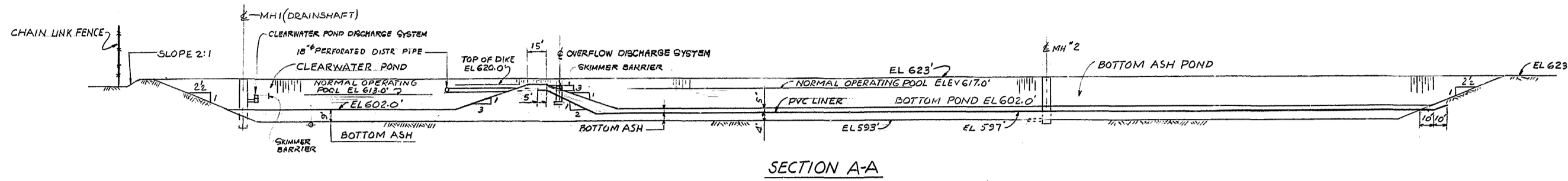
- REFERENCES**
- EXISTING TOPOGRAPHY IS 2018 USGS TOPO.
 - BOREHOLE LOCATIONS ARE EITHER PROVIDED BY AEP OR INFERRED FROM THE DRAWINGS SUPPLIED BY AEP.
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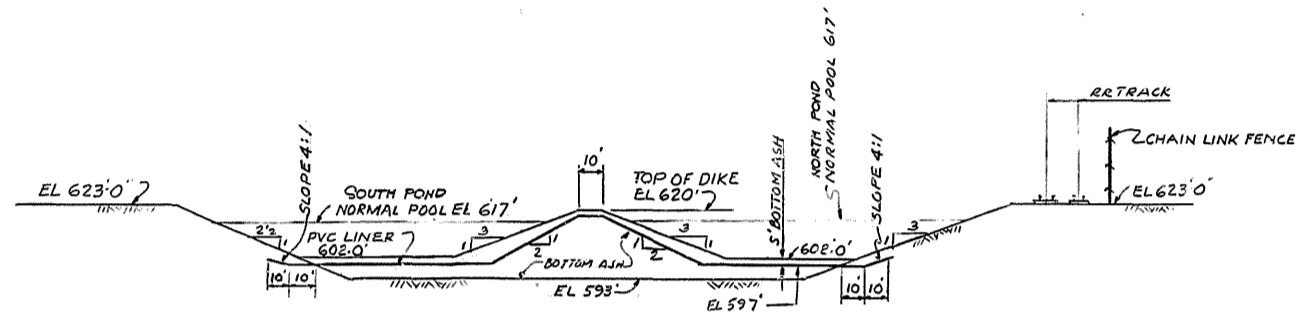
G:\American Electric Power\Kanawha River Site\Safety\Factor Assessment\Report\BAP\Production\2_Slope Stability Section\Location Plan.dwg LAYOUT: 2 SAV: 15/2026 10:29 AM PAGES: 2 OF 2 PLOTTED: 15/2026 POWELL, CHRIS

APPENDIX A

Historical Drawings



SECTION A-A



SECTION B-B

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REVISIONS			

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APPALACHIAN POWER CO.
KANAWHA RIVER PLANT
GLASGOW WEST VA.

BOTTOM ASH STORAGE AREA
MODIFICATION
SECTIONS

DR. NO. 12-11341-3851

ARCH.	ELEC.	MECH.	STR.
SCALE: 3/4" = 1'	ENGINEERING DIV.		
DR. LEN	DESIGN DIV.		
CH.	DATE: 4/11/77		
SO. LDR. M.E.	AMERICAN ELECTRIC POWER SERVICE CORP.		

APPENDIX B

Geotechnical Strength Selection



APPENDIX C: GEOTECHNICAL STRENGTH SELECTION

DATE April, 2026
TO AEP
FROM WSP

Project No. US0041868.3860

1 INTRODUCTION

The following calculation package provides an assessment of the soil strength data collected for the former Kanawha River Power Plant to assess the strength of the soil materials to be evaluated in the factor of safety assessment. Coal Combustion Residuals (CCRs), underlying alluvial soils, and embankment fill material were evaluated in this analysis. A discussion of the available data is provided along with the proposed selected material strength properties for each stratum.

2 INVESTIGATIONS AND LABORATORY TESTING

Subsurface investigations at the EFA Pond and BAP sites have been conducted over multiple phases to characterize site conditions and support closure planning. The Geotechnical Data Report (GDR) discusses the historic explorations and lab testing conducted for each site. Laboratory testing performed on relatively undisturbed samples collected with Shelby tubes included Consolidated-Undrained (CU with pore pressures) Triaxial Compression Tests (ASTM D4767), Unit Weight (ASTM D7263), Natural Moisture Content (ASTM D2216), and Atterberg Limits (ASTM D4318). Additionally, limited direct shear analyses were performed (ASTM D3080).

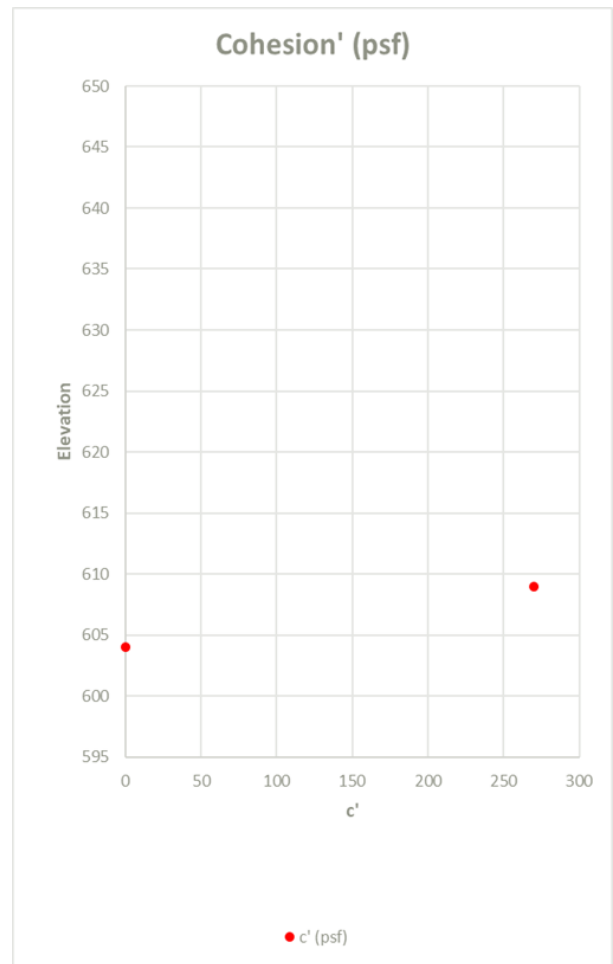
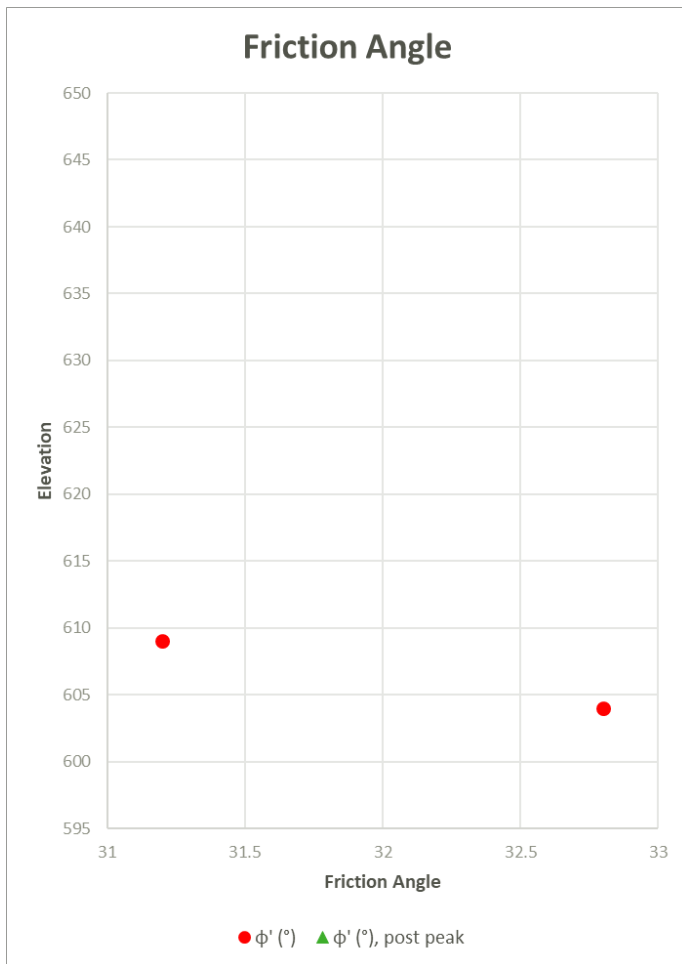
3 STRENGTH PARAMETER SELECTION

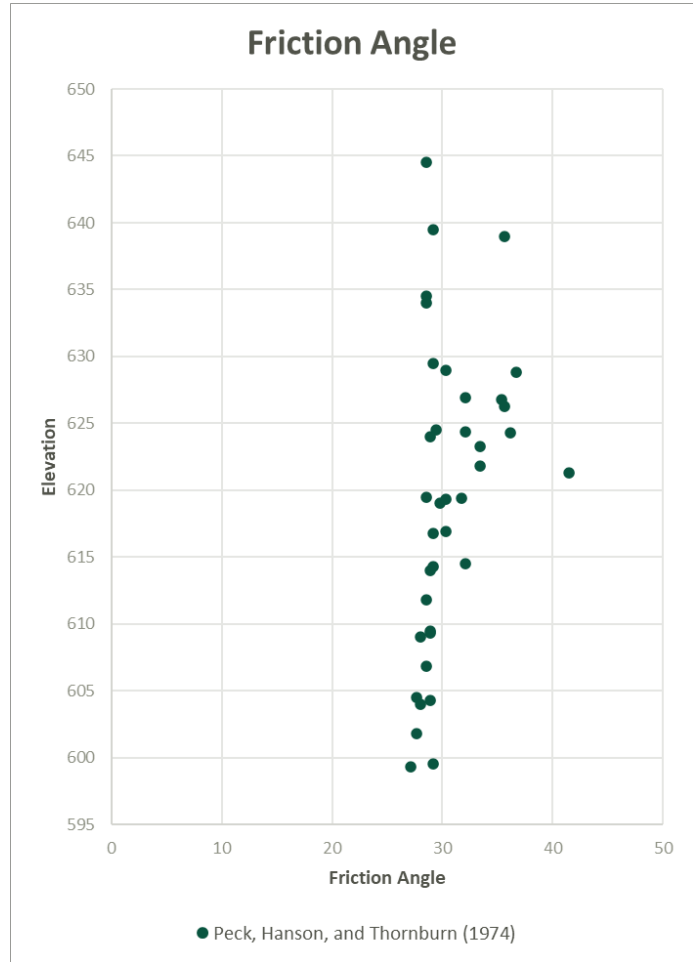
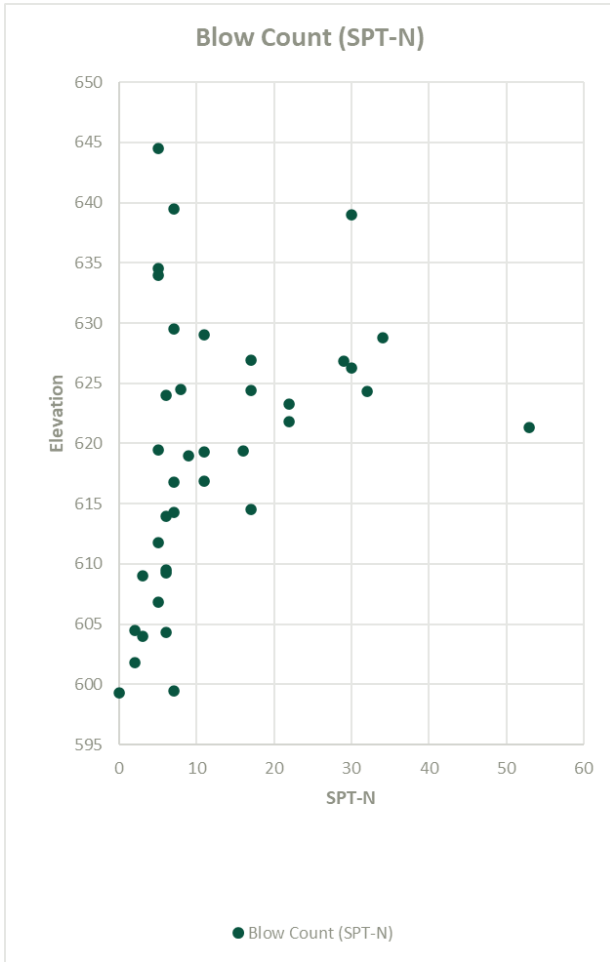
The data collected in the field program for the various soil strata were plotted and evaluated to determine representative strength properties for each soil material type. The following data were used in the analysis. Laboratory Strength Parameters: Data from laboratory testing were used to estimate cohesion (c) and friction angle (ϕ) for each material type.

SPT Correlation and Strength Parameters: SPT values were correlated with shear strength parameters using established empirical relationships commonly applied in geotechnical engineering practice. These correlations consider soil type, relative density, and consistency, and were used to estimate cohesion (c) and friction angle (ϕ). The calculations followed procedures outlined in ASTM D1586 and supplemented by published correlations (Peck, Hanson & Thornburn, 1974).

3.1 STRENGTH PARAMETERS - CCR IMPOUNDMENT MATERIALS

From previous analysis at the site, the triaxial compression tests performed on the samples of fly ash yielded effective shear strengths with ϕ ranging from 30.5° to 32.8° and apparent cohesions ranging from 0 to 270 psf. Previous slope stability calculations had used a ϕ angle of 31 degrees and an apparent cohesion of 0 psf. The following plots show the laboratory strength data and the correlated strength data from SPT values.

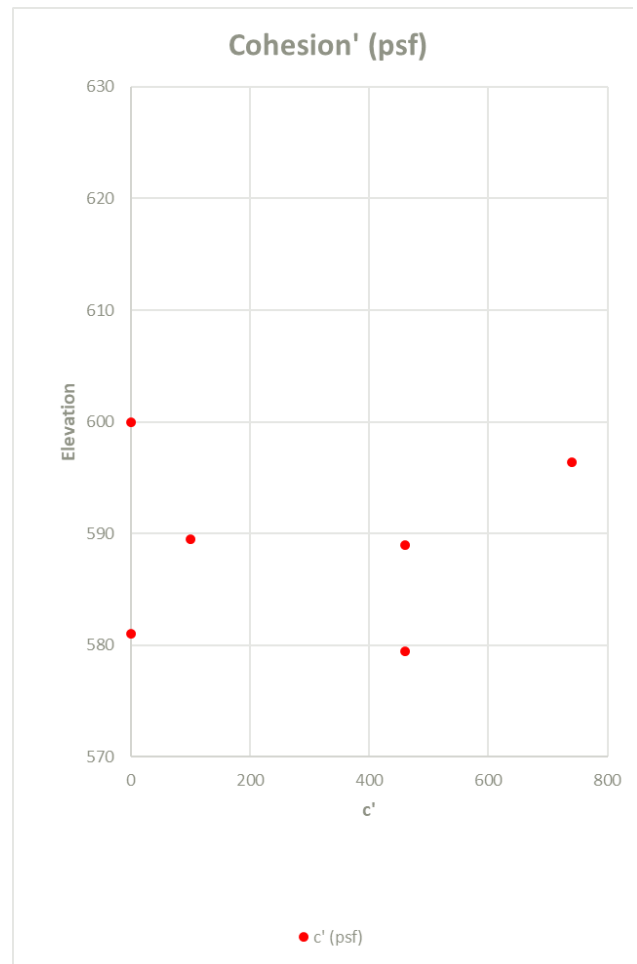
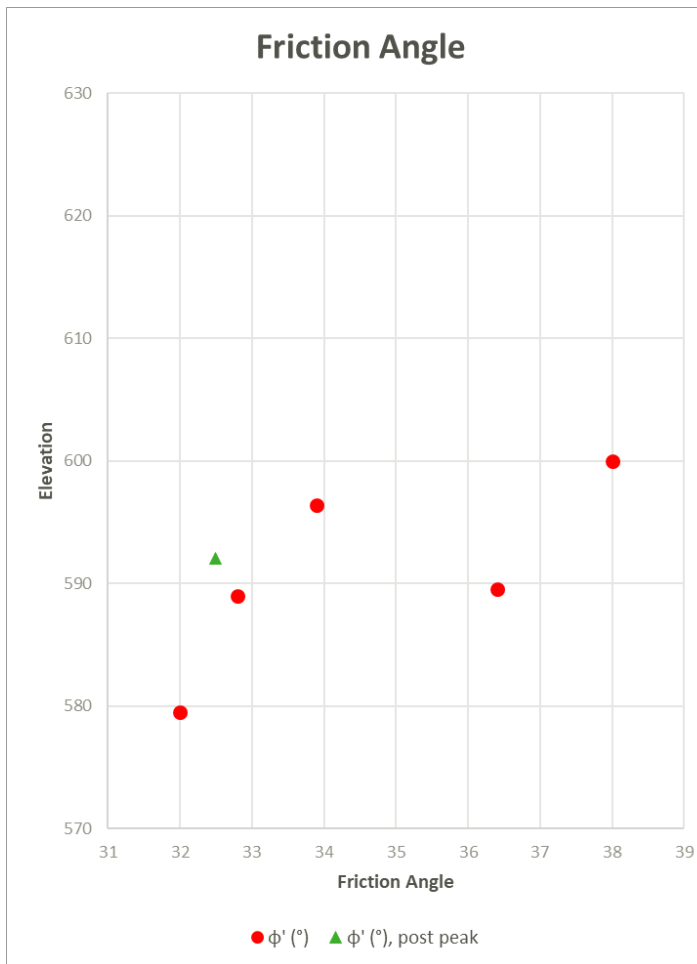


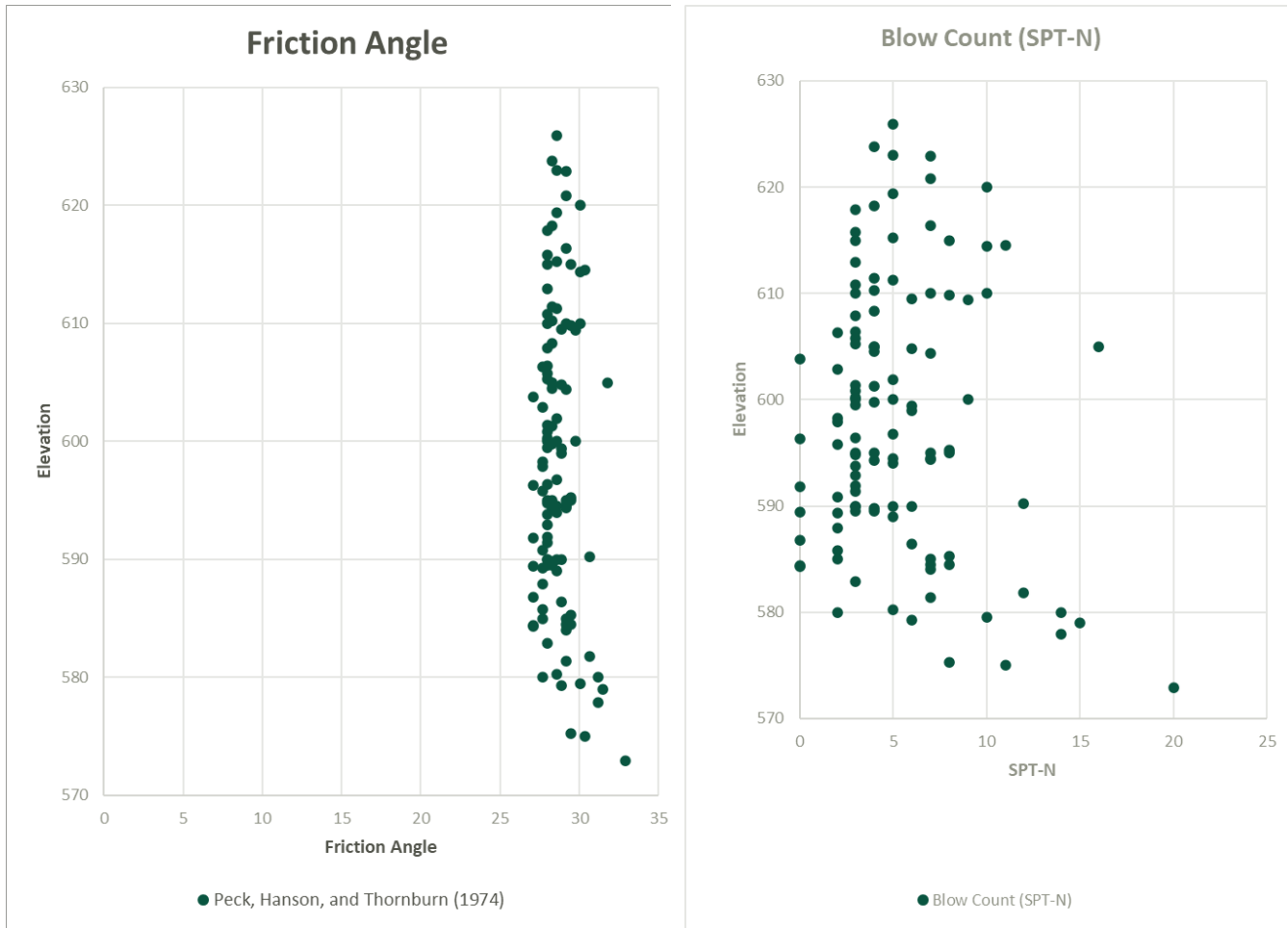


Based on an evaluation of the data, WSP's experience at other sites, and a comparison of data for dry, previously sluiced CCR materials, WSP selected representative strength parameters for the CCR materials: ϕ angle of 28 degrees and apparent cohesion of 0 psf.

3.2 STRENGTH PARAMETERS - ALLUVIUM 1

From previous analysis at the site, the triaxial compression tests performed on the samples of alluvial soils from 2005 yielded effective shear strengths with ϕ ranging from 32° to 36.4° and apparent cohesions ranging from 100 to 460 psf. Previous slope stability calculations had used a ϕ angle of 30 degrees and an apparent cohesion of 0 psf. The following plots show laboratory strength data and correlated SPT-based strength data.

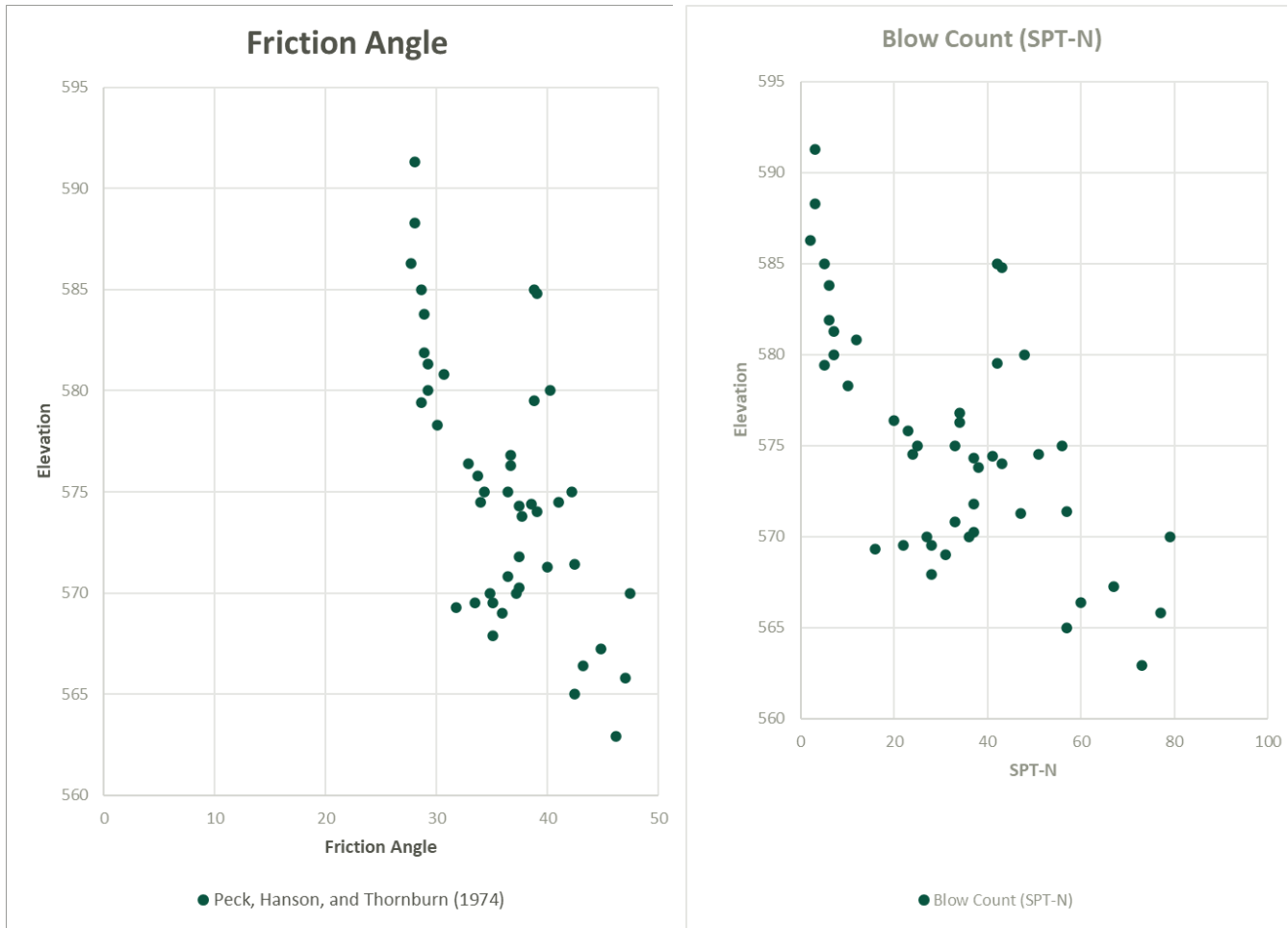




Based on an evaluation of the data and WSP’s experience at other sites, WSP selected representative strength parameters for the Alluvial 1 material of: ϕ angle of 32 degrees and apparent cohesion of 100 psf.

3.3 STRENGTH PARAMETERS - ALLUVIUM 2

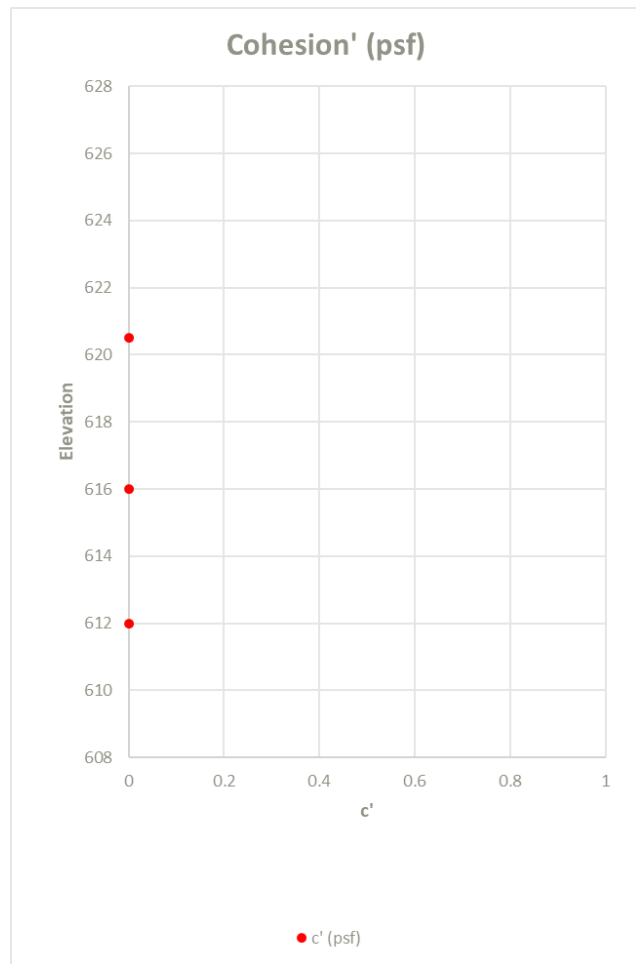
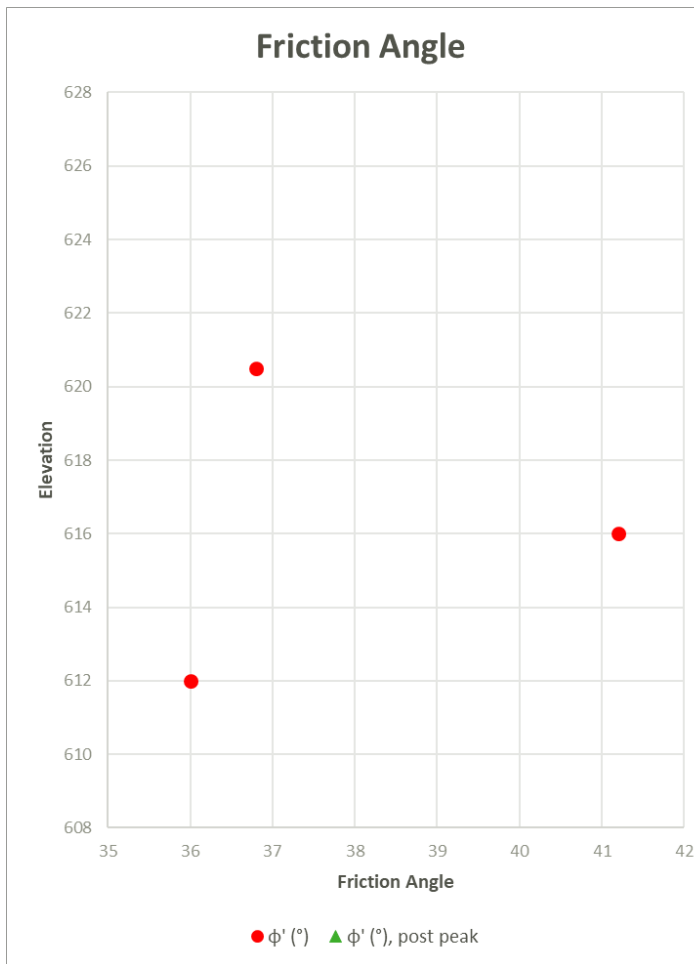
From previous analysis at the site, the triaxial compression tests performed on the samples of alluvial soils from 2005 yielded effective shear strengths with ϕ ranging from 32° to 36.4° and apparent cohesions ranging from 100 to 460 psf. Previous slope stability calculations had used a ϕ angle of 30 degrees and an apparent cohesion of 0 psf. The following plots show the correlated strength data from SPT values.

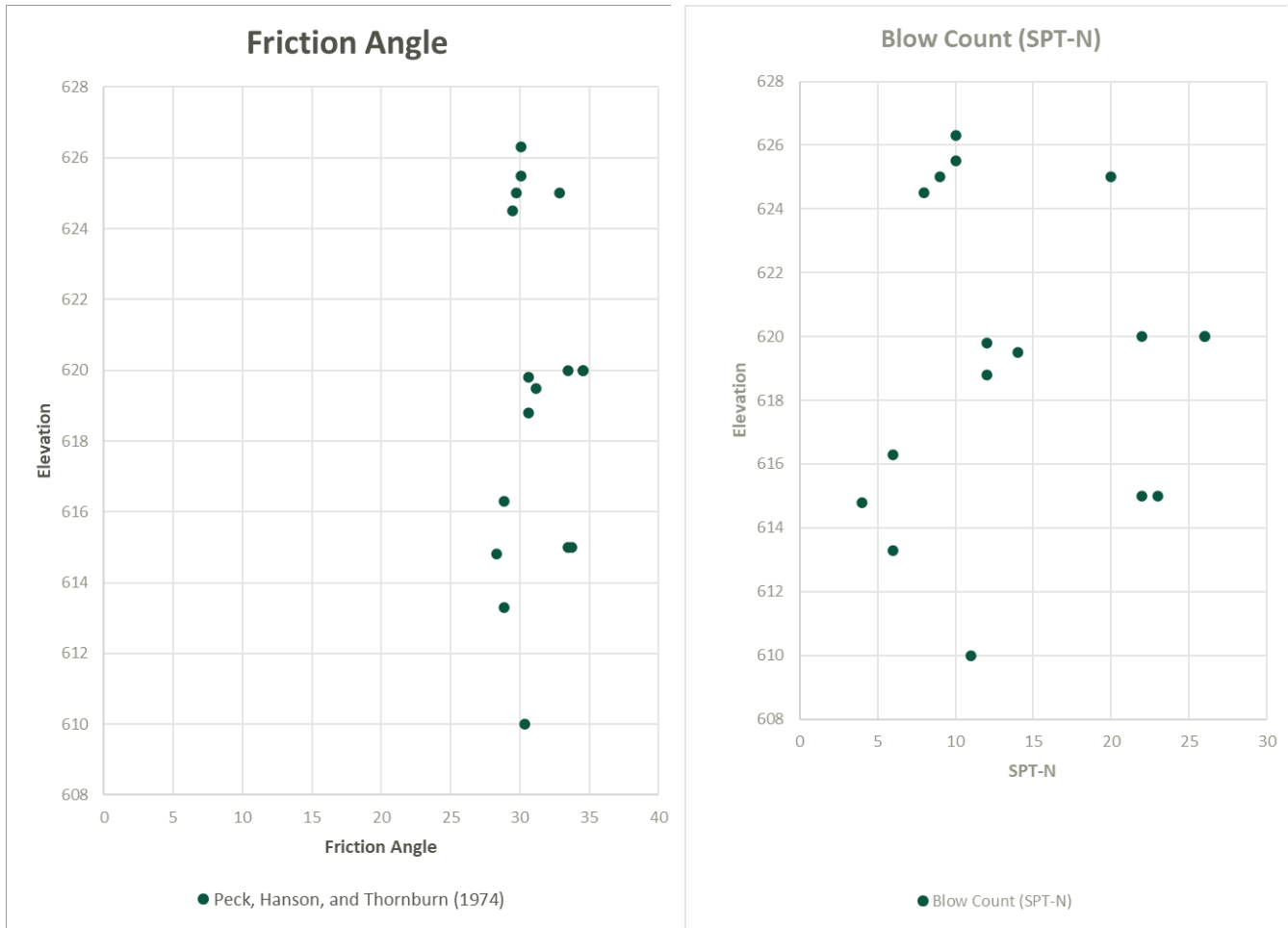


Based on an evaluation of the data and WSP’s experience at other sites, WSP selected representative strength parameters for the Alluvial 2 materials of: ϕ angle of 32 degrees and apparent cohesion of 0 psf. The underlying alluvial materials are more granular (less cohesive) than the upper alluvium and are expected to have less apparent cohesion.

3.4 STRENGTH PARAMETERS - EMBANKMENT MATERIALS

From previous analysis at the site, the shear strength properties for the embankment materials were estimated based on the results of the tests performed on the site’s native subsoil (alluvial material), with some minor modification to account for the soil being compacted during placement. Previous slope stability calculations had used a ϕ angle of 30 degrees and an apparent cohesion of 125 psf. The following plots show laboratory strength data and correlated SPT-based strength data.





Based on an evaluation of the data and WSP's experience at other sites, WSP selected representative strength parameters for the embankment materials of: ϕ angle of 32 degrees and apparent cohesion of 100 psf.

4 SUMMARY OF STRENGTH PARAMETERS

Strength parameters based on the geotechnical information provided in the GDR

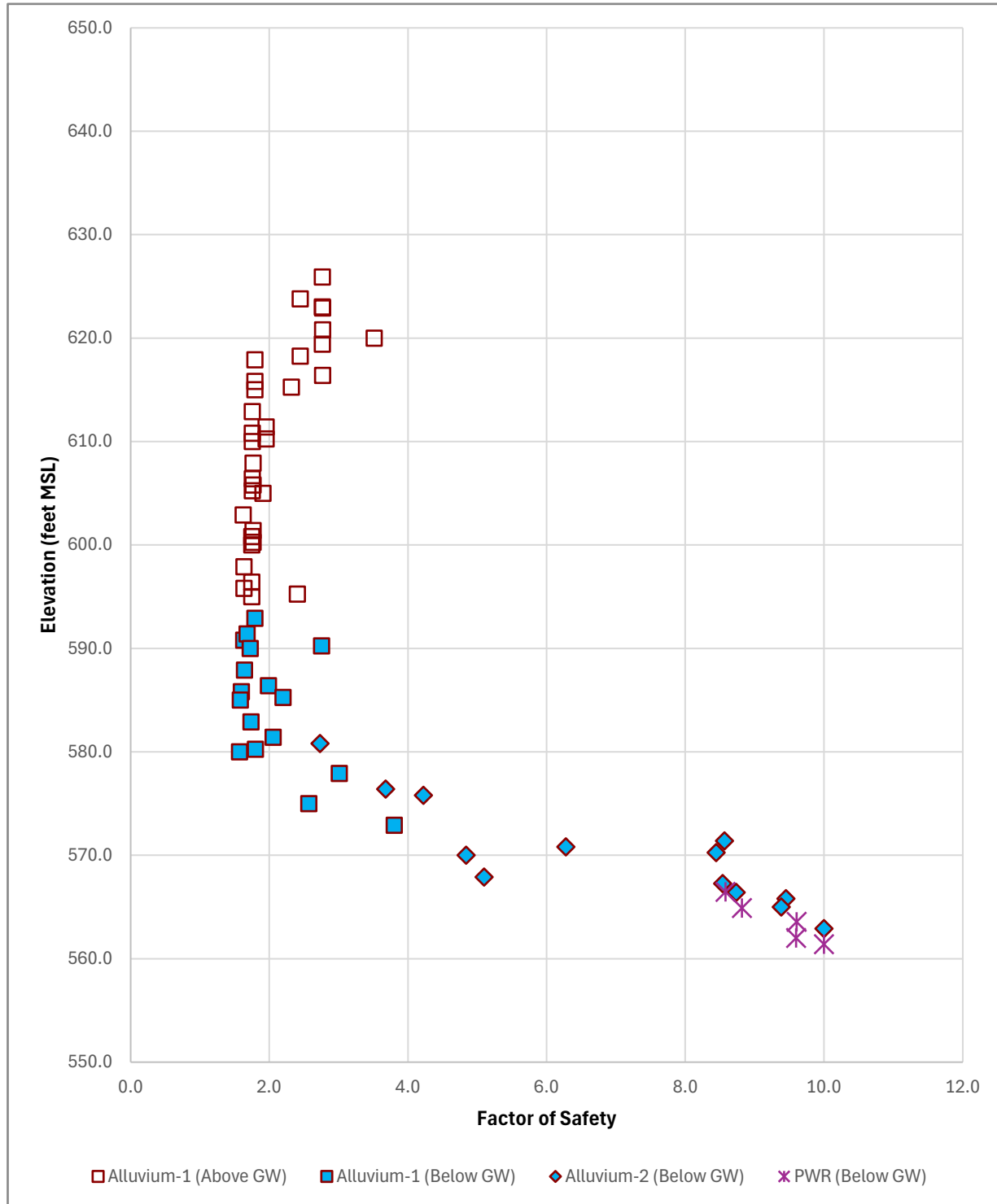
Table 1: Summary of Geotechnical Material Properties

Material	Unit Weight (pcf)	Drained Strength		Undrained Strength	
		Effective Internal Friction ϕ' (°)	Effective Cohesion c' (psf)	Total Internal Friction ϕ' (°)	Total Cohesion c' (psf)
Embankment Materials					
Earthfill	125	32	100	-	-
Coal Combustion Residuals					
CCR-Impounded	100	28	0	-	-
Alluvium					
Alluvium-1	120	32	100	0	26
Alluvium-2	115	32	0	0	26

APPENDIX C

Liquefaction Assessment

APPENDIX D - LIQUEFACTION ASSESSMENT RESULTS Kanawha River Ash Pond (BAP), Former Kanawha River Power Plant, Glasgow, WV Initial Safety Factor Assessment 2026



Note(s)

1. Reported Factor of Safety is an average value obtained from consecutive SPT records of a same material.
2. Calculated Factor of Safety is for comparison only. Liquefaction would not occur in unsaturated dike materials.

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID B-16		a_{max} (USGS Hazard tool)			Corrections to SPT	
Groundwater Elevation=	594 feet	10% in 50 yrs	0.053 Lat	38.2045	C _N =	see each cell <i>Overburden</i>
Ground Surface Elevation=	628.9 feet	5% in 50 yrs	0.085 Long	-81.4132	C _E =	0.8 <i>Energy Ratio</i>
Depth to Water=	34.9 feet - below ground surface	2% in 50 yrs	0.153		C _B =	1.05 <i>Borehole Diameter</i>
M _w =	5.96 <i>Moment magnitude of earthquake at the Site</i>				C _R =	see each cell <i>Rod Length</i>
MSF=	1.80 <i>Magnitude Scaling Factor</i>				C _S =	1 <i>Sampling Method</i>
a _{max} =	0.153 <i>peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool</i>				Pa=	2116.2 <i>Atmospheric Pressure, psf</i>

Material Type	Unit Weight (pcf)	Avg % Fines
Alluvium-1	120	≥35%
Alluvium-2	115	≥35%
PWR	140	≤5%

Groundwater (Approximate)
(GW is assumed to be at elevation 594 feet according to the OHWM)

Bottom Elevation (feet)	Bottom Depth (feet)	Depth (m)	Predominant Material Type	Unit Weight, γ _b (pcf)	Total Stress, σ _v (psf)	Effective Stress, σ' _v (psf)	Stress Reduction Coefficient, r _d	Cyclic Stress Ratio, CSR	SPT-N ¹ (bpf)	Corrected SPT-N, (N ₁) ₆₀ (bpf)	Fine Content (based on lab results or visual classification)	Correction to SPT, Overburden Factor, C _N	Correction to SPT-Rod Length, C _R	(N ₁) ₆₀ adjusted to equivalent clean-sand value, (N ₁) _{60cs} (bpf)	Cyclic Resistance Ratio (CRR) for earthquakes for M _w = 7.5, CRR _{7.5}	CRR for Mw (5.96) at the Site, CRR	Factor of Safety, FS
625.9	3.0	0.91	Alluvium-1	120	360	360	0.99	0.099	5	8	≥35%	2.4	0.75	14	0.152	0.273	2.77
622.9	6.0	1.83	Alluvium-1	120	720	720	0.99	0.098	7	8	≥35%	1.7	0.75	14	0.151	0.272	2.77
617.9	11.0	3.35	Alluvium-1	120	1320	1320	0.97	0.097	3	3	≥35%	1.3	0.80	8	0.096	0.174	1.79
612.9	16.0	4.88	Alluvium-1	120	1920	1920	0.96	0.096	3	2	≥35%	1.0	0.85	8	0.093	0.168	1.76
607.9	21.0	6.40	Alluvium-1	120	2520	2520	0.95	0.095	3	2	≥35%	0.9	0.95	8	0.093	0.167	1.77
602.9	26.0	7.93	Alluvium-1	120	3120	3120	0.94	0.093	2	1	≥35%	0.8	0.95	7	0.084	0.152	1.62
597.9	31.0	9.5	Alluvium-1	120	3720	3720	0.92	0.092	2	1	≥35%	0.8	0.95	6	0.083	0.150	1.63
592.9	36.0	10.98	Alluvium-1	120	4320	4251	0.88	0.089	3	2	≥35%	0.7	1.00	7	0.089	0.160	1.79
587.9	41.0	12.50	Alluvium-1	120	4920	4539	0.84	0.091	2	1	≥35%	0.7	1.00	6	0.083	0.149	1.64
582.9	46.0	14.02	Alluvium-1	120	5520	4827	0.80	0.091	3	2	≥35%	0.7	1.00	7	0.088	0.158	1.74
577.9	51.0	15.55	Alluvium-1	120	6120	5115	0.76	0.090	14	8	≥35%	0.6	1.00	14	0.151	0.272	3.01
572.9	56.0	17.07	Alluvium-1	120	6720	5403	0.72	0.089	20	11	≥35%	0.6	1.00	18	0.188	0.338	3.80
567.9	61.0	18.60	Alluvium-2	115	7295	5666	0.68	0.087	28	14	≥35%	0.6	1.00	22	0.246	0.442	5.10
562.9	66.0	20.12	Alluvium-2	115	7870	5929	0.64	0.084	73	37	≥35%	0.6	1.00	30	0.468	0.842	10.00
561.4	67.5	20.58	PWR	140	8080	6046	0.62	0.083	100	50	≤5%	0.6	1.00	30	0.468	0.842	10.00

Note(s)

1. Measured resistance from Standard Penetration Test (SPT), blow count (N)
2. m=meter; ft-msl= feet above mean sea level; pcf= pounds per cubic foot; psf= pounds per square foot; bpf= blows per foot; yrs= years; USGS=United States Geological Survey
3. Average % Fines of the material type were used for (N1)60cs correction when lab results were not available for a given depth

Reference(s)

1. Youd, B. T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Jr, L. F. H., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Iii, W. F. M., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Li, K. H. S. (2001). Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER / NSF Workshops on Evaluation. Journal of Geotechnical and Geoenvironmental Engineering, 127(10), 817–833. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2001\)127:10\(817\)](https://doi.org/10.1061/(ASCE)1090-0241(2001)127:10(817))
2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: SM Date: 12/30/2025
Checked by: NG Date: 12/31/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID B-17		a_{max} (USGS Hazard tool)		
Groundwater Elevation=	594 feet	10% in 50 yrs	0.053 Lat	38.2045
Ground Surface Elevation=	626.8 feet	5% in 50 yrs	0.085 Long	-81.4132
Depth to Water=	32.8 feet - below ground surface	2% in 50 yrs	0.153	
M_w =	5.96 <i>Moment magnitude of earthquake at the Site</i>			
MSF=	1.80 <i>Magnitude Scaling Factor</i>			
a_{max} =	0.153 <i>peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool</i>			

Corrections to SPT

C_N =	see each cell	<i>Overburden</i>
C_E =		0.8 <i>Energy Ratio</i>
C_B =		1.05 <i>Borehole Diameter</i>
C_R =	see each cell	<i>Rod Length</i>
C_S =		1 <i>Sampling Method</i>
P_a =		2116.2 <i>Atmospheric Pressure, psf</i>

Material Type	Unit Weight (pcf)	Avg % Fines
Alluvium-1	120	≥35%
Alluvium-2	115	≥35%
PWR	140	≤5%

Groundwater (Approximate)
(GW is assumed to be at elevation 594 feet according to the OHWM)

Bottom Elevation (feet)	Bottom Depth (feet)	Depth (m)	Predominant Material Type	Unit Weight, γ_b (pcf)	Total Stress, σ_v (psf)	Effective Stress, σ'_v (psf)	Stress Reduction Coefficient, r_d	Cyclic Stress Ratio, CSR	SPT-N ¹ (bpf)	Corrected SPT-N, $(N_1)_{60}$ (bpf)	Fine Content (based on lab results or visual classification)	Correction to SPT, Overburden Factor, C_N	Correction to SPT-Rod Length, C_R	$(N_1)_{60}$ adjusted to equivalent clean-sand value, $(N_1)_{60cs}$ (bpf)	Cyclic Resistance Ratio (CRR) for earthquakes for $M_w = 7.5$, $CRR_{7.5}$	CRR for M_w (5.96) at the Site, CRR	Factor of Safety, FS
623.8	3.0	0.91	Alluvium-1	120	360	360	0.99	0.099	4	6	≥35%	2.4	0.75	12	0.134	0.242	2.45
620.8	6.0	1.83	Alluvium-1	120	720	720	0.99	0.098	7	8	≥35%	1.7	0.75	14	0.151	0.272	2.77
615.8	11.0	3.35	Alluvium-1	120	1320	1320	0.97	0.097	3	3	≥35%	1.3	0.80	8	0.096	0.174	1.79
610.8	16.0	4.88	Alluvium-1	120	1920	1920	0.96	0.096	3	2	≥35%	1.0	0.85	8	0.093	0.168	1.76
605.8	21.0	6.40	Alluvium-1	120	2520	2520	0.95	0.095	3	2	≥35%	0.9	0.95	8	0.093	0.167	1.77
600.8	26.0	7.93	Alluvium-1	120	3120	3120	0.94	0.093	3	2	≥35%	0.8	0.95	7	0.091	0.163	1.75
595.8	31.0	9.5	Alluvium-1	120	3720	3720	0.92	0.092	2	1	≥35%	0.8	0.95	6	0.083	0.150	1.63
590.8	36.0	10.98	Alluvium-1	120	4320	4120	0.88	0.092	2	1	≥35%	0.7	1.00	6	0.083	0.150	1.63
585.8	41.0	12.50	Alluvium-1	120	4920	4408	0.84	0.093	2	1	≥35%	0.7	1.00	6	0.083	0.149	1.60
580.8	46.0	14.02	Alluvium-2	115	5495	4671	0.80	0.094	12	7	≥35%	0.7	1.00	13	0.142	0.255	2.73
575.8	51.0	15.55	Alluvium-2	115	6070	4934	0.76	0.093	23	13	≥35%	0.7	1.00	20	0.218	0.392	4.22
570.8	56.0	17.07	Alluvium-2	115	6645	5197	0.72	0.091	33	18	≥35%	0.6	1.00	26	0.318	0.573	6.28
565.8	61.0	18.60	Alluvium-2	115	7220	5460	0.68	0.089	77	40	≥35%	0.6	1.00	30	0.468	0.842	9.45
563.6	63.3	19.28	PWR	140	7535	5635	0.66	0.088	100	51	≤5%	0.6	1.00	30	0.468	0.842	9.61

Note(s)

1. Measured resistance from Standard Penetration Test (SPT), blow count (N)
2. m=meter; ft-msl= feet above mean sea level; pcf= pounds per cubic foot; psf= pounds per square foot; bpf= blows per foot; yrs= years; USGS=United States Geological Survey
3. Average % Fines of the material type were used for $(N_1)_{60cs}$ correction when lab results were not available for a given depth

Reference(s)

1. Youd, B. T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Jr, L. F. H., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Iii, W. F. M., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Li, K. H. S. (2001). Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER / NSF Workshops on Evaluation. Journal of Geotechnical and Geoenvironmental Engineering, 127(10), 817–833. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2001\)127:10\(817\)](https://doi.org/10.1061/(ASCE)1090-0241(2001)127:10(817))
2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: SM Date: 12/30/2025
Checked by: NG Date: 12/31/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID B-18		a_{max} (USGS Hazard tool)		
Groundwater Elevation=	594 feet	10% in 50 yrs	0.053 Lat	38.2045
Ground Surface Elevation=	622.4 feet	5% in 50 yrs	0.085 Long	-81.4132
Depth to Water=	28.4 feet - below ground surface	2% in 50 yrs	0.153	
M_w =	5.96 <i>Moment magnitude of earthquake at the Site</i>			
MSF=	1.80 <i>Magnitude Scaling Factor</i>			
a_{max} =	0.153 <i>peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool</i>			

Corrections to SPT

C_N =	see each cell	<i>Overburden</i>
C_E =		0.8 <i>Energy Ratio</i>
C_B =		1.05 <i>Borehole Diameter</i>
C_R =	see each cell	<i>Rod Length</i>
C_S =		1 <i>Sampling Method</i>
P_a =		2116.2 <i>Atmospheric Pressure, psf</i>

Material Type	Unit Weight (pcf)	Avg % Fines
Alluvium-1	120	≥35%
Alluvium-2	115	≥35%
PWR	140	≤5%

Groundwater (Approximate)
(GW is assumed to be at elevation 594 feet according to the OHWM)

Bottom Elevation (feet)	Bottom Depth (feet)	Depth (m)	Predominant Material Type	Unit Weight, γ_b (pcf)	Total Stress, σ_v (psf)	Effective Stress, σ'_v (psf)	Stress Reduction Coefficient, r_d	Cyclic Stress Ratio, CSR	SPT-N ¹ (bpf)	Corrected SPT-N, $(N_1)_{60}$ (bpf)	Fine Content (based on lab results or visual classification)	Correction to SPT, Overburden Factor, C_N	Correction to SPT-Rod Length, C_R	$(N_1)_{60}$ adjusted to equivalent clean-sand value, $(N_1)_{60cs}$ (bpf)	Cyclic Resistance Ratio (CRR) for earthquakes for $M_w = 7.5$, $CRR_{7.5}$	CRR for M_w (5.96) at the Site, CRR	Factor of Safety, FS
619.4	3.0	0.91	Alluvium-1	120	360	360	0.99	0.099	5	8	≥35%	2.4	0.75	14	0.152	0.273	2.77
616.4	6.0	1.83	Alluvium-1	120	720	720	0.99	0.098	7	8	≥35%	1.7	0.75	14	0.151	0.272	2.77
611.4	11.0	3.35	Alluvium-1	120	1320	1320	0.97	0.097	4	3	≥35%	1.3	0.80	9	0.105	0.189	1.95
606.4	16.0	4.88	Alluvium-1	120	1920	1920	0.96	0.096	3	2	≥35%	1.0	0.85	8	0.093	0.168	1.76
601.4	21.0	6.40	Alluvium-1	120	2520	2520	0.95	0.095	3	2	≥35%	0.9	0.95	8	0.093	0.167	1.77
596.4	26.0	7.9	Alluvium-1	120	3120	3120	0.94	0.093	3	2	≥35%	0.8	0.95	7	0.091	0.163	1.75
591.4	31.0	9.45	Alluvium-1	120	3720	3558	0.92	0.096	3	2	≥35%	0.8	0.95	7	0.089	0.161	1.68
586.4	36.0	10.98	Alluvium-1	120	4320	3846	0.88	0.098	6	4	≥35%	0.7	1.00	9	0.109	0.196	1.99
581.4	41.0	12.50	Alluvium-1	120	4920	4134	0.84	0.099	7	4	≥35%	0.7	1.00	10	0.114	0.204	2.06
576.4	46.0	14.02	Alluvium-2	115	5495	4397	0.80	0.099	20	12	≥35%	0.7	1.00	19	0.203	0.366	3.68
571.4	51.0	15.55	Alluvium-2	115	6070	4660	0.76	0.098	57	32	≥35%	0.7	1.00	30	0.468	0.842	8.56
566.4	56.0	17.07	Alluvium-2	115	6645	4923	0.72	0.096	60	33	≥35%	0.7	1.00	30	0.468	0.842	8.73
564.9	57.5	17.53	PWR	140	6855	5039	0.71	0.096	100	54	≤5%	0.6	1.00	30	0.468	0.842	8.82

Note(s)

1. Measured resistance from Standard Penetration Test (SPT), blow count (N)
2. m=meter; ft-msl= feet above mean sea level; pcf= pounds per cubic foot; psf= pounds per square foot; bpf= blows per foot; yrs= years; USGS=United States Geological Survey
3. Average % Fines of the material type were used for $(N_1)_{60cs}$ correction when lab results were not available for a given depth

Reference(s)

1. Youd, B. T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Jr, L. F. H., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Iii, W. F. M., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Li, K. H. S. (2001). Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER / NSF Workshops on Evaluation. Journal of Geotechnical and Geoenvironmental Engineering, 127(10), 817–833. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2001\)127:10\(817\)](https://doi.org/10.1061/(ASCE)1090-0241(2001)127:10(817))
2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: SM Date: 12/30/2025
Checked by: NG Date: 12/31/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID B-19		a_{max} (USGS Hazard tool)		
Groundwater Elevation=	594 feet	10% in 50 yrs	0.053 Lat	38.2045
Ground Surface Elevation=	626.0 feet	5% in 50 yrs	0.085 Long	-81.4132
Depth to Water=	32.0 feet - below ground surface	2% in 50 yrs	0.153	
M_w =	5.96 <i>Moment magnitude of earthquake at the Site</i>			
MSF=	1.80 <i>Magnitude Scaling Factor</i>			
a_{max} =	0.153 <i>peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool</i>			

Corrections to SPT

C_N =	see each cell	<i>Overburden</i>
C_E =		0.8 <i>Energy Ratio</i>
C_B =		1.05 <i>Borehole Diameter</i>
C_R =	see each cell	<i>Rod Length</i>
C_S =		1 <i>Sampling Method</i>
P_a =		2116.2 <i>Atmospheric Pressure, psf</i>

Material Type	Unit Weight (pcf)	Avg % Fines
Alluvium-1	120	≥35%
Alluvium-2	115	≥35%
PWR	140	≤5%

Groundwater (Approximate)
(GW is assumed to be at elevation 594 feet according to the OHWM)

Bottom Elevation (feet)	Bottom Depth (feet)	Depth (m)	Predominant Material Type	Unit Weight, γ_b (pcf)	Total Stress, σ_v (psf)	Effective Stress, σ'_v (psf)	Stress Reduction Coefficient, r_d	Cyclic Stress Ratio, CSR	SPT-N ¹ (bpf)	Corrected SPT-N, $(N_1)_{60}$ (bpf)	Fine Content (based on lab results or visual classification)	Correction to SPT, Overburden Factor, C_N	Correction to SPT-Rod Length, C_R	$(N_1)_{60}$ adjusted to equivalent clean-sand value, $(N_1)_{60cs}$ (bpf)	Cyclic Resistance Ratio (CRR) for earthquakes for $M_w = 7.5$, $CRR_{7.5}$	CRR for M_w (5.96) at the Site, CRR	Factor of Safety, FS
623.0	3.0	0.91	Alluvium-1	120	360	360	0.99	0.099	5	8	≥35%	2.4	0.75	14	0.152	0.273	2.77
620.0	6.0	1.83	Alluvium-1	120	720	720	0.99	0.098	10	11	≥35%	1.7	0.75	18	0.191	0.345	3.51
615.0	11.0	3.35	Alluvium-1	120	1320	1320	0.97	0.097	3	3	≥35%	1.3	0.80	8	0.096	0.174	1.79
610.0	16.0	4.88	Alluvium-1	120	1920	1920	0.96	0.096	3	2	≥35%	1.0	0.85	8	0.093	0.168	1.76
605.0	21.0	6.40	Alluvium-1	120	2520	2520	0.95	0.095	4	3	≥35%	0.9	0.95	9	0.100	0.180	1.91
600.0	26.0	7.93	Alluvium-1	120	3120	3120	0.94	0.093	3	2	≥35%	0.8	0.95	7	0.091	0.163	1.75
595.0	31.0	9.5	Alluvium-1	120	3720	3720	0.92	0.092	3	2	≥35%	0.8	0.95	7	0.089	0.160	1.75
590.0	36.0	10.98	Alluvium-1	120	4320	4070	0.88	0.093	3	2	≥35%	0.7	1.00	7	0.089	0.160	1.73
585.0	41.0	12.50	Alluvium-1	120	4920	4358	0.84	0.094	2	1	≥35%	0.7	1.00	6	0.083	0.149	1.58
580.0	46.0	14.02	Alluvium-1	120	5520	4646	0.80	0.094	2	1	≥35%	0.7	1.00	6	0.083	0.149	1.57
575.0	51.0	15.55	Alluvium-1	120	6120	4934	0.76	0.094	11	6	≥35%	0.7	1.00	12	0.134	0.241	2.57
570.0	56.0	17.07	Alluvium-2	115	6695	5197	0.72	0.092	27	14	≥35%	0.6	1.00	22	0.247	0.445	4.84
565.0	61.0	18.60	Alluvium-2	115	7270	5460	0.68	0.090	57	30	≥35%	0.6	1.00	30	0.468	0.842	9.39
562.0	64.0	19.51	PWR	140	7690	5693	0.65	0.088	100	51	≤5%	0.6	1.00	30	0.468	0.842	9.60

Note(s)

1. Measured resistance from Standard Penetration Test (SPT), blow count (N)
2. m=meter; ft-msl= feet above mean sea level; pcf= pounds per cubic foot; psf= pounds per square foot; bpf= blows per foot; yrs= years; USGS=United States Geological Survey
3. Average % Fines of the material type were used for $(N_1)_{60cs}$ correction when lab results were not available for a given depth

Reference(s)

1. Youd, B. T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Jr, L. F. H., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Iii, W. F. M., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Li, K. H. S. (2001). Liquefaction Resistance of Soils Summary Report From the 1996 NCEER and 1998 NCEER / NSF Workshops on Evaluation. Journal of Geotechnical and Geoenvironmental Engineering, 127(10), 817–833. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2001\)127:10\(817\)](https://doi.org/10.1061/(ASCE)1090-0241(2001)127:10(817))
2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: SM Date: 12/30/2025
Checked by: NG Date: 12/31/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID B-20		a_{max} (USGS Hazard tool)		
Groundwater Elevation=	594 feet	10% in 50 yrs	0.053 Lat	38.2045
Ground Surface Elevation=	621.3 feet	5% in 50 yrs	0.085 Long	-81.4132
Depth to Water=	27.3 feet - below ground surface	2% in 50 yrs	0.153	
M_w =	5.96 <i>Moment magnitude of earthquake at the Site</i>			
MSF=	1.80 <i>Magnitude Scaling Factor</i>			
a_{max} =	0.153 <i>peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool</i>			

Corrections to SPT

C_N =	see each cell	<i>Overburden</i>
C_E =		0.8 <i>Energy Ratio</i>
C_B =		1.05 <i>Borehole Diameter</i>
C_R =	see each cell	<i>Rod Length</i>
C_S =		1 <i>Sampling Method</i>
P_a =		2116.2 <i>Atmospheric Pressure, psf</i>

Material Type	Unit Weight (pcf)	Avg % Fines
Alluvium-1	120	≥35%
Alluvium-2	115	≥35%
PWR	140	≤5%

Groundwater (Approximate)
(GW is assumed to be at elevation 594 feet according to the OHWM)

Bottom Elevation (feet)	Bottom Depth (feet)	Depth (m)	Predominant Material Type	Unit Weight, γ_b (pcf)	Total Stress, σ_v (psf)	Effective Stress, σ'_v (psf)	Stress Reduction Coefficient, r_d	Cyclic Stress Ratio, CSR	SPT-N ¹ (bpf)	Corrected SPT-N, $(N_1)_{60}$ (bpf)	Fine Content (based on lab results or visual classification)	Correction to SPT, Overburden Factor, C_N	Correction to SPT-Rod Length, C_R	$(N_1)_{60}$ adjusted to equivalent clean-sand value, $(N_1)_{60cs}$ (bpf)	Cyclic Resistance Ratio (CRR) for earthquakes for $M_w = 7.5$, $CRR_{7.5}$	CRR for M_w (5.96) at the Site, CRR	Factor of Safety, FS
618.3	3.0	0.91	Alluvium-1	120	360	360	0.99	0.099	4	6	≥35%	2.4	0.75	12	0.134	0.242	2.45
615.3	6.0	1.83	Alluvium-1	120	720	720	0.99	0.098	5	5	≥35%	1.7	0.75	11	0.126	0.228	2.32
610.3	11.0	3.35	Alluvium-1	120	1320	1320	0.97	0.097	4	3	≥35%	1.3	0.80	9	0.105	0.189	1.95
605.3	16.0	4.88	Alluvium-1	120	1920	1920	0.96	0.096	3	2	≥35%	1.0	0.85	8	0.093	0.168	1.76
600.3	21.0	6.40	Alluvium-1	120	2520	2520	0.95	0.095	3	2	≥35%	0.9	0.95	8	0.093	0.167	1.77
595.3	26.0	7.9	Alluvium-1	120	3120	3120	0.94	0.093	8	5	≥35%	0.8	0.95	11	0.125	0.225	2.41
590.3	31.0	9.45	Alluvium-1	120	3720	3486	0.92	0.098	12	7	≥35%	0.8	0.95	14	0.150	0.270	2.76
585.3	36.0	10.98	Alluvium-1	120	4320	3774	0.88	0.100	8	5	≥35%	0.7	1.00	11	0.122	0.220	2.20
580.3	41.0	12.50	Alluvium-1	120	4920	4062	0.84	0.101	5	3	≥35%	0.7	1.00	9	0.101	0.182	1.80
575.3	46.0	14.02	Alluvium-1	120	5520	4350	0.80	0.101	8	5	≥35%	0.7	1.00	11	0.119	0.214	2.12
570.3	51.0	15.55	Alluvium-2	115	6095	4613	0.76	0.100	37	21	≥35%	0.7	1.00	30	0.468	0.842	8.44
567.3	54.0	16.46	Alluvium-2	115	6440	4771	0.73	0.099	67	37	≥35%	0.7	1.00	30	0.468	0.842	8.54
566.5	54.8	16.71	PWR	140	6552	4833	0.73	0.098	100	56	≤5%	0.7	1.00	30	0.468	0.842	8.58

Note(s)

1. Measured resistance from Standard Penetration Test (SPT), blow count (N)
2. m=meter; ft-msl= feet above mean sea level; pcf= pounds per cubic foot; psf= pounds per square foot; bpf= blows per foot; yrs= years; USGS=United States Geological Survey
3. Average % Fines of the material type were used for $(N_1)_{60cs}$ correction when lab results were not available for a given depth

Reference(s)

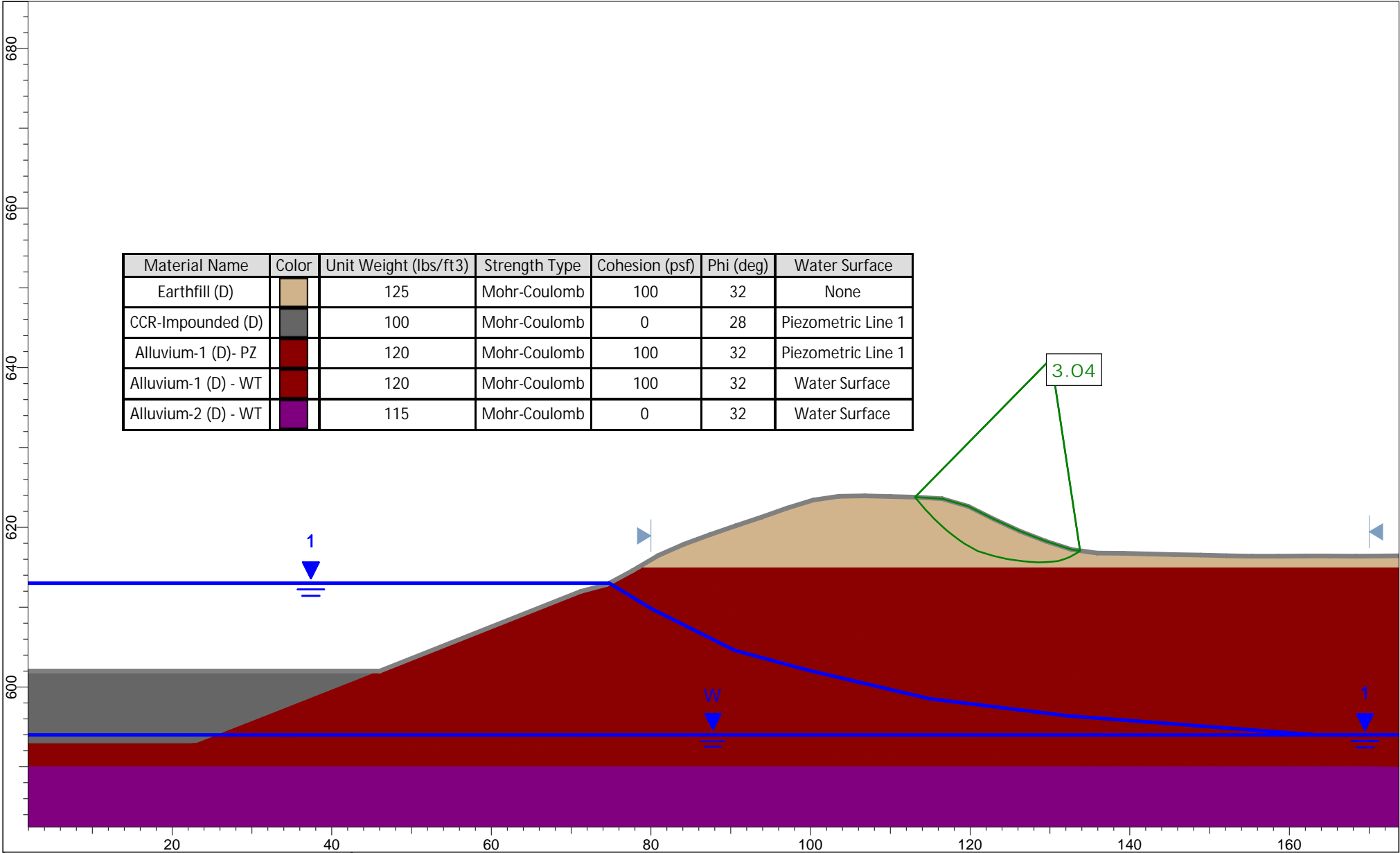
1. Youd, B. T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Jr, L. F. H., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Iii, W. F. M., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Li, K. H. S. (2001). Liquefaction Resistance of Soils Summary Report From the 1996 NCEER and 1998 NCEER / NSF Workshops on Evaluation. Journal of Geotechnical and Geoenvironmental Engineering, 127(10), 817–833. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2001\)127:10\(817\)](https://doi.org/10.1061/(ASCE)1090-0241(2001)127:10(817))
2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: SM Date: 12/30/2025
Checked by: NG Date: 12/31/2025



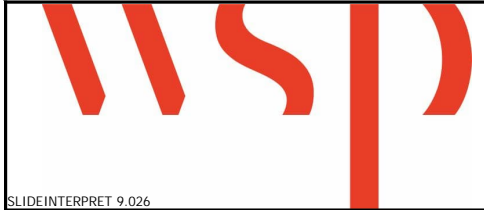
APPENDIX D

Slope Stability Analyses Outputs

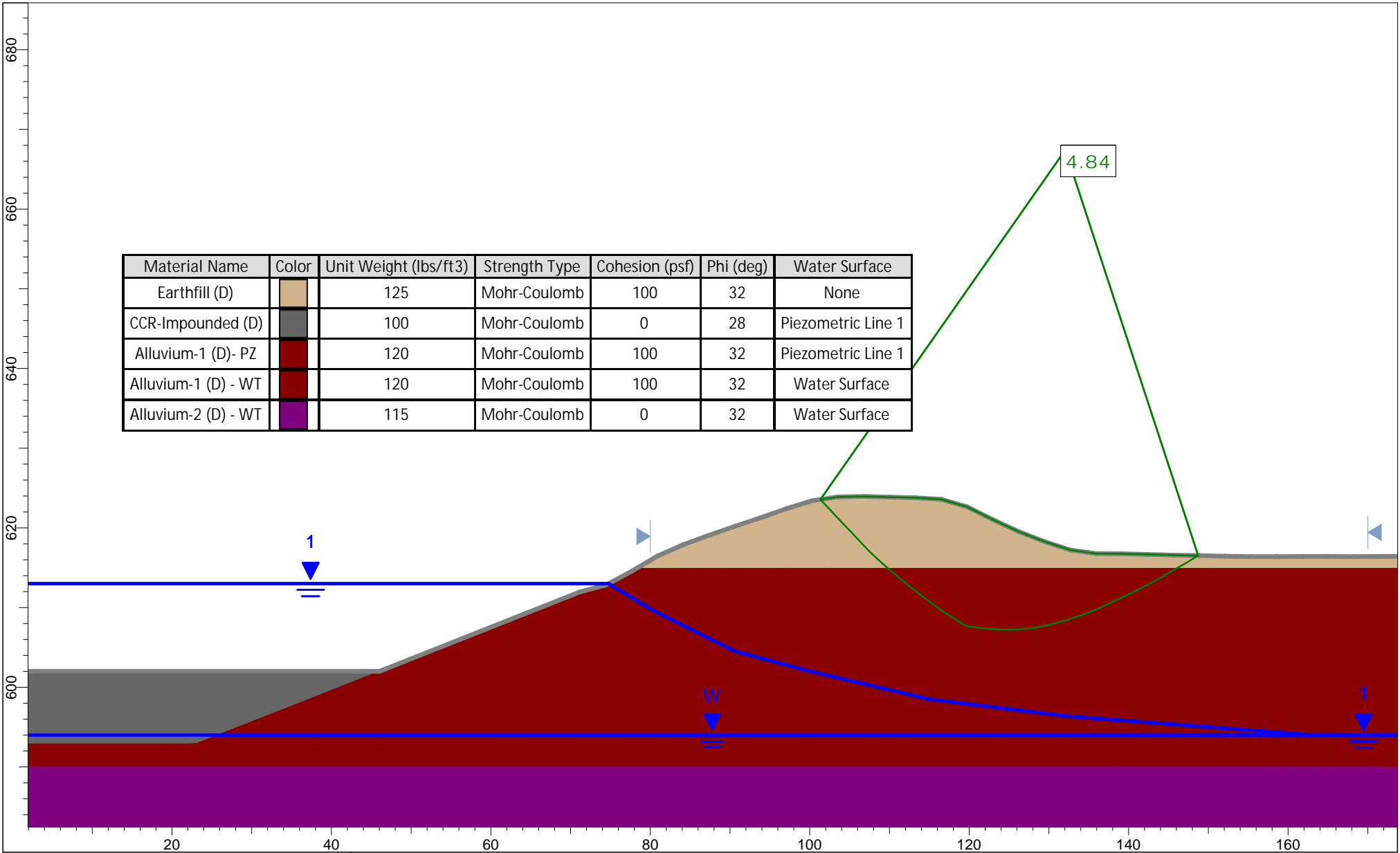


Material Name	Color	Unit Weight (lbs/ft ³)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Earthfill (D)		125	Mohr-Coulomb	100	32	None
CCR-Impounded (D)		100	Mohr-Coulomb	0	28	Piezometric Line 1
Alluvium-1 (D)- PZ		120	Mohr-Coulomb	100	32	Piezometric Line 1
Alluvium-1 (D) - WT		120	Mohr-Coulomb	100	32	Water Surface
Alluvium-2 (D) - WT		115	Mohr-Coulomb	0	32	Water Surface

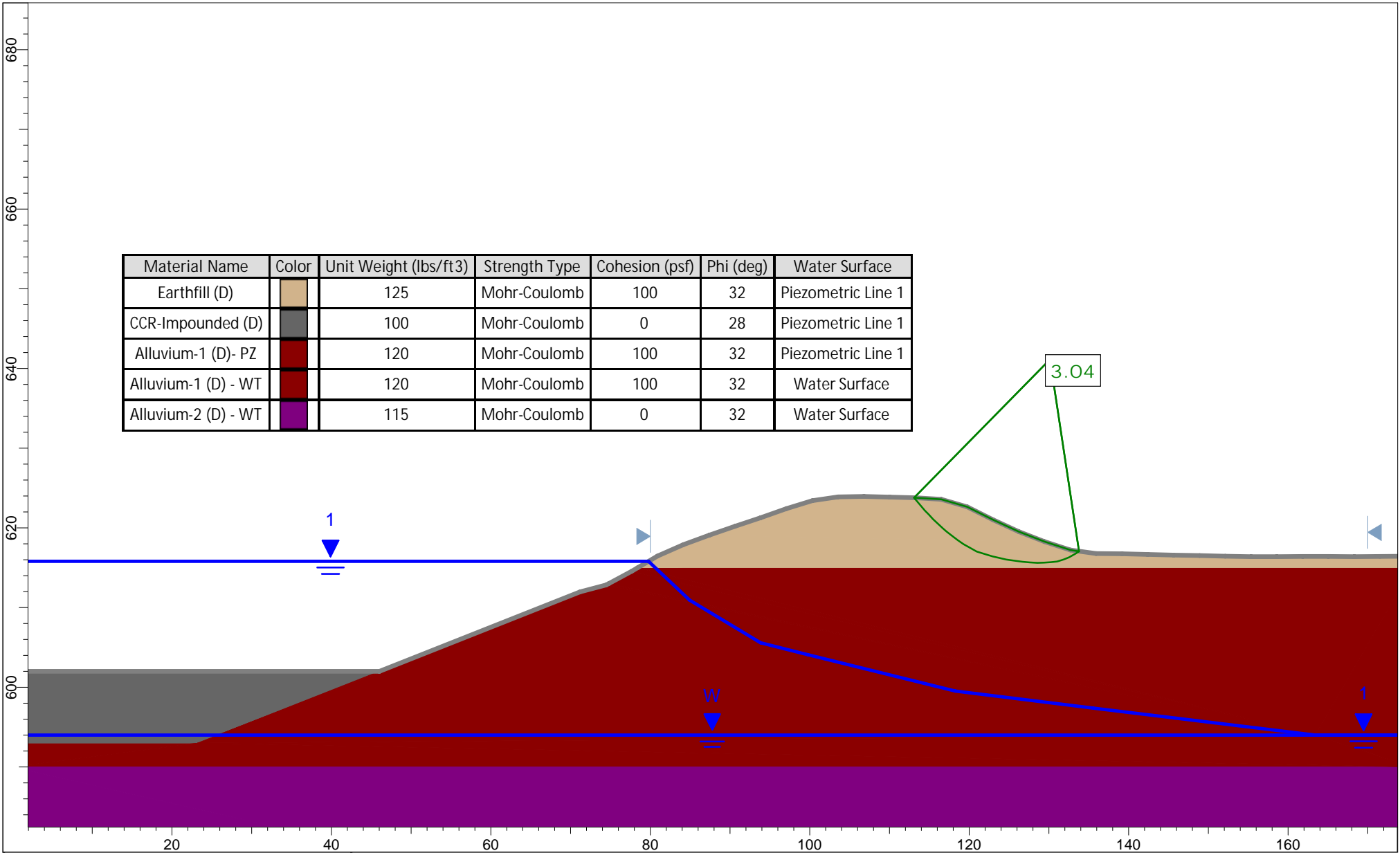
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Project							Bottom Ash Pond - Slope Stability Analyses				
Description:							AEP Sec A-A (Existing Conditions, Long-term Static) - Local Stability Analyses				
Drawn by:	NG	Checked by:	SM	Reviewed by:	SM	Scale:	1:200	Client:	AEP	Figure:	E-1
Date:	3/4/2026										



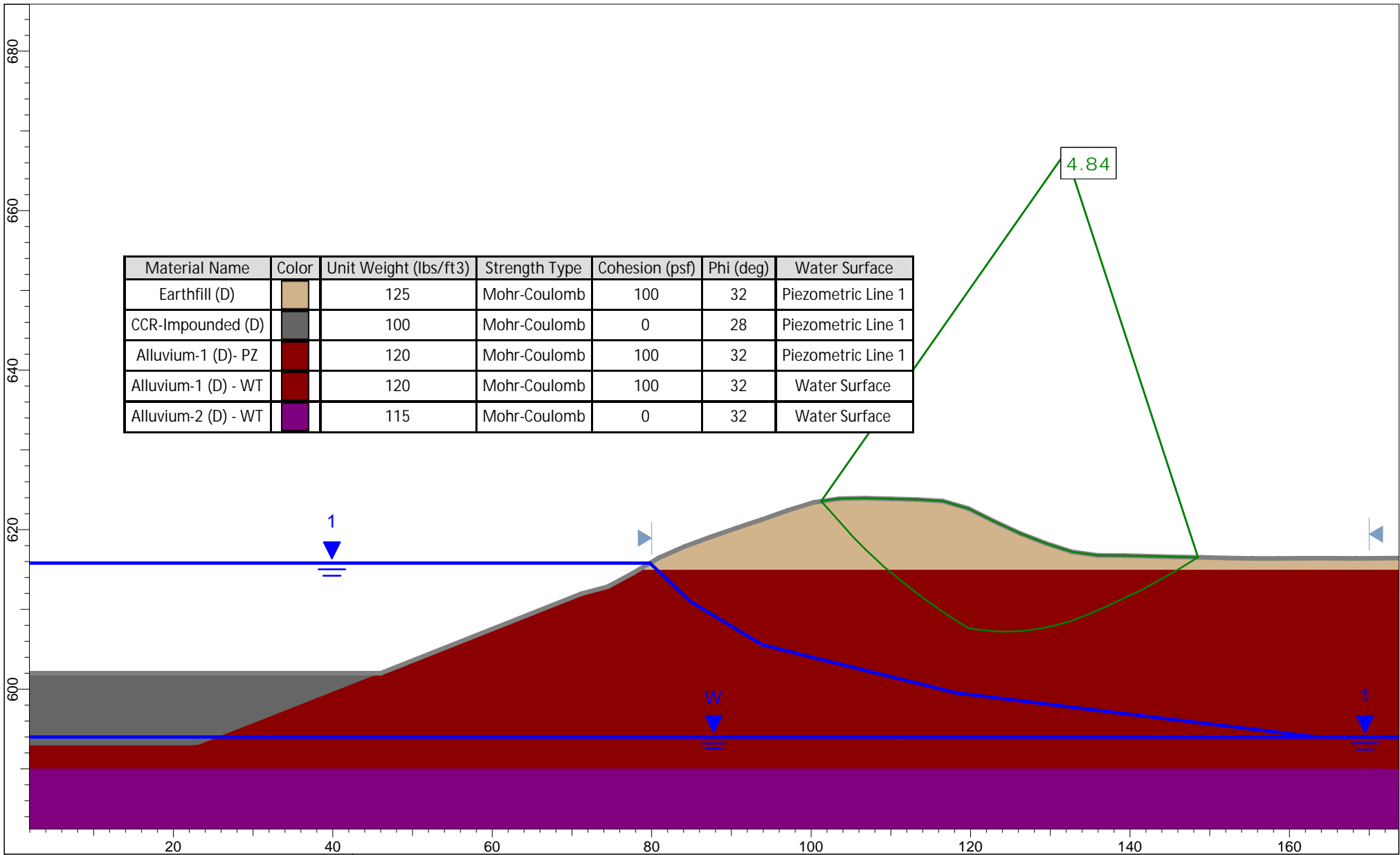
Project							Bottom Ash Pond - Slope Stability Analyses				
Description:							AEP Sec A-A (Existing Conditions, Long-term Static) - Global Stability Analyses				
Drawn by:	NG	Checked by:	SM	Reviewed by:	SM	Scale:	1:200	Client:	AEP	Figure:	E-2
Date:	3/4/2026										



Material Name	Color	Unit Weight (lbs/ft ³)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Earthfill (D)		125	Mohr-Coulomb	100	32	Piezometric Line 1
CCR-Impounded (D)		100	Mohr-Coulomb	0	28	Piezometric Line 1
Alluvium-1 (D)- PZ		120	Mohr-Coulomb	100	32	Piezometric Line 1
Alluvium-1 (D) - WT		120	Mohr-Coulomb	100	32	Water Surface
Alluvium-2 (D) - WT		115	Mohr-Coulomb	0	32	Water Surface



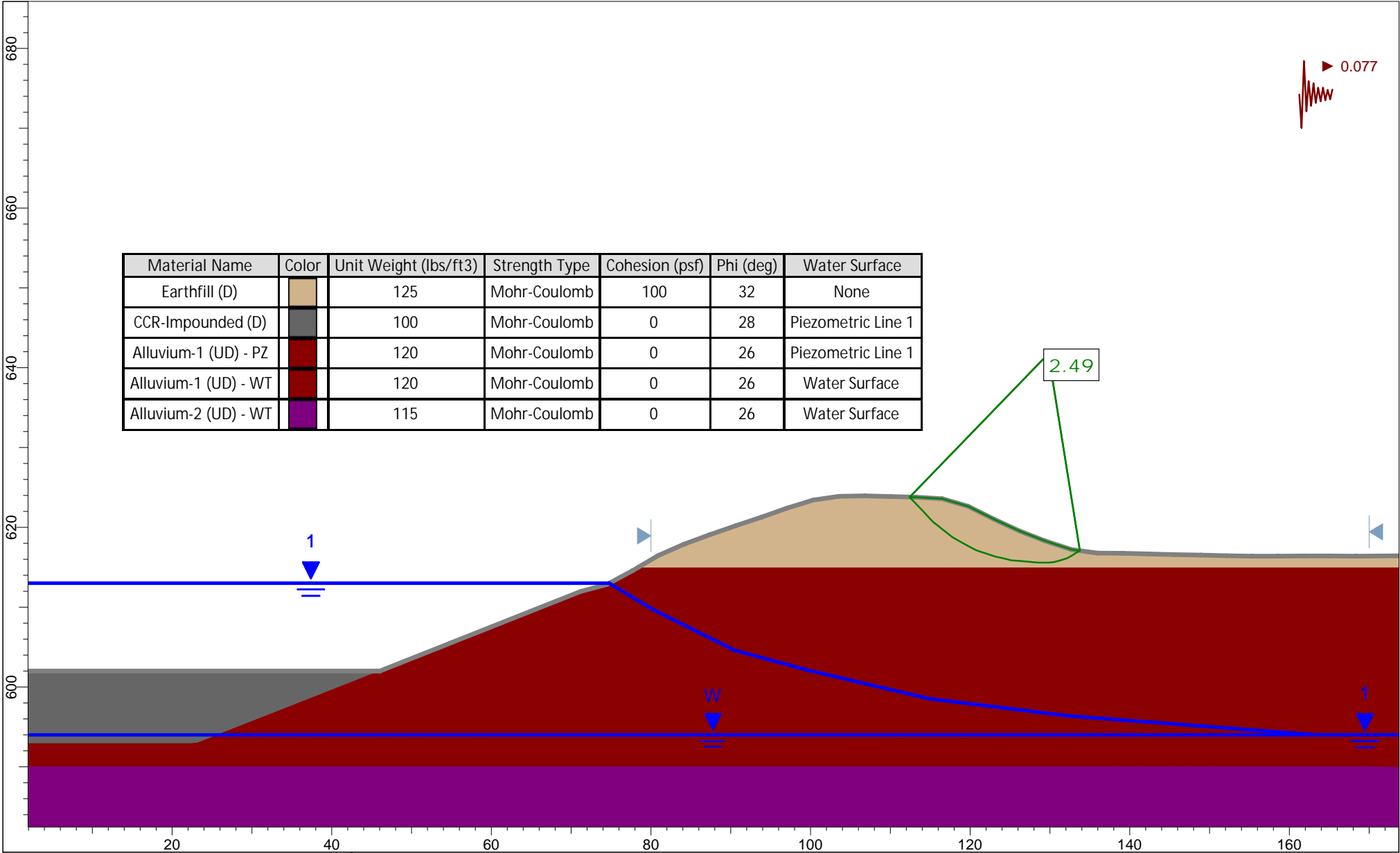
Project							Bottom Ash Pond - Slope Stability Analyses				
Description:							AEP Sec A-A (Existing Conditions, Maximum Surcharge Pool Condition - Long-term Static) - Local Stability Analyses				
Drawn by:	NG	Checked by:	SM	Reviewed by:	SM	Scale:	1:200	Client:	AEP	Figure:	E-3
Date:	3/4/2026										



Material Name	Color	Unit Weight (lbs/ft ³)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Earthfill (D)		125	Mohr-Coulomb	100	32	Piezometric Line 1
CCR-Impounded (D)		100	Mohr-Coulomb	0	28	Piezometric Line 1
Alluvium-1 (D)- PZ		120	Mohr-Coulomb	100	32	Piezometric Line 1
Alluvium-1 (D) - WT		120	Mohr-Coulomb	100	32	Water Surface
Alluvium-2 (D) - WT		115	Mohr-Coulomb	0	32	Water Surface



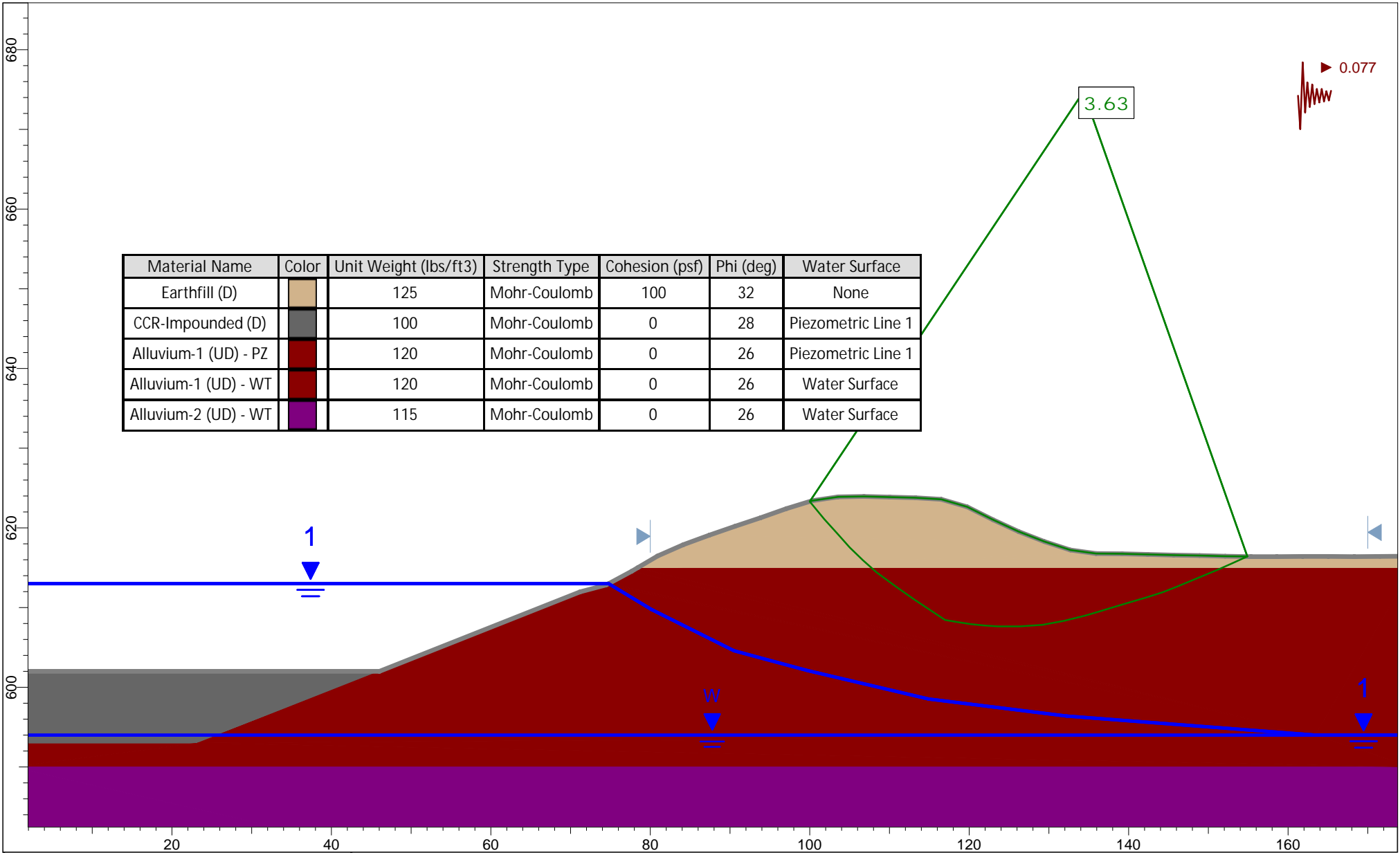
Project						Bottom Ash Pond - Slope Stability Analyses		
Description:						AEP Sec A-A (Existing Conditions, Maximum Surcharge Pool Condition - Long-term Static) - Global Stability Analyses		
Drawn by:	NG	Checked by:	SM	Reviewed by:	SM	Scale:	1:200	
Date:	3/4/2026			Client:	AEP		Figure:	E-4



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Earthfill (D)		125	Mohr-Coulomb	100	32	None
CCR-Impounded (D)		100	Mohr-Coulomb	0	28	Piezometric Line 1
Alluvium-1 (UD) - PZ		120	Mohr-Coulomb	0	26	Piezometric Line 1
Alluvium-1 (UD) - WT		120	Mohr-Coulomb	0	26	Water Surface
Alluvium-2 (UD) - WT		115	Mohr-Coulomb	0	26	Water Surface



Project							Bottom Ash Pond - Slope Stability Analyses				
Description:							AEP Sec A-A (Existing Conditions, Pseudo-static) - Local Stability Analyses				
Drawn by:	NG	Checked by:	SM	Reviewed by:	SM	Scale:	1:200	Client:	AEP	Figure:	E-5
Date:	3/4/2026										



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Earthfill (D)		125	Mohr-Coulomb	100	32	None
CCR-Impounded (D)		100	Mohr-Coulomb	0	28	Piezometric Line 1
Alluvium-1 (UD) - PZ		120	Mohr-Coulomb	0	26	Piezometric Line 1
Alluvium-1 (UD) - WT		120	Mohr-Coulomb	0	26	Water Surface
Alluvium-2 (UD) - WT		115	Mohr-Coulomb	0	26	Water Surface



Project						Bottom Ash Pond - Slope Stability Analyses		
Description:						AEP Sec A-A (Existing Conditions, Pseudo-static) - Global Stability Analyses		
Drawn by:	NG	Checked by:	SM	Reviewed by:	SM	Scale:	1:200	
Date:	3/4/2026			Client:	AEP		Figure:	E-6

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