SAFETY FACTOR ASSESSMENT

40 CFR 257.73 (e)

Ash Pond

Cabin Creek Site

Chelyan, WV

Revision 1- July 2026

Prepared for: Appalachian Power Company

Prepared by: American Electric Power Service Corporation

1 Riverside Plaza

Columbus, OH 43215

Cabin Creek Site

Ash Pond

Safety Factor Assessment

PREPARED BY Dan Murphy Digitally signed by Dan Murphy DN: cn=Dan Murphy, c=US, email=dsmurphy1@aep.com Date: 2026.07.28 14:17:13 -04'00' DATE _____

Dan Murphy, P.E.

REVIEWED BY Blake Arthur, PE Digitally signed by Blake Arthur, PE DN: cn=Blake Arthur, PE, c=US, email=barthur@aep.com Date: 2026.07.28 14:29:11 -04'00' DATE _____

Blake Arthur, P.E.

APPROVED BY David Anthony Miller DATE 08.10.2026

David Miller, P.E.

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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Log of revisions

Revision No.	Date	Change
1	July 2026	Findings of timber pile assessment incorporated in safety factor assessment.

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 Ash Pond at the Cabin Creek Site is subjected to this rule.

2.0 DESCRIPTION OF THE CCR UNIT

The Cabin Creek site is located near Chelyan, West Virginia, near the intersection of MacCorkle Ave SE and Cabin Creek Rd., Cabin Creek, WV 25035. The latitude/longitude of the facility is: 38°11'52"N / 81°29' 6" W. The Ash Pond is approximately 19 acres. The Cabin Creek Power Plant was placed in service in 1914, stopped generating electricity in 1977, retired in 1981 and the site was razed in 1985-1987. The Ash Pond was constructed sometime around 1956.

The Ash Pond is located near the Kanawha River on the west side of the plant property. The Ash Pond was located on the west side of Cabin Creek, and the Power Plant was located on the East side of Cabin Creek.

According to the 1956 U.S. Army Corps Permit application, the ash pond was constructed by building a dike around Cabin Creek Island and filling the former river backchannel with dredged river sediments. To offset the loss of channel volume, approximately 200,000 cubic yards of compensatory dredging was completed adjacent to the Cabin Creek Island within the main river channel. The dike included a 650-foot-long southwest section reinforced with 35-foot-deep wood piling and rockfill, and a similarly long northeast section protecting the shoreline with riprap armor. The exterior slope of the dike was designed at 2H:1V and the interior slope was designed at 1.5H:1V. The 2H:1V slope extends below the navigational pool of the Marmet Lock and Dam (590 ft-msl) to an elevation of 575 ft-msl, 15 below the normal pool.

The former discharge structure is a pipe and riser type structure located at the western end of the Ash Pond. The riser structure is made of concrete and has grooves for receiving stop logs for controlling the pool level when the pond was active. The outlet pipe is a 24-inch-diameter reinforced concrete pipe. The discharge structure no longer functions, and the facility no longer impounds water.

3.0 SAFETY FACTOR ASSESSMENT 257.73(e)

The Initial Safety Factor Assessment was prepared by WSP USA, Inc. Available records for the Cabin Creek Ash Pond indicate timber piling was installed along the normal pool of the Kanawha River for at least a partition of the dikes length. Remnants of the timber pile system were observed were assessed by Wood Research and Development in June 2026. Stress wave tests, pile echo tests , species identification and specific gravity tests were performed on the timber piles. The results of the timber pile assessment were incorporated into an updated Safety Factor Assessment- included as Attachment A.

The results of the safety factor assessment indicate that one section of the Cabin Creek Ash Pond Dam does not meet the required safety factors of the 40 CFR 257.73 (e) for global stability. Specifically, this section of the dam does not appear to have timber piling present based on site inspections and historical documents. AEP is evaluating options for addressing this deficiency.

ATTACHMENT A

Safety Factor Assessment Report
July 2026

REPORT

Initial Safety Factor Assessment

Cabin Creek Ash Pond, Former Cabin Creek Power Plant, Cabin Creek, West Virginia

Submitted to:

Ms. Jane Petrie

American Electric Power Company, Inc.
Project Manager
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Submitted by:

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US0041868.3924

July 9, 2026

Distribution List

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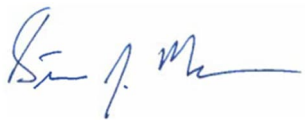
CERTIFICATION

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

July 9, 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 Cabin Creek Ash Pond, an inactive Coal Combustion Residuals (CCR) surface impoundment. This assessment has been prepared in accordance with the requirements of 40 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 Cabin Creek Ash Pond 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 Cabin Creek Power Station (Plant), owned and operated by the Appalachian Power Company, is located along West Virginia State Route 61, adjacent to the Kanawha River (Figure 1). Cabin Creek is located between the former Plant and the ash pond and discharges into the Kanawha River, as shown in Figure 2 and Figure 3. Operations at the Plant began in 1914 and ceased in 1977. The Plant and associated structures were demolished in 1981, and the ash pond was closed in compliance with applicable state regulations at that time. The ash pond covers approximately 19 acres and is surrounded by commercial and residential properties.

1.3 Scope of Assessment

For this initial assessment, WSP conducted a field investigation program to characterize the subsurface conditions at the ash pond site. The results of the field program are presented in the Geotechnical Data Report (GDR) for the Cabin Creek Power Station (WSP 2026). This assessment includes an evaluation of the factors of safety assessment for the Cabin Creek Ash Pond.

2 BACKGROUND

The former Cabin Creek Power Station utilized a single surface impoundment, the Cabin Creek Ash Pond, to manage sluiced ash. This impoundment is located west of Cabin Creek and is classified as a legacy CCR impoundment under the United States Environmental Protection Agency's (USEPA) Legacy CCR Final Rule (May 28, 2024), which established regulatory requirements for legacy CCR surface impoundments and CCR management units.

The Cabin Creek Ash Pond is an inactive impoundment, which no longer impounds surface water, surrounded by an earthen berm that ranges in elevation from approximately 590 to 620 feet Mean Sea Level (MSL). The thickness of the ash within the bermed area ranges between approximately 5 and 33 feet. Only limited design and construction drawings are available, primarily from a United States Army Corps of Engineers (USACE) permit application from the 1950s (APPENDIX A).

The ash pond was constructed in the 1950s by dredging alluvial materials and sediments from the Kanawha River north of Cabin Creek Island and placing the dredged materials in the back channel on the southern side of the island. Historical drawings and boring logs indicate that these dredged materials were used to construct the perimeter dike to an approximate elevation of 605 feet MSL. The original USACE drawings show a timber pile retaining wall along a portion of the northern dike adjacent to the Kanawha River. Initial field observations (GDR, 2026) indicate that the north side of the pilings has filled in with sediments, and the top of the wall no longer

extends above the waterline in the river. During the annual inspection of the ash pond performed in September 2025, AEP located remnants of these timber piles along the shoreline of the ash pond's northern dike. The images 1a and 1b (shown below) were taken in the western portion of the ash pond's northern dike and show the piles at the approximate locations depicted in the 1950 USACE drawings (APPENDIX A).

Image 1a

Image 1b

Images 1a and 1b: Timber piles along the shoreline in the western half of the ash pond's northern dike

The Cabin Creek Ash Pond received CCR material from the former Plant between approximately 1958 and 1977, after which time the Plant ceased operation. During its active period, bottom ash and fly ash were sluiced into the pond, and water levels were controlled via an outfall structure located on the western end of the ash pond that conveyed stormwater from the impoundment to the Kanawha River. Based on a review of the embankment boring logs, it appears that the perimeter dike was raised at least twice to its current elevation of approximately 620 feet MSL. Initially, CCR materials were used to raise the height of the perimeter dike, followed by the use of CCR-Soil Mix to construct the uppermost section of the dike. CCR–Soil Mix refers to Earthfill material that contains inclusions of CCR. It is not meant to imply a uniformly blended mix of CCR and Earthfill.

Prior to 2025, the Cabin Creek Ash Pond was overgrown with vegetation and trees. In April and May 2025, AEP cleared vegetation from the top of the ash pond in preparation for subsequent investigations. Additionally, later in 2025, AEP cleared pathways on the northern embankment adjacent to the Kanawha River to facilitate access to this portion of the embankment for future inspections.

In June 2026, WSP retained Wood Research & Development, Inc. (WRD) to perform an inspection of the timber piles along the shoreline of the ash pond's northern dike. The results of the inspection presenting the number of piles identified in the field, the pile dimensions and estimated shear strength of the piles are provided in APPENDIX B. The approximate extent of the timber piles along the Kanawha River is shown in Figure 2.

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 Cabin Creek Ash Pond were conducted to determine whether the calculated FoS for the critical cross-sections of the perimeter dike meet or exceed 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-sections. The FoS is calculated by dividing the resisting forces by the driving forces along the critical slip surface.

The stability analyses were performed for three cross-sections of the ash pond dike, as shown in Figure 2 and Figure 3. These cross sections were selected as the most susceptible to structural failure based on appropriate engineering considerations, including loading conditions. Slope geometry for the ash pond dike was taken from the topographic survey performed in 2025 for the Cabin Creek Ash Pond (Figure 2 and Figure 3). WSP's subsurface investigation and previous investigations, analyses, and reports included in the GDR were used to determine the material strength properties, presented in APPENDIX C and Table 1, for the subsurface stratum at the Cabin Creek Ash Pond.

The existing dike primarily consists of recompacted sandy and silty alluvial soil and CCR materials, which are generally non-cohesive. The exterior slopes of the dike vary across the site, with the slopes of the most critical sections steeper than 1.7H:1V. The maximum height of the dike is approximately 30 feet. Based on WRD's

investigation (APPENDIX B), timber piles are present along the western half of the shoreline of the ash pond's northern dike. They are approximately 10 inches in diameter and average approximately 33 feet below the mudline. The timber piles represent structural elements that support the ash pond and increases the overall stability of the pond. As such, the piles were considered in the analysis performed for Section A-A.

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 Cabin Creek Ash Pond is an inactive impoundment that no longer impounds surface water; therefore, a maximum surcharge pool loading condition is not applicable.

WSP developed a potentiometric map for the Cabin Creek Ash Pond based on groundwater observations from soil borings drilled at the site (GDR, 2026) as shown in Figure 4. The interpreted potentiometric surface, together with an assumed river elevation of 590 feet MSL, was used to define the phreatic surface conditions for each cross-section evaluated under long-term static loading.

4.2.2 Maximum Surcharge Pool Condition

As noted in Section 4.2.1, the Cabin Creek Ash Pond is an inactive impoundment and no longer retains surface water. Consequently, a maximum surcharge pool loading condition, which typically represents a temporary increase in water level within the impoundment due to storm inflow or operational surcharging, is not applicable for this site.

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:

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

Thus, the pseudo-static condition was modeled 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 loading.

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 obtained during the geotechnical exploration program. FoS against liquefaction were calculated for all borings performed by WSP. 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, 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 Mw = 5.97, 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 D.

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 sections evaluated in accordance with 40 CFR §257.73(e)(1)(i)–(iv) of the CCR Rule and detailed slope stability analyses outputs are provided in APPENDIX E.

The analyses evaluated both local and global failure mechanisms under long-term static and pseudo-static seismic loading conditions. Local stability results are governed by relatively shallow failure surfaces near the crest and slope face, whereas global stability results are associated with deeper failure mechanisms extending into the embankment and underlying foundation materials. Local failures are typically limited in extent and can generally be addressed through routine inspections and proper maintenance to address required slope repairs as necessary. In contrast, global failure mechanisms are more representative of the overall stability of the embankment system and are therefore considered more critical to long-term performance. AEP is expected to implement diligent inspection protocols at the Cabin Creek Ash Pond until the site is closed in the coming years.

For Section A-A, the critical failure mechanism is a shallow surficial failure along the exterior slope. Under long-term static loading, the global stability criterion is satisfied due to the stabilizing influence of the existing timber piles located at the waterline along the slope, which provide resistance to deeper-seated failure planes. Under pseudo-static seismic loading, both local and global stability criteria are satisfied. The critical shallow failure zone for this cross section is due to localized steeper slopes present within the embankment that occurred over time due to erosion.

Section B-B is characterized by relatively steep exterior slopes, resulting in reduced stability for both shallow and deep-seated failure mechanisms. Consequently, both local and global stability criteria are not satisfied under either long-term static or pseudo-static seismic loading conditions. As shown in Figure 2, the extent of the timber piles on the northern dike do not extend to this cross section; therefore, the resulting factors of safety are lower than those calculated for Section A-A.

For Section C-C, global stability criteria are satisfied under long-term static and pseudo-static seismic loading, indicating adequate resistance to deep-seated failure mechanisms. Local stability is also satisfactory under pseudo-static loading and is only marginally below the long-term static criterion.

Review of the Sections A-A and B-B indicate that the lower FoS values for the local analyses are associated with the steep exterior slopes of the perimeter dike as shown in Figure 5.

Given the facility's historical performance, ongoing inspection program, and planned closure activities in the near term, slightly lower factors of safety associated with localized failure mechanisms may be considered acceptable when evaluated in the context of overall embankment stability.

6 REFERENCES

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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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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

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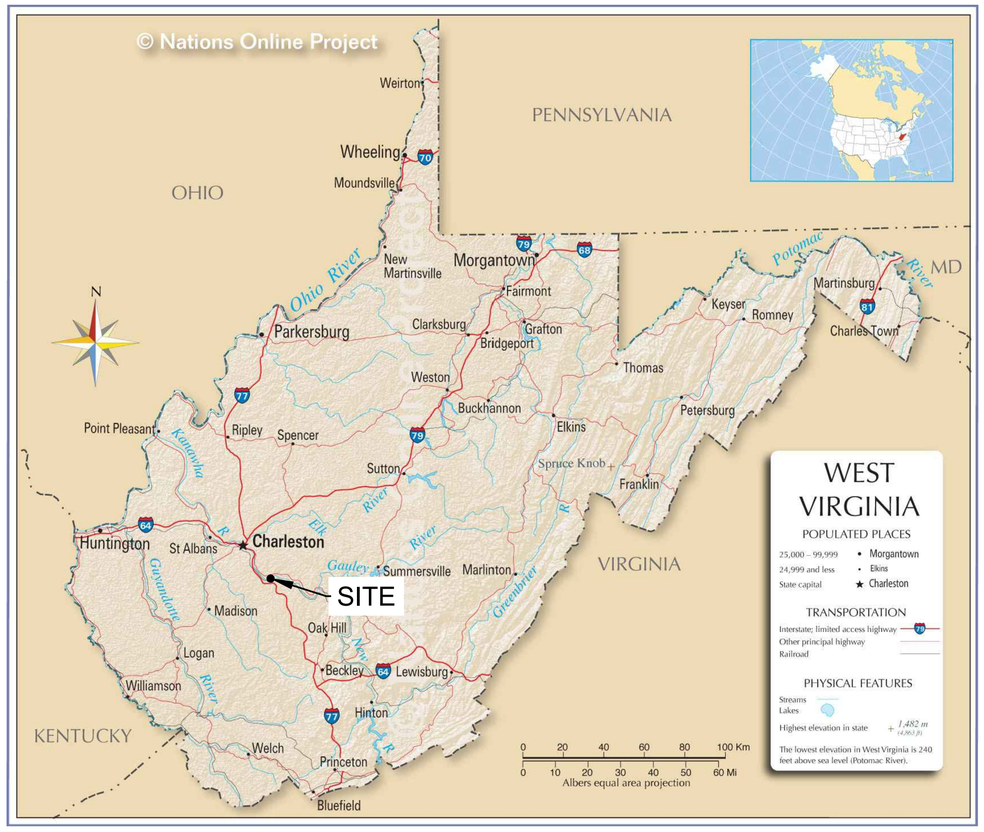
NG/SM/ng

https://wsponlinenam.sharepoint.com/sites/us-aep-cabin-creek-c/shared documents/5 technical work/safety factor assessment/sfa_cabincreek_ashpond_ccr_engsupport.docx

Figures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Keyser
Romney
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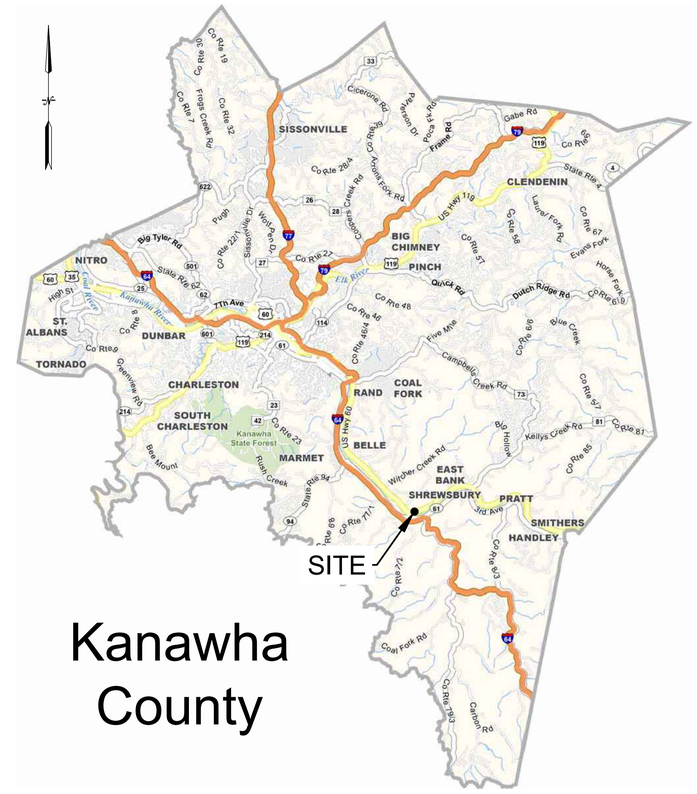
Petersburg
Point Pleasant
Ripley
Spencer
Sutton
Spruce Knob
Franklin
Huntington
St Albans
Charleston
SITE
Summersville
Marlinton
Madison
Oak Hill
Logan
Beckley
Lewisburg
Williamson
Hinton
Welch
Princeton
Bluefield
Ohio River
Potomac River
Kanawha R.
Elk River
Gauley R.
Greenbrier R.
Guyandotte River
New River
WEST VIRGINIA
POPULATED PLACES
25,000 – 99,999 • Morgantown
24,999 and less • Elkins
State capital ★ Charleston
TRANSPORTATION
Interstate; limited access highway
Other principal highway
Railroad
PHYSICAL FEATURES
Streams
Lakes
Highest elevation in state 1,482 m (4,863 ft)
The lowest elevation in West Virginia is 240 feet above sea level (Potomac River).
0 20 40 60 80 100 Km
0 10 20 30 40 50 60 Mi
Albers equal area projection


SISSONVILLE
CLENDENIN
BIG CHIMNEY
PINCH
NITRO
ST. ALBANS
DUNBAR
TORNADO
CHARLESTON
SOUTH CHARLESTON
Kanawha State Forest
MARMET
BELLE
RAND
COAL FORK
EAST BANK
SHREWSBURY
PRATT
SMITHERS
HANDLEY
SITE
Kanawha River
Elk River
Big Tyler Rd
7th Ave
Frame Rd
Gabe Rd
Quick Rd
Dutch Ridge Rd
Five Mile
Campbell Creek Rd
Kellys Creek Rd
Witcher Creek Rd
Coal Fork Rd
Carbon Rd
US Hwy 119
US Hwy 60
US Hwy 61
Co Rte 19
Co Rte 32
Co Rte 27
Co Rte 48
Co Rte 85
Co Rte 87
Co Rte 71/1
Co Rte 71/3
Co Rte 23
Co Rte 9
Co Rte 6
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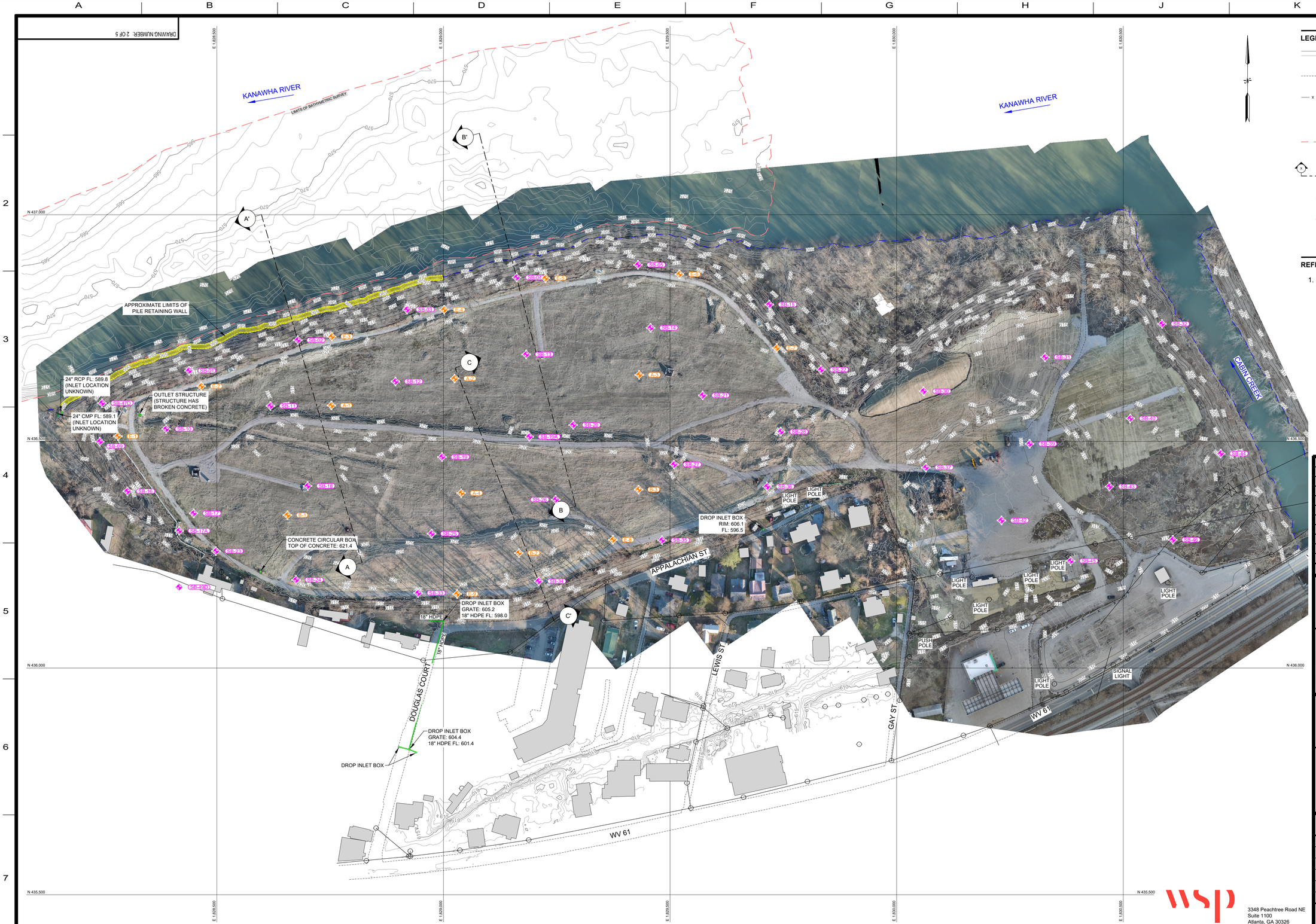

LEGEND

- EXISTING CONTOURS
- EXISTING ROADS
- EXISTING FENCE
- EXISTING UTILITY POLES
- LIMITS OF BATHYMETRIC SURVEY
- SLOPE STABILITY SECTION LOCATIONS
- WSP BORING LOCATIONS
- AEP/EHS GEOPROBE LOCATIONS

REFERENCES

1. EXISTING TOPOGRAPHY, BATHYMETRY AND AERIAL IMAGERY PROVIDED BY AMERICAN ELECTRIC POWER.

100 0 100 200 FT

DATE	NO.	DESCRIPTION	APPD.

REVISIONS

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APPALACHIAN POWER COMPANY

CABIN CREEK ASH POND

CABIN CREEK WEST VIRGINIA

SLOPE STABILITY SECTION LOCATION PLAN WITH AERIAL

UNIT:	DRAWING NUMBER:	REV:
FEET	2 OF 5	--

SCALE: 1" = 100'
DR: CCP
CH: NKG
SUP: NKG
ENG: SJM
DATE: 06/30/2026
APPROVED BY

wsp
3348 Peachtree Road NE
Suite 1100
Atlanta, GA 30326

AEP AMERICAN ELECTRIC POWER

AEP SERVICE CORP.
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

PLOT DATE: 7/9/2026
PLOT TIME: 10:28 AM
PLOTTED BY: POWELL, CHRIS

G:\American Electric Power\Cabin Creek Site\Factor of Safety Assessment\Production\263 - Slope Stability Section Locations.dwg LAYOUT: 2 SAVED: 7/9/2026 10:23 AM PAGESETUP: ---- PLOTTED: 7/9/2026 POWELL, CHRIS

DRAWING NUMBER 3 OF 5

KANAWHA RIVER

LIMITS OF BATHYMETRIC SURVEY

APPROXIMATE LIMITS OF PILE RETAINING WALL

24" RCP FL: 589.8 (INLET LOCATION UNKNOWN)

OUTLET STRUCTURE (STRUCTURE HAS BROKEN CONCRETE)

24" CMP FL: 589.1 (INLET LOCATION UNKNOWN)

CONCRETE CIRCULAR BOX TOP OF CONCRETE: 621.4

DROP INLET BOX GRATE: 605.2 18" HDPE FL: 598.0

18" HDPE

DROP INLET BOX GRATE: 604.4 18" HDPE FL: 601.4

DROP INLET BOX

DROP INLET BOX RIM: 606.1 FL: 596.5

LIGHT POLE

PUSH POLE

SIGNAL LIGHT

APPALACHIAN ST

DOUGLAS COURT

LEWIS ST

GAY ST

WV 61

CABIN CREEK

N 437,000

N 436,500

N 436,000

N 435,500

E 1,828,500

E 1,829,000

E 1,829,500

E 1,830,000

E 1,830,500

A' B' C' A B C

LEGEND

- EXISTING CONTOURS
- EXISTING ROADS
- EXISTING FENCE
- EXISTING UTILITY POLES
- LIMITS OF BATHYMETRIC SURVEY
- SLOPE STABILITY SECTION LOCATIONS
- WSP BORING LOCATIONS
- AEP/EHS GEOPROBE LOCATIONS

REFERENCES

1. EXISTING TOPOGRAPHY AND BATHYMETRY PROVIDED BY AMERICAN ELECTRIC POWER.

100 0 100 200 FT

DATE	NO.	DESCRIPTION	APPD.

REVISIONS

THIS DRAWING IS CLASSIFIED AS:

REFERENCE AEP's CORPORATE INFORMATION SECURITY POLICY

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APPALACHIAN POWER COMPANY

CABIN CREEK ASH POND

CABIN CREEK WEST VIRGINIA

SLOPE STABILITY SECTION LOCATION PLAN

UNIT:	DRAWING NUMBER:	REV:
FEET	3 OF 5	--

SCALE: 1" = 100'
DR: CCP
CH: NKG
SUP: NKG
ENG: SJM
DATE: 06/30/2026

APPROVED BY

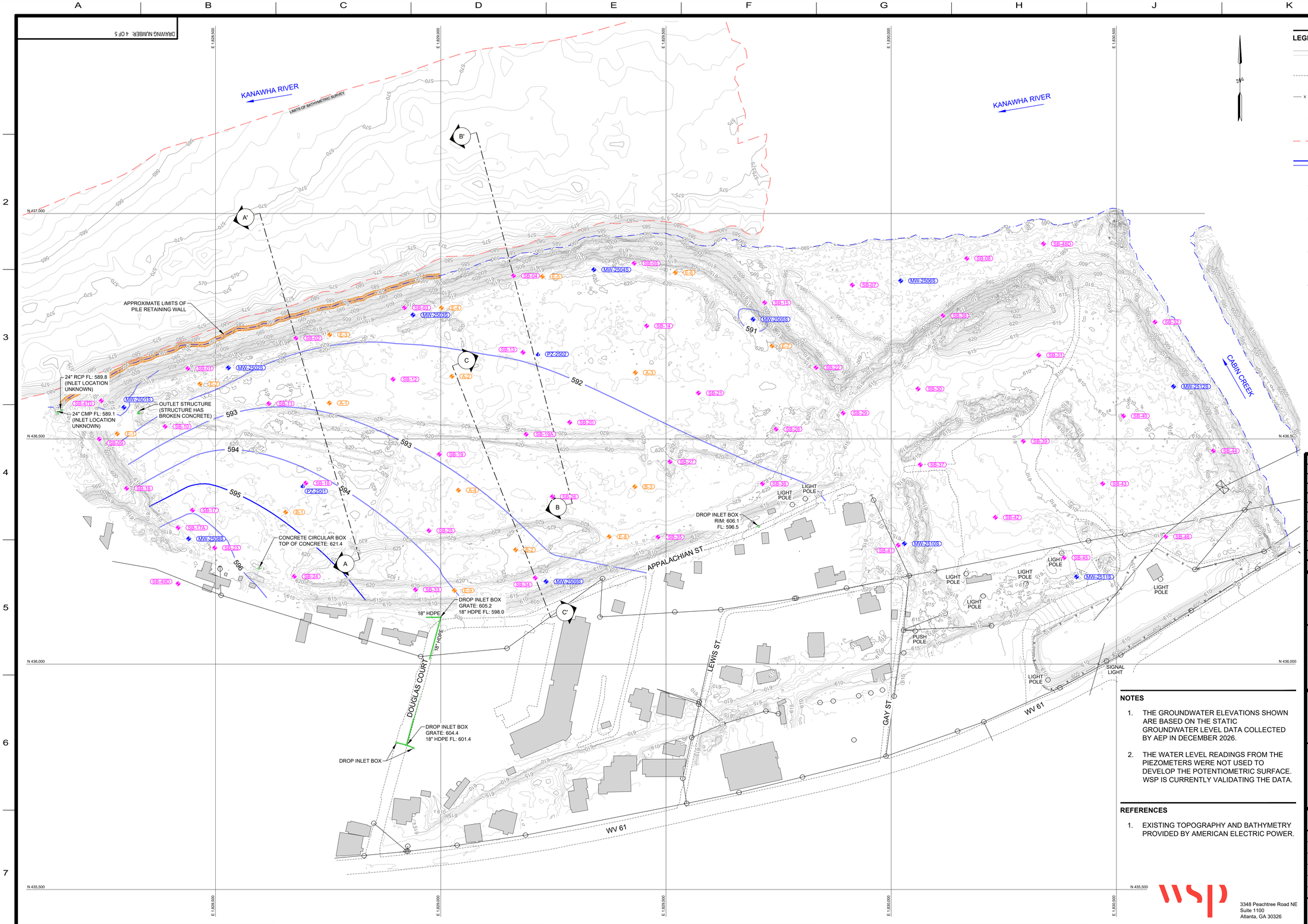
3348 Peachtree Road NE
Suite 1100
Atlanta, GA 30326

AEP AMERICAN ELECTRIC POWER

AEP SERVICE CORP.
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

PLOT DATE: 7/9/2026
PLOT TIME: 10:23 AM
BY: POWELL, CHRIS

G:\American Electric Power\Cabin Creek Site\Factor of Safety Assessment\Production\263 - Slope Stability Section Locations.dwg LAYOUT: 3 SAVED: 7/9/2026 10:23 AM PAGESETUP: ---- PLOTTED: 7/9/2026 POWELL, CHRIS



LEGEND

- EXISTING CONTOURS
- EXISTING ROADS
- EXISTING FENCE
- EXISTING UTILITY POLES
- LIMITS OF BATHYMETRIC SURVEY
- GROUNDWATER CONTOURS
- WSP BORING LOCATIONS (A-, B-, AND E- SERIES), 2025
- AEP/EHS GEOPROBE LOCATIONS (SB-XX), 2025
- AEP/EHS MONITORING WELLS (MW-25XXS), 2025
- AEP/EHS PIEZOMETERS (SB-XX), 2025

NOTES

1. THE GROUNDWATER ELEVATIONS SHOWN ARE BASED ON THE STATIC GROUNDWATER LEVEL DATA COLLECTED BY AEP IN DECEMBER 2026.
2. THE WATER LEVEL READINGS FROM THE PIEZOMETERS WERE NOT USED TO DEVELOP THE POTENTIOMETRIC SURFACE. WSP IS CURRENTLY VALIDATING THE DATA.

REFERENCES

1. EXISTING TOPOGRAPHY AND BATHYMETRY PROVIDED BY AMERICAN ELECTRIC POWER.

wsp
3348 Peachtree Road NE
Suite 1100
Atlanta, GA 30326

DATE	NO.	DESCRIPTION	APPD.

REVISIONS

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APPALACHIAN POWER COMPANY

CABIN CREEK ASH POND

CABIN CREEK — WEST VIRGINIA

POTENTIOMETRIC MAP WITH SLOPE STABILITY SECTION LOCATIONS

UNIT:	DRAWING NUMBER:	REV:
FEET	4 OF 5	--

SCALE: 1" = 100'

APPROVED BY: DR: CCP; CH: NG; SUP: NG; ENG: SJM; DATE: 06/30/2026

AEP AMERICAN ELECTRIC POWER

AEP SERVICE CORP.
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

G:\American Electric Power\Cabin Creek Site\Factor of Safety Assessment\Production\4 - Potentiometric Map.dwg LAYOUT: 4 SAVED: 7/9/2026 10:27 AM PAGESETUP: ---- PLOTTED: 7/9/2026 POWELL, CHRIS

DRAWING NUMBER: 5 OF 5

SLOPES TABLE

SLOPE RANGE	COLOR
FLATTER THAN 2H:1V	
2H:1V to 1.875H:1V	
1.875H:1V to 1.75H:1V	
1.75H:1V to 1.625H:1V	
1.625H:1V to 1.5H:1V	
1.5H:1V to 1.375H:1V	
1.375H:1V to 1.25H:1V	
STEEPER THAN 1.25H:1V	

KANAWHA RIVER

LIMITS OF BATHYMETRIC SURVEY

24" RCP FL: 589.8 (INLET LOCATION UNKNOWN)

24" CMP FL: 589.1 (INLET LOCATION UNKNOWN)

OUTLET STRUCTURE (STRUCTURE HAS BROKEN CONCRETE)

CONCRETE CIRCULAR BOX TOP OF CONCRETE: 621.4

DROP INLET BOX GRATE: 605.2 18" HDPE FL: 598.0

18" HDPE

DROP INLET BOX GRATE: 604.4 18" HDPE FL: 601.4

DROP INLET BOX

DROP INLET BOX RIM: 606.1 FL: 596.5

LIGHT POLE

PUSH POLE

SIGNAL LIGHT

APPALACHIAN ST

DOUGLAS COURT

LEWIS ST

GAY ST

WV 61

CABIN CREEK

N 437,000

N 436,500

N 436,000

N 435,500

E 1,828,500

E 1,829,000

E 1,829,500

E 1,830,000

E 1,830,500

LEGEND

- EXISTING CONTOURS
- EXISTING ROADS
- EXISTING FENCE
- EXISTING UTILITY POLES
- LIMITS OF BATHYMETRIC SURVEY
- WSP BORING LOCATIONS (A-, B-, AND E- SERIES), 2025
- AEP/EHS GEOPROBE LOCATIONS (SB-XX), 2025
- AEP/EHS MONITORING WELLS (MW-25XXS), 2025
- AEP/EHS PIEZOMETERS (SB-XX), 2025
- APPROX. FORMER BACK CHANNEL

REFERENCES

1. EXISTING TOPOGRAPHY AND BATHYMETRY PROVIDED BY AMERICAN ELECTRIC POWER.

100 0 100 200 FT

DATE	NO.	DESCRIPTION	APPD.

REVISIONS

THIS DRAWING IS CLASSIFIED AS:

REFERENCE AEP's CORPORATE INFORMATION SECURITY POLICY

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APPALACHIAN POWER COMPANY

CABIN CREEK ASH POND

CABIN CREEK WEST VIRGINIA

SLOPE GRADIENTS

UNIT:	DRAWING NUMBER:	REV:
FEET	5 OF 5	--

SCALE: 1" = 100'

APPROVED BY: DR: CCP; CH: NG; SUP: NG; ENG: SJM; DATE: 06/30/2026

wsp
3348 Peachtree Road NE
Suite 1100
Atlanta, GA 30326

AEP AMERICAN ELECTRIC POWER
AEP SERVICE CORP.
1 RIVERSIDE PLAZA
COLUMBUS, OH 43215

COPYDATE: 7/1/2026
PLOT TIME: 9:36 AM
PLOTTED BY: CHRIS POWELL

G:\American Electric Power\Cabin Creek Site\Factor of Safety Assessment\Production\5 - Slope Gradients.dwg LAYOUT: 5 SAVED: 7/1/2026 9:15 AM PAGESETUP: ---- PLOTTED: 7/1/2026 POWELL, CHRIS

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)
Fill and Embankment Materials					
CCR-Soil Mix (Embankment)	85	30	0	-	-
CCR (Embankment)	85	30	0	-	-
Dredge Spoil	100	28	0	-	-
Earthfill	125	32	100	-	-
Coal Combustion Residuals					
CCR-Soil Mix (Impounded)	75	28	0	-	-
Sluiced CCR	75	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

Slope Failure	Scenario	Long-term static (CCR Rule Threshold FoS ≥1.50)[3]	Pseudo-static seismic loading (CCR Rule Threshold FoS ≥1.00)[3]
Local[1]	A-A	**1.00**	1.04
	B-B	**1.15**	**0.78**
	C-C	**1.42**	1.19
Global[2]	A-A	1.56	1.04
	B-B	**1.18**	**0.78**
	C-C	2.27	1.87

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

3. Results shown in bold do not meet the minimum Factor of Safety (FoS) requirement per CCR Rule.

APPENDIX A

Historical Drawings

800.6(Kanawha R. - Dike & Dredging)-14.91

11 January 1956

Appalachian Electric Power Company
30 Church Street
New York 6, New York

Gentlemen:

In accordance with your written request dated 6 January 1956, the revised plans hereto attached are approved to supersede the plans for the works authorized by the Secretary of the Army, in letter dated 14 April 1954, from the Acting District Engineer at Huntington, West Virginia, to construct a dike around Cabin Creek Island, dredge approximately 200,000 cubic yards of material along the island and fill the back channel of the island, in the Kanawha River, at Cabin Creek Junction, West Virginia, 74.0 miles above the mouth of the river.

The revised plans provide for the addition of a line of wood piles, with rock fill landward of the piling, for a distance of 650 feet along the downstream riverward portion of the dike, and stone riprap for a distance of 650 feet along the upstream riverward portion of the dike.

The conditions to which the work is made subject remain in full force and effect.

Please acknowledge receipt of this letter.

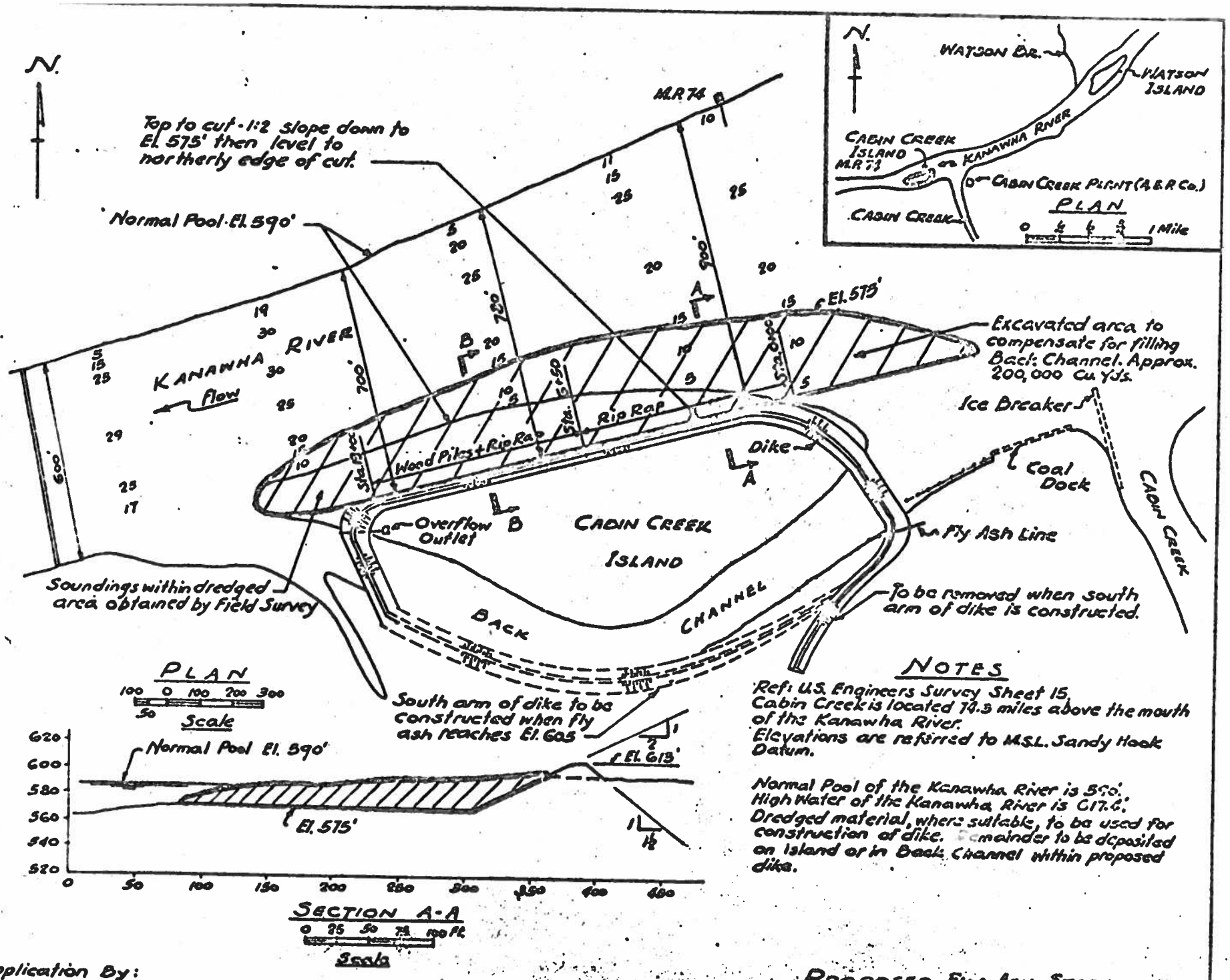
By authority of the Secretary of the Army:

G. T. Derby

G. T. DERBY
Colonel, Corps of Engineers
District Engineer

Drawing No. S-SK 10153,
in 3 sheets, revised
6 January 1956, attached

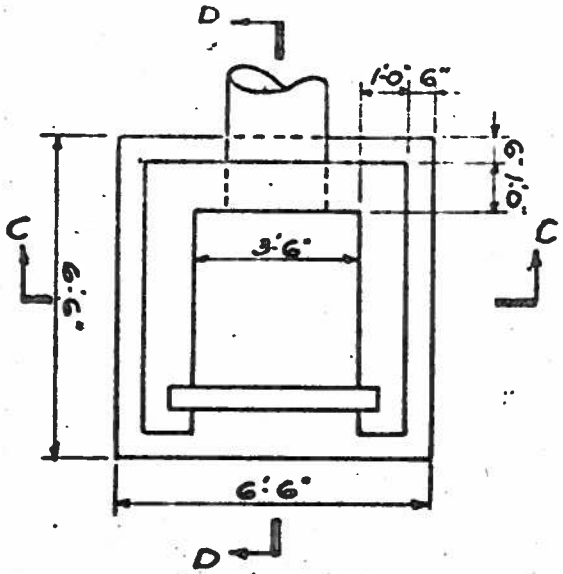
800.6 (Kanawha R. - Dike & Dredging) - 14.91



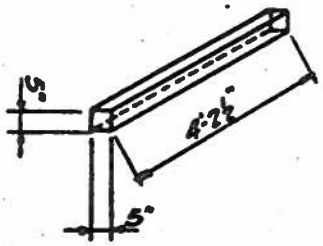
Application By:
Appalachian Electric Power Company.
Cabin Creek, West Virginia.
October 21, 1953

Revised 2-18-54

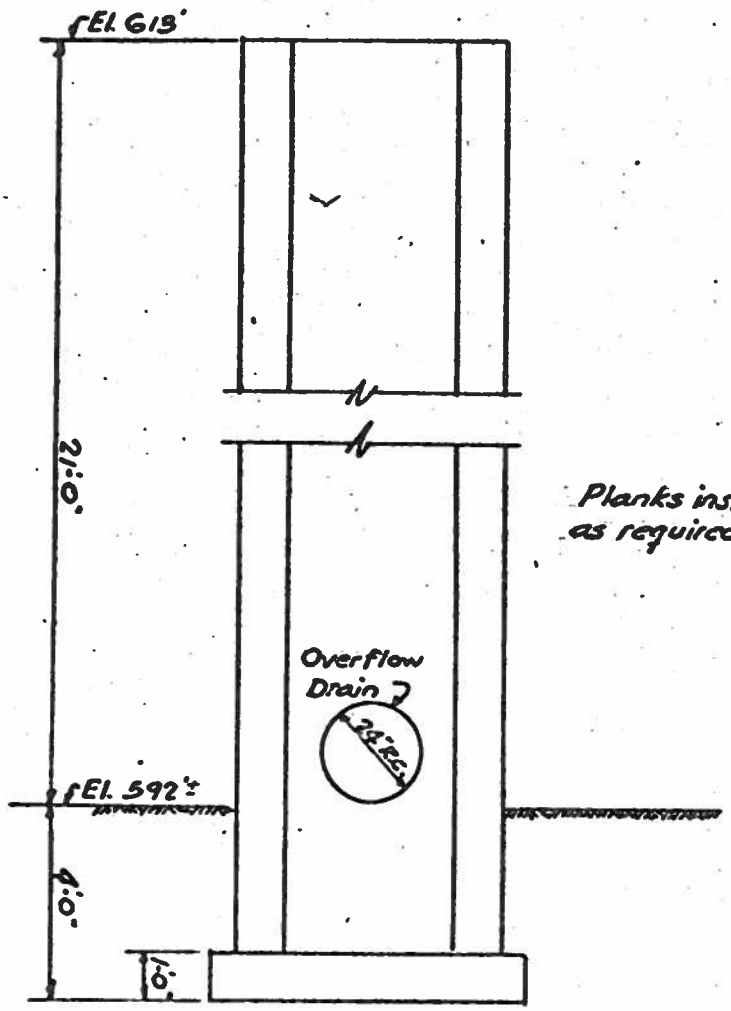
PROPOSED FLY ASH STORAGE
AT CABIN CREEK ISLAND
CABIN CREEK POWER PLANT



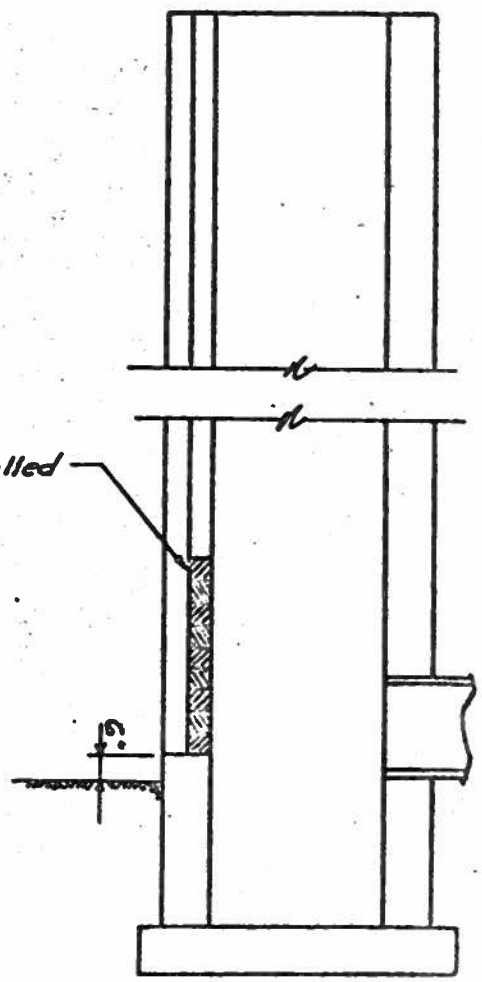
PLAN



DETAIL OF CONCRETE PLANK



SECTION C-C



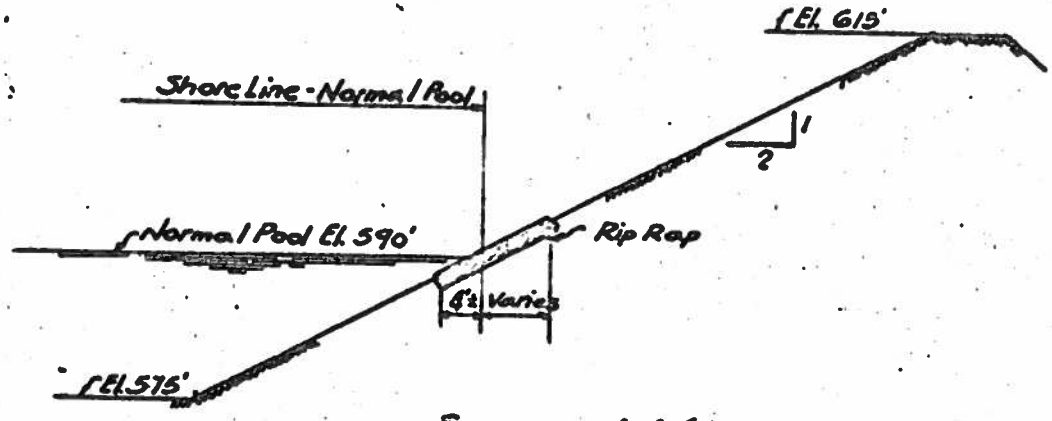
SECTION D-D

DETAIL OF OVERFLOW OUTLET
IN FLY ASH STORAGE AREA
AT CABIN CREEK ISLAND
CABIN CREEK POWER PLANT

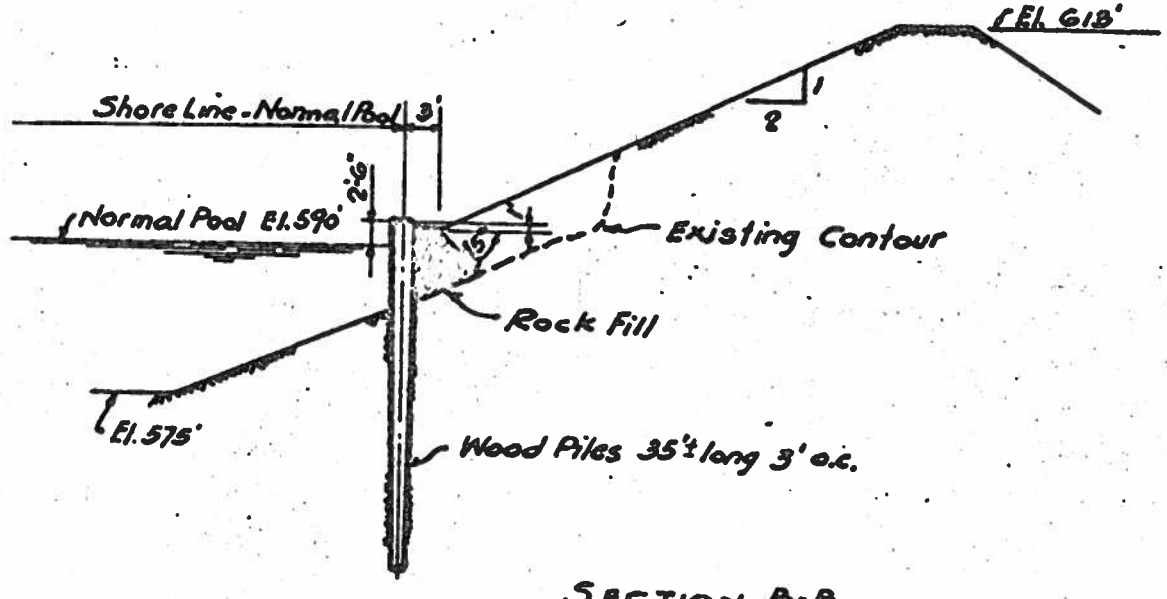
Application By:
Appalachian Electric Power Company
Cabin Creek, West Virginia
October 24, 1958

Revised 1-6-58

Dwg No 5-[illegible]-1958



SECTION A-A (ENLARGED)
Typical For Stations 0+00 - 6+50



SECTION B-B
Typical For Stations 6+50 - 13+00

0 5 10 15 20
Scale

Application By:
Appalachian Electric Power Company
Cabin Creek, West Virginia
January 6, 1956

PROPOSED FLY ASH STORAGE
AT CABIN CREEK ISLAND
CABIN CREEK POWER PLANT

DEPARTMENT OF THE ARMY

NOTE.—It is to be understood that this instrument does not give any property rights either in real estate or material, or any exclusive privileges; and that it does not authorize any injury to private property or invasion of private rights, or any infringement of Federal, State, or local laws or regulations, nor does it obviate the necessity of obtaining *State assent* to the work authorized. IT MERELY EXPRESSES THE ASSENT OF THE FEDERAL GOVERNMENT SO FAR AS CONCERNS THE PUBLIC RIGHTS OF NAVIGATION. (See *Cummings* v. *Chicago*, 188 U. S., 410.)

16—12108-2

PERMIT

Huntington District
Corps of Engineers.
Huntington 16, W. Va.
14 April , 1954 .

Appalachian Electric Power Company
30 Church Street
New York 6, New York

Gentlemen:

Referring to written request dated 19 February 1954 (signed by H. A. Kanner, Vice President) addressed to the District Engineer, Huntington District, Corps of Engineers, U. S. Army, P. O. Box 2127, Huntington, West Virginia,

I have to inform you that, upon the recommendation of the Chief of Engineers, and under the provisions of Section 10 of the Act of Congress approved March 3, 1899, entitled "An act making appropriations for the construction, repair, and preservation of certain public works on rivers and harbors, and for other purposes," you are hereby authorized by the Secretary of the Army.

to construct a dike around Cabin Creek Island, dredge approximately 200,000 cubic yards of material along the island and fill the back channel of the island,
(Here describe the proposed structure or work.)

in the Kanawha River,
(Here to be named the river, harbor, or waterway concerned.)

at Cabin Creek Junction, West Virginia, 74.0 miles above the mouth of the river,
(Here to be named the nearest well-known locality—preferably a town or city—and the distance in miles and tenths from some definite point in the same, stating whether above or below or giving direction by points of compass.)

in accordance with the plans shown in red on the drawing attached hereto marked "Proposed Fly Ash Storage at Cabin Creek Island ###, Dwg. No. S-SX-10153," in two sheets, dated 21 October 1953, revised 16 February 1954,
(Or drawings: give file number or other definite identification marks.)

subject to the following conditions:

(a) That the work shall be subject to the supervision and approval of the District Engineer, Corps of Engineers, in charge of the locality, who may modify or suspend the work at any time, if in his judgment the interests of navigation so require.

(b) That any material dredged in the prosecution of the work herein authorized shall be removed evenly and no large refuse piles, ridges across the bed of the waterway, or deep holes that may have a tendency to cause injury to navigable channels or to the banks of the waterway shall be left. If any pipe, wire, or cable hereby authorized is laid in a trench, the formation of a ridge across the bed of the waterway shall be avoided and the back filling shall be so done as not to increase the cost of future dredging for navigation. Any material to be deposited or dumped under this authorization, either in the waterway or on shore above high-water mark, shall be deposited or dumped at the locality shown on the drawing herein attached, and, if so prescribed thereon, within or behind a good and substantial bulkhead or bulkheads, such as will prevent escape of the material in the waterway. If the material is to be deposited in the harbor of New York, or in its adjacent or tributary waters, or in Long Island Sound, a permit therefor must be previously obtained from the Supervisor of New York Harbor, Whitehall Building, New York City.

(c) That there shall be no unreasonable interference with navigation by the work herein authorized.

(d) That if inspections or any other operations by the United States are necessary in the interest of navigation, all expenses connected therewith shall be borne by the permittee.

(e) That no attempt shall be made by the permittee or the owner to forbid the full and free use by the public of all navigable waters at or adjacent to the work or structure.

(f) That if future operations by the United States require an alteration in the position of the structure or work herein authorized, or if, in the opinion of the Secretary of the Army, it shall cause unreasonable obstruction to the free navigation of said water, the owner will be required upon due notice from the Secretary of the Army, to remove or alter the structural work or obstructions caused thereby without expense to the United States, so as to render navigation reasonably free, easy, and unobstructed; and if, upon the expiration or revocation of this permit, the structure, fill, excavation, or other modification of the watercourse hereby authorized shall not be completed, the owners shall, without expense to the United States, and to such extent and in such time and manner as the Secretary of the Army may require, remove all or any portion of the uncompleted structure or fill and restore to its former condition the navigable capacity of the watercourse. No claim shall be made against the United States on account of any such removal or alteration.

(g) That the United States shall in no case be liable for any damage or injury to the structure or work herein authorized which may be caused by or result from future operations undertaken by the Government for the conservation or improvement of navigation, or for other purposes, and no claim or right to compensation shall accrue from any such damage.

(h) That if the display of lights and signals on any work hereby authorized is not otherwise provided for by law, such lights and signals as may be prescribed by the U. S. Coast Guard, shall be installed and maintained by and at the expense of the owner.

(i) That the permittee shall notify the said district engineer at what time the work will be commenced, and as far in advance of the time of commencement as the said district engineer may specify, and shall also notify him promptly, in writing, of the commencement of work, suspension of work, if for a period of more than one week, resumption of work, and its completion.

(j) That if the structure or work herein authorized is not completed on or before thirty-first(31st) day of December, 19 77, this permit, if not previously revoked or specifically extended, shall cease and be null and void.

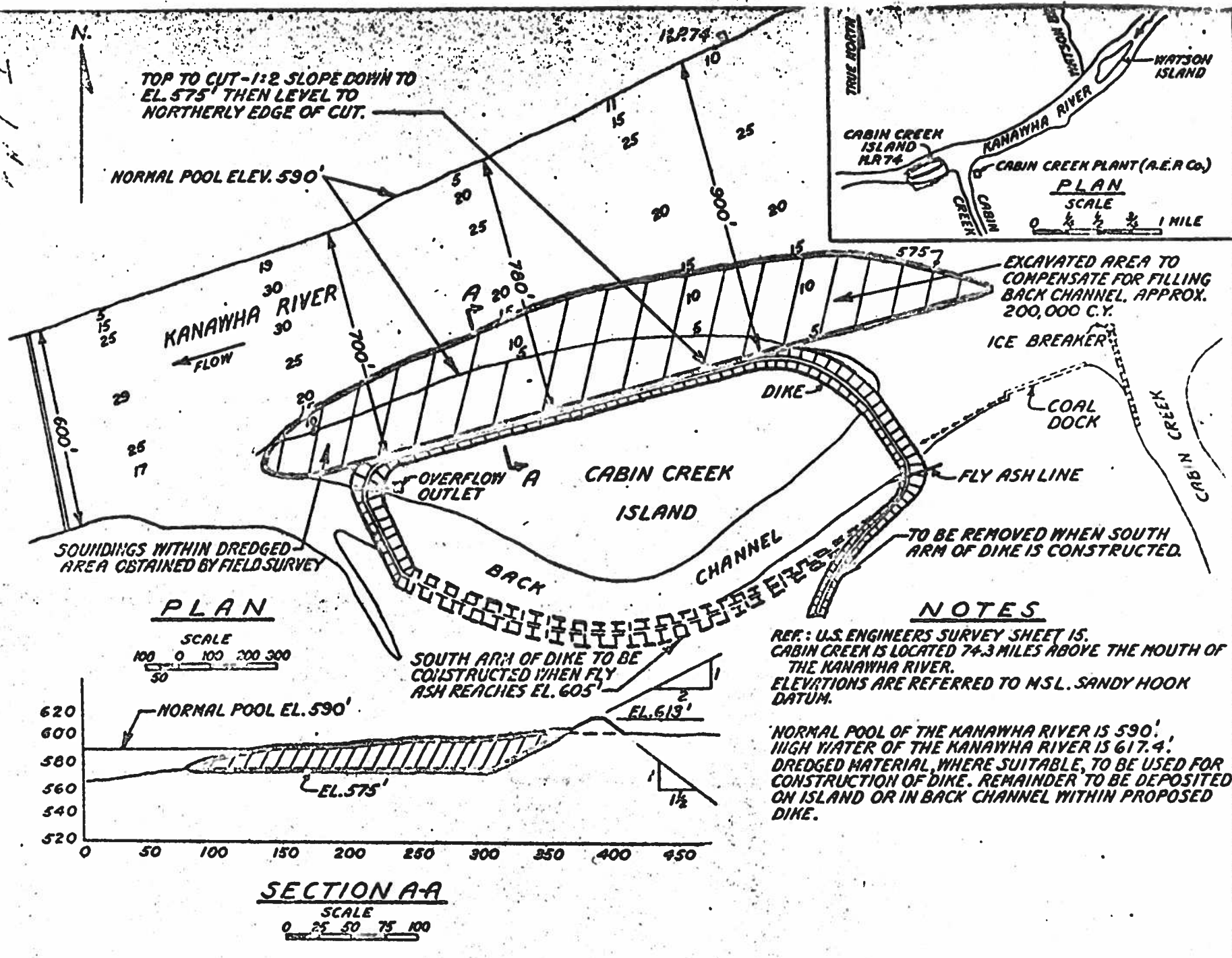
By authority of the Secretary of the Army:

HARRY PECKHAS
Chief Engineering Assistant
Acting District Engineer

ENG FORM 1721 (Civil) This form supersedes ED Form 96, dated 1 Apr 48, which may be used until exhausted.
1 SEP 48

U. S. GOVERNMENT PRINTING OFFICE 16—18168-8

Incl. 1



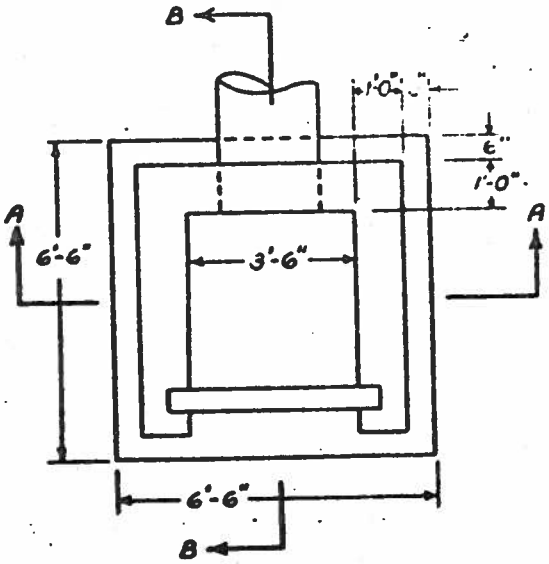
NOTES

REF.: U.S. ENGINEERS SURVEY SHEET 15.
CABIN CREEK IS LOCATED 74.3 MILES ABOVE THE MOUTH OF THE KANAWHA RIVER.
ELEVATIONS ARE REFERRED TO M.S.L. SANDY HOOK DATUM.

NORMAL POOL OF THE KANAWHA RIVER IS 590'.
HIGH WATER OF THE KANAWHA RIVER IS 617.4'.
DREDGED MATERIAL, WHERE SUITABLE, TO BE USED FOR CONSTRUCTION OF DIKE. REMAINDER TO BE DEPOSITED ON ISLAND OR IN BACK CHANNEL WITHIN PROPOSED DIKE.

APPLICATION BY:
APPALACHIAN ELECTRIC POWER COMPANY
CABIN CREEK, WEST VIRGINIA
OCTOBER 21, 1953 REVISED 3-16-54

PROPOSED FLY ASH STORAGE
AT CABIN CREEK ISLAND
CABIN CREEK POWER PLANT
SHEET 1 OF 2 DWG. No. S-SK-10153



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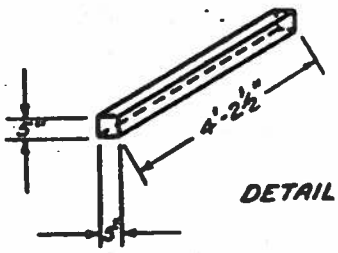
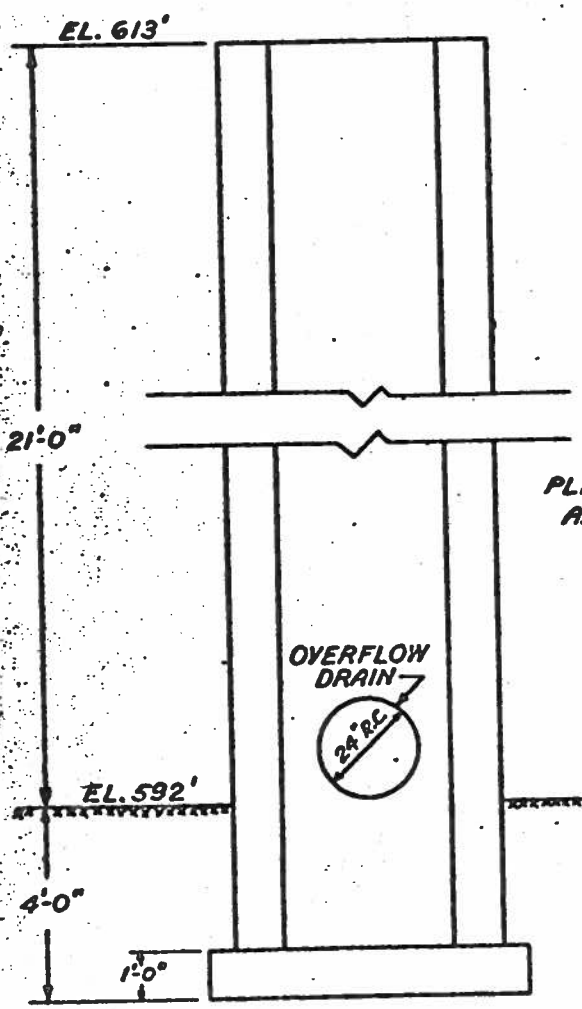
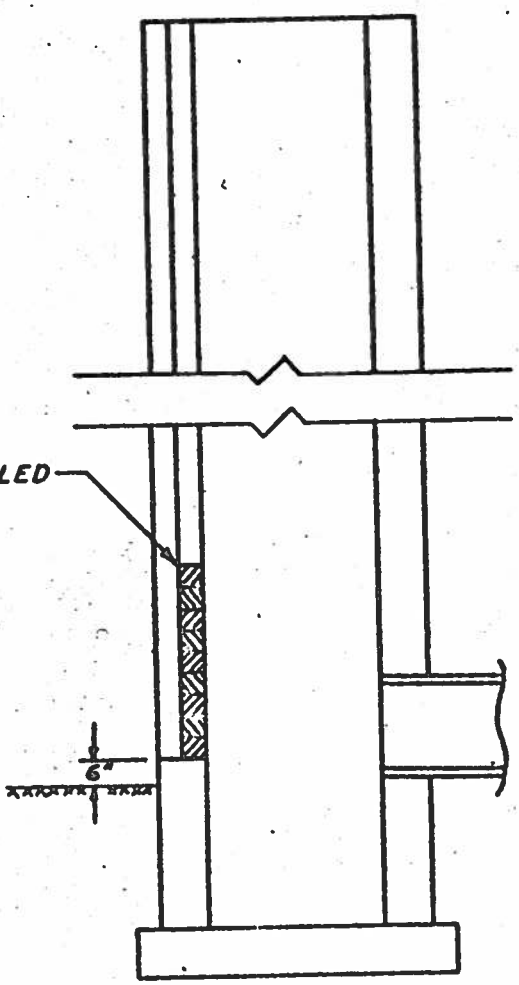


DETAIL OF CONCRETE PLANK

DETAIL OF OVERFLOW OUTLET
FLY ASH STORAGE AREA
CABIN CREEK POWER PLANT
SHEET 2 OF 2 - DWG. No. S-SK-10153
SCALE: ¼" = 1'-0"



SECTION A-A



SECTION B-B

APPLICATION BY:
APPALACHIAN ELECTRIC POWER COMPANY
CABIN CREEK, WEST VIRGINIA
OCTOBER 21, 1953

WAR DEPARTMENT

NOTE.—It is to be understood that this instrument does not give any property rights either in real estate or material, or any exclusive privileges; and that it does not authorize any injury to private property or invasion of private rights, or any infringement of Federal, State, or local laws or regulations, nor does it obviate the necessity of obtaining *State assent* to the work authorized. IT MERELY EXPRESSES THE ASSENT OF THE FEDERAL GOVERNMENT SO FAR AS CONCERNS THE PUBLIC RIGHTS OF NAVIGATION. (See *Cummings* v. *Chicago*, 188 U. S., 410.) 10—12168

PERMIT

United States Engineer Office.
Ohio River Division
Columbus, Ohio August 2, 19 63.

Appalachian Electric Power Company
301 Virginia Street
Charleston, West Virginia

Gentlemen:

Referring to written request dated July 20, 1963 (signed by D. F. Montague, Civil Engineer) addressed to the District Engineer, United States Engineer Office, Huntington, West Virginia

I have to inform you that, upon the recommendation of the Chief of Engineers, and under the provisions of Section 10 of the Act of Congress approved March 3, 1899, entitled "An act making appropriations for the construction, repair, and preservation of certain public works on rivers and harbors, and for other purposes," you are hereby authorized by the Secretary of War,

to construct a 18-inch sanitary sewer outlet and concrete headwall,
(Here describe the proposed structure or work.)

in Kanawha River, left bank,
(Here to be named the river, harbor, or waterway concerned.)

at Cabin Creek, West Virginia, 2400 feet above the mouth of Cabin Creek, 71.0
(Here to be named the nearest well-known locality—preferably a town or city—and the distance in miles and tenths from some definite point in the same, stating whether above or below or giving direction by points of compass.)

miles above the mouth of the Kanawha River,

in accordance with the plans shown on the drawing attached hereto marked "Proposed
(Or drawings; give file number or other definite identification marks.)

Headwall & Outlet For A 18" Sanitary Sewer . . .", in one sheet dated July 16, 1963

subject to the following conditions:

INCL. #2

(a) That the work shall be subject to the supervision and approval of the District Engineer, Engineer Department at Large, in charge of the locality, who may temporarily suspend the work at any time, if in his judgment, the interests of navigation so require.

(b) That any material dredged in the prosecution of the work herein authorized shall be removed evenly, and no large refuse piles, ridges across the bed of the waterway, or deep holes that may have a tendency to cause injury to navigable channels or to the banks of the waterway shall be left. If any pipe, wire, or cable hereby authorized is laid in a trench, the formation of permanent ridges across the bed of the waterway shall be avoided and the back filling shall be so done as not to increase the cost of future dredging for navigation. Any material to be deposited or dumped under this authorization, either in the waterway or on shore above high-water mark, shall be deposited or dumped at the locality shown on the drawing hereto attached, and, if so prescribed thereon, within or behind a good and substantial bulkhead or bulkheads, such as will prevent escape of the material into the waterway. If the material is to be deposited in the harbor of New York, or in its adjacent or tributary waters, or in Long Island Sound, a permit therefor must be previously obtained from the Supervisor of New York Harbor, Army Building, New York City.

(c) That there shall be no unreasonable interference with navigation by the work herein authorized.

(d) That if inspections or any other operations by the United States are necessary in the interests of navigation, all expenses connected therewith shall be borne by the permittee.

(e) That no attempt shall be made by the permittee or the owner to forbid the full and free use by the public of all navigable waters at or adjacent to the work or structure.

(f) That if future operations by the United States require an alteration in the position of the structure or work herein authorized, or if, in the opinion of the Secretary of War, it shall cause unreasonable obstruction to the free navigation of said water, the owner will be required, upon due notice from the Secretary of War, to remove or alter the structural work or obstructions caused thereby without expense to the United States, so as to render navigation reasonably free, easy, and unobstructed; and if, upon the expiration or revocation of this permit, the structure, fill, excavation, or other modification of the watercourse hereby authorized shall not be completed, the owners shall, without expense to the United States, and to such extent and in such time and manner as the Secretary of War may require, remove all or any portion of the uncompleted structure or fill and restore to its former condition the navigable capacity of the watercourse. No claim shall be made against the United States on account of any such removal or alteration.

(g) That the United States shall in no case be liable for any damage or injury to the structure or work herein authorized which may be caused by or result from future operations undertaken by the Government for the conservation or improvement of navigation, or for other purposes, and no claim or right to compensation shall accrue from any such damage.

(h) That if the display of lights and signals on any work hereby authorized is not otherwise provided for by law, such lights and signals as may be prescribed by the U. S. Coast Guard, shall be installed and maintained by and at the expense of the owner.

(i) That the permittee shall notify the said district engineer at what time the work will be commenced, and as far in advance of the time of commencement as the said district engineer may specify, and shall also notify him promptly, in writing, of the commencement of work, suspension of work, if for a period of more than one week, resumption of work, and its completion.

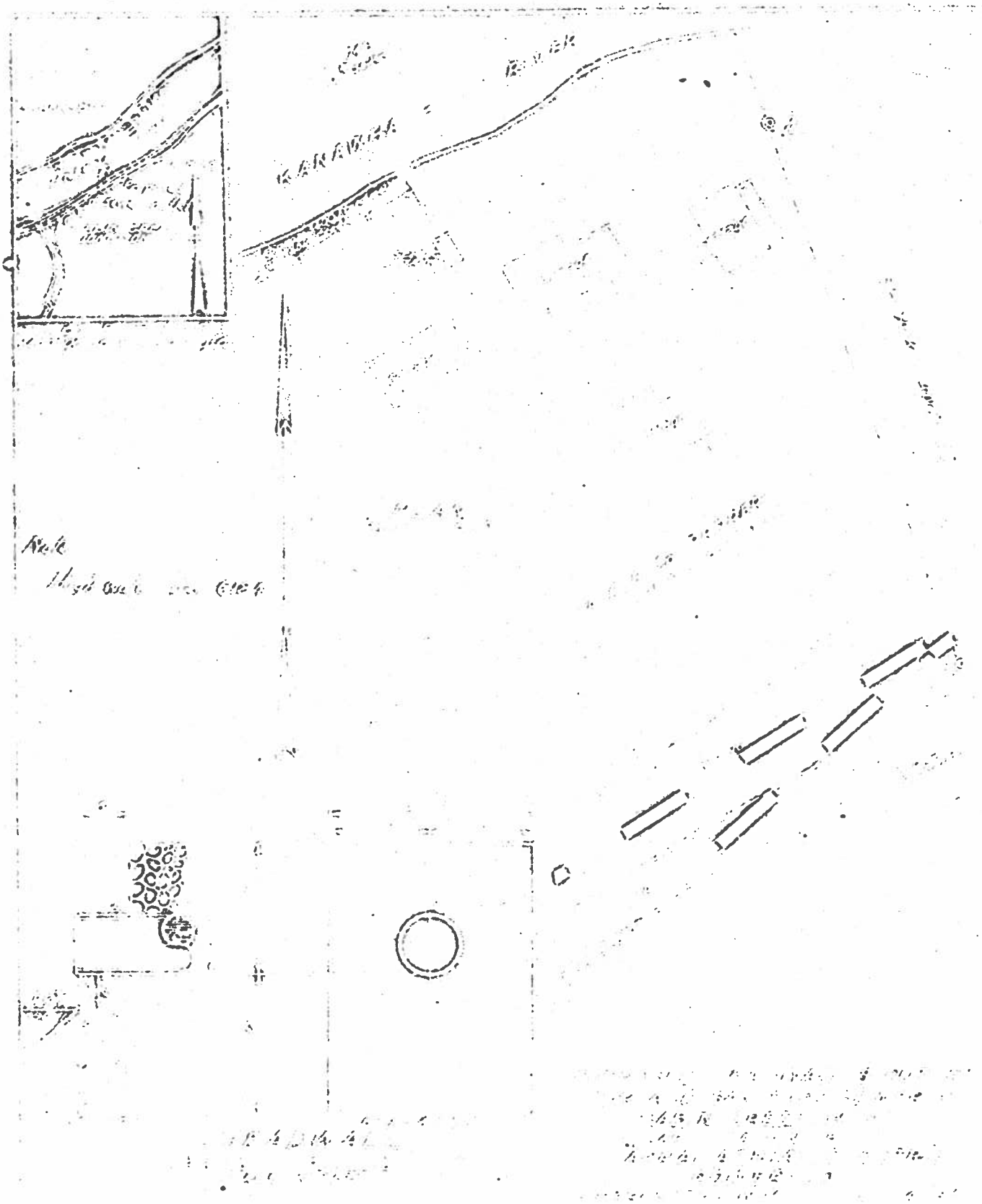
(j) That if the structure or work herein authorized is not completed on or before [illegible] (31st) day of December, 1951, this permit, if not previously revoked or specifically extended, shall cease and be null and void.

By authority of the Secretary of War:

Colonel, Corps of Engineers
Division Engineer

WAR DEPARTMENT
O. C. of E.
Form No. 96
Revised Jan. 17, 1940

16—12168 U. S. GOVERNMENT PRINTING OFFICE


KANAWHA
RIVER

AEP Confidential - Moderate Risk - Approved - 89-5148 - 01/20/1972 - acf_na_aep_89-5148_1.tif

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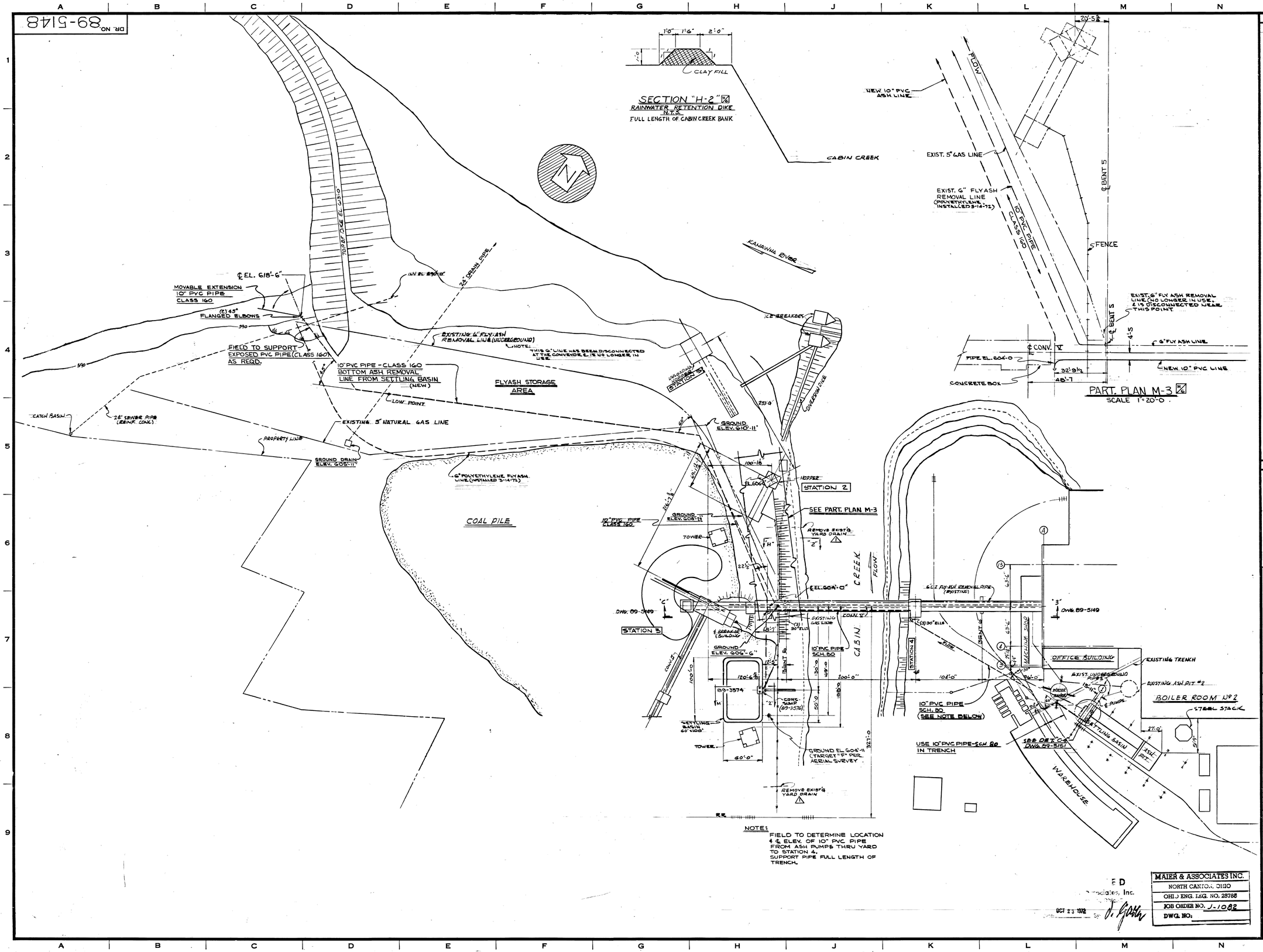

GENERAL NOTES

SEE DWG. Nº 89-5151

REFERENCE DWGS

SECTIONS & DETAILS	89-5149
SECTIONS & DETAILS	89-5150
SECTIONS & DETAILS	89-5151

DATE	NO.	DESCRIPTION	APPD.
1/20/72	1	REMOVED EXIST. DRAIN FROM COAL PILE TO CREEK ADDED SUMP SECT. "H-2"; REV. 10" BOTTOM ASH LINE / SETTLING BASIN.	

REVISIONS

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APPALACHIAN ELECTRIC POWER CO.
CABIN CREEK PLANT
CABIN CREEK, W. VA.

BOILER Nºs 81, 82, 91, & 92
ASH SETTLING BASIN
BOTTOM ASH REMOVAL PIPE

DR. NO. 89-5148-1

ARCH. | ELEC | MECH. | STR.

SCALE: 1"=50'-0"
DR. JGC & FA
CH
SQ. LDR
DATE 8-18-71

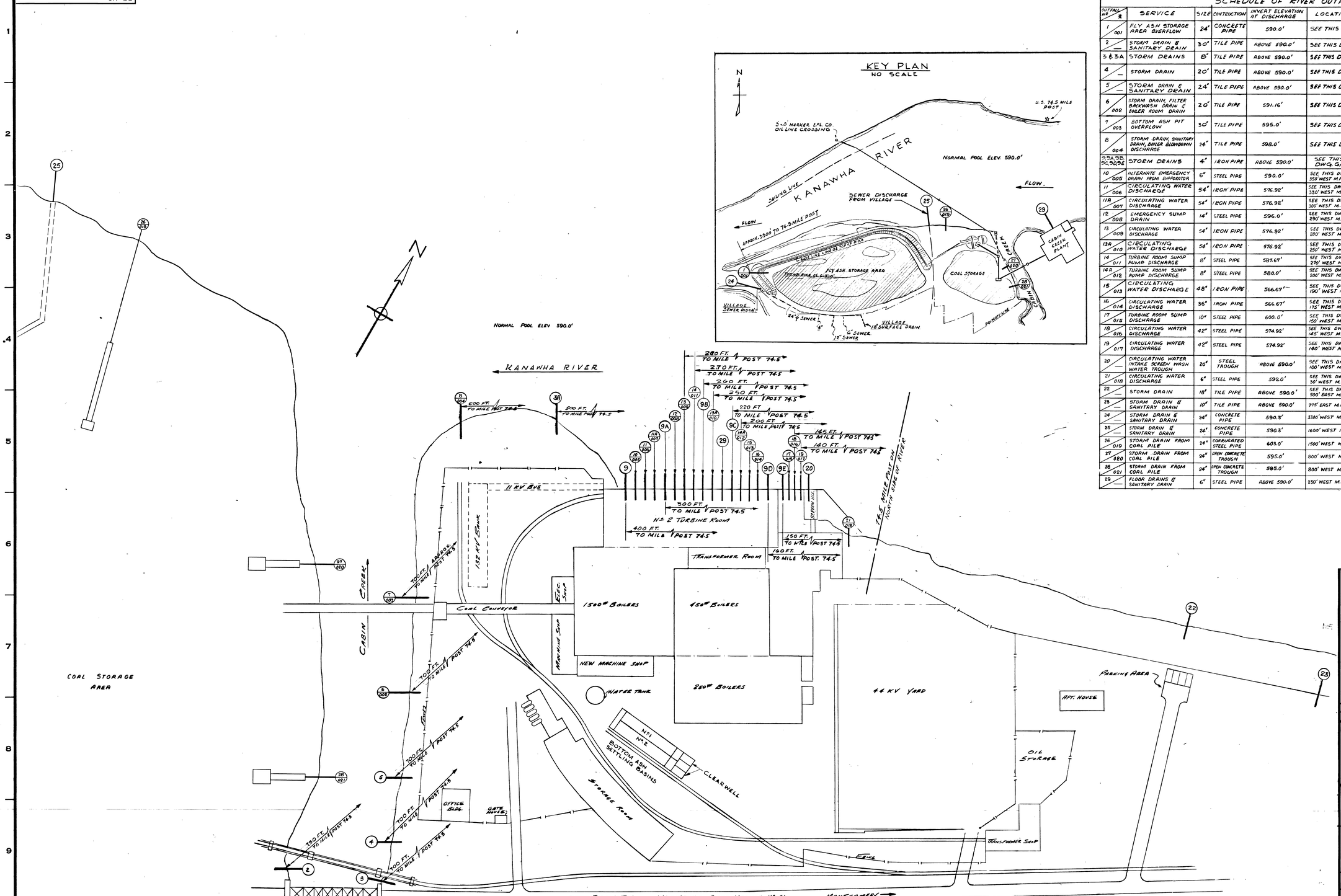
APPROVED BY: ENGINEERING DIV. CP Lugmm 1/20/72; DESIGN DIV.

AMERICAN ELECTRIC POWER SERVICE CORP.
P. O. BOX 487 CANTON, OHIO, 44701

MAIER & ASSOCIATES INC.
NORTH CANTON, OHIO
OHIO ENG. REG. NO. 23788
JOB ORDER NO. J-1082
DWG. NO.

ED
Associates, Inc.
OCT 23 1972

DR. NO. WR-60-5000



* ARMY CORPS OF ENGINEERS DISCHARGE SERIAL NUMBER

SCHEDULE OF RIVER OUTFALLS

OUTFALL NO. / *	SERVICE	SIZE	CONSTRUCTION	INVERT ELEVATION AT DISCHARGE	LOCATION	REMARKS
1 / 001	FLY ASH STORAGE AREA OVERFLOW	24"	CONCRETE PIPE	590.0'	SEE THIS DWG. H/3	INTERMITTENT FLOW — WEIR & SKIMMER FOR SOLIDS RETENTION AT OVERFLOW
2 / —	STORM DRAIN & SANITARY DRAIN	30"	TILE PIPE	ABOVE 590.0'	SEE THIS DWG. C/9	UNCONTAMINATED STORM WATER & TREATED SANITARY WASTES
3 & 3A	STORM DRAINS	8"	TILE PIPE	ABOVE 590.0'	SEE THIS DWG. D/9	UNCONTAMINATED STORM WATER (2 PIPES)
4 / —	STORM DRAIN	20"	TILE PIPE	ABOVE 590.0'	SEE THIS DWG. D/9	UNCONTAMINATED STORM WATER
5 / —	STORM DRAIN & SANITARY DRAIN	24"	TILE PIPE	ABOVE 590.0'	SEE THIS DWG. D/8	UNCONTAMINATED STORM WATER & TREATED SANITARY WASTES
6 / 002	STORM DRAIN, FILTER BACKWASH DRAIN & BOILER ROOM DRAIN	20"	TILE PIPE	591.16'	SEE THIS DWG. D/7	UNCONTAMINATED STORM WATER & INTERMITTENT DISCHARGE OF PROCESS WASTES FROM FILTERS
7 / 003	BOTTOM ASH PIT OVERFLOW	30"	TILE PIPE	595.0'	SEE THIS DWG. D/6	WEIR & SKIMMER FOR SOLIDS RETENTION AT OVERFLOW
8 / 004	STORM DRAIN, SANITARY DRAIN, BOILER BLOWDOWN DISCHARGE	24"	TILE PIPE	598.0'	SEE THIS DWG. F/5	UNCONTAMINATED STORM WATER, TREATED SANITARY WASTES, UNTREATED PROCESS WASTES
9A, 9B, 9C, 9D & 9E	STORM DRAINS	4"	IRON PIPE	ABOVE 590.0'	SEE THIS DWG. G/6, H/6	UNCONTAMINATED STORM WATER (6 PIPES)
10 / 005	ALTERNATE EMERGENCY DRAIN FROM EVAPORATOR	6"	STEEL PIPE	590.0'	SEE THIS DWG. G/5 350' WEST M.P. 74.5	EMERGENCY PROCESS DRAIN UNTREATED PROCESS WASTES
11 / 006	CIRCULATING WATER DISCHARGE	54"	IRON PIPE	576.92'	SEE THIS DWG. G/5 330' WEST M.P. 74.5	UNCONTAMINATED COOLING WATER CONDENSER #9
11A / 007	CIRCULATING WATER DISCHARGE	54"	IRON PIPE	576.92'	SEE THIS DWG. G/5 300' WEST M.P. 74.5	UNCONTAMINATED COOLING WATER, CONDENSER #9
12 / 008	EMERGENCY SUMP DRAIN	14"	STEEL PIPE	596.0'	SEE THIS DWG. G/5 290' WEST M.P. 74.5	UNCONTAMINATED TURBINE ROOM FLOOR DRAINS
13 / 009	CIRCULATING WATER DISCHARGE	54"	IRON PIPE	576.92'	SEE THIS DWG. G/5 280' WEST M.P. 74.5	UNCONTAMINATED COOLING WATER CONDENSER #8
13A / 010	CIRCULATING WATER DISCHARGE	54"	IRON PIPE	576.92'	SEE THIS DWG. G/5 250' WEST M.P. 74.5	UNCONTAMINATED COOLING WATER CONDENSER #8
14 / 011	TURBINE ROOM SUMP PUMP DISCHARGE	8"	STEEL PIPE	587.67'	SEE THIS DWG. G/5 270' WEST M.P. 74.5	INTERMITTENT DISCHARGE UNTREATED PROCESS WASTES
14A / 012	TURBINE ROOM SUMP PUMP DISCHARGE	8"	STEEL PIPE	580.0'	SEE THIS DWG. H/5 200' WEST M.P. 74.5	INTERMITTENT DISCHARGE UNTREATED PROCESS WASTES
15 / 013	CIRCULATING WATER DISCHARGE	48"	IRON PIPE	566.67'	SEE THIS DWG. H/5 190' WEST M.P. 74.5	UNCONTAMINATED COOLING WATER, CONDENSER #6 INACTIVE — COLD RESERVE
16 / 014	CIRCULATING WATER DISCHARGE	36"	IRON PIPE	566.67'	SEE THIS DWG. H/5 175' WEST M.P. 74.5	UNCONTAMINATED COOLING WATER CONDENSER #5, INACTIVE — COLD RESERVE
17 / 015	TURBINE ROOM SUMP DISCHARGE	10"	STEEL PIPE	600.0'	SEE THIS DWG. H/5 150' WEST M.P. 74.5	INACTIVE — NO FLOW #3 & 4 TURBINE DEACTIVATED
18 / 016	CIRCULATING WATER DISCHARGE	42"	STEEL PIPE	574.92'	SEE THIS DWG. H/5 145' WEST M.P. 74.5	UNCONTAMINATED COOLING WATER CONDENSER #3 & 4 INACTIVE COLD RESERVE
19 / 017	CIRCULATING WATER DISCHARGE	42"	STEEL PIPE	574.92'	SEE THIS DWG. H/5 140' WEST M.P. 74.5	UNCONTAMINATED COOLING WATER — CONDENSER #3 INACTIVE COLD RESERVE
20 / —	CIRCULATING WATER INTAKE SCREEN WASH WATER TROUGH	20"	STEEL TROUGH	ABOVE 590.0'	SEE THIS DWG. H/5 100' WEST M.P. 74.5	THE RETURN TO THE RIVER OF RIVER TRASH WASHED OFF THE INTAKE SCREENS
21 / 018	CIRCULATING WATER DISCHARGE	6"	STEEL PIPE	592.0'	SEE THIS DWG. J/6 30' WEST M.P. 74.5	UNCONTAMINATED COOLING WATER FROM TRANSFORMERS
22 / —	STORM DRAIN	18"	TILE PIPE	ABOVE 590.0'	SEE THIS DWG. M/7 500' EAST M.P. 74.5	UNCONTAMINATED STORM WATER
23 / —	STORM DRAIN & SANITARY DRAIN	10"	TILE PIPE	ABOVE 590.0'	775' EAST M.P. 74.5	UNCONTAMINATED STORM WATER & TREATED SANITARY WASTES
24 / —	STORM DRAIN & SANITARY DRAIN	24"	CONCRETE PIPE	590.3'	3300' WEST M.P. 74.5	UNCONTAMINATED STORM WATER & SANITARY WASTES FROM VILLAGE OF CABIN CREEK
25 / —	STORM DRAIN & SANITARY DRAIN	24"	CONCRETE PIPE	590.3'	1600' WEST M.P. 74.5	UNCONTAMINATED STORM WATER & SANITARY WASTES FROM VILLAGE OF CABIN CREEK
26 / 019	STORM DRAIN FROM COAL PILE	24"	CORRUGATED STEEL PIPE	603.0'	1500' WEST M.P. 74.5	STORM WATER RUNOFF FROM COAL STORAGE AREA
27 / 020	STORM DRAIN FROM COAL PILE	24"	OPEN CONCRETE TROUGH	595.0'	800' WEST M.P. 74.5	STORM WATER RUNOFF FROM COAL STORAGE AREA
28 / 021	STORM DRAIN FROM COAL PILE	24"	OPEN CONCRETE TROUGH	595.0'	800' WEST M.P. 74.5	STORM WATER RUNOFF FROM COAL STORAGE AREA
29 / —	FLOOR DRAINS & SANITARY DRAIN	6"	STEEL PIPE	ABOVE 590.0'	230' WEST M.P. 74.5	UNCONTAMINATED FLOOR DRAINS & SANITARY WASTES FROM VILLAGE OF CABIN CREEK

DATE	NO.	DESCRIPTION	APPD.
3/28/72	3	ADDED: ARMY CORPS OF ENGINEERS DISCH. SERIAL NO'S FOR OUTFALLS NO'S 6, 7, 10, 11, 11A, 12, 13, 13A, 14, 14A, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29; OUTFALLS NO'S 2, 3, 3A, 4, 5, 8, 9A, 9B, 9C, 9D, 9E, 11, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29; OUTFALLS NO'S 1, 2, 3, 3A, 4, 5, 6, 7, 8, 9A, 9B, 9C, 9D, 9E, 10, 11, 11A, 12, 13, 13A, 14, 14A, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29 TO VILLAGE SEWER DISCHARGES; NO. 29 N/A. REVISED: SCHEDULE OF RIVER OUTFALLS TABLE, N/12.	LWB
2/19/70	2	ADDED: SEWER DISCH. FROM VILLAGE, L/13. REVISED: SCHEDULE OF RIVER OUTFALLS TABLE, N/12. WR-5081/70	LWB
12/20/69	1	ADDED KEY PLAN & NOTE. WR-5041/69	LWB

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APPALACHIAN POWER CO.
CABIN CREEK PLANT
KANAWHA COUNTY
WEST VIRGINIA

PLANT LAYOUT OUTFALLS TO RIVER

DR. NO. WR-60-5000-3

ARCH.	ELEC.	MECH.	STR.

SCALE 1"=50'
DR. R.S. Hall
CH. N.S.
SQ. LDR. R.S.
DATE 4-1-69
APPROVED BY — ENGINEERING DIV. H.H. Williams — TAM
DESIGN DIV. — LWB

AMERICAN ELECTRIC POWER SERVICE CORP.
2 BROADWAY NEW YORK

14-1-C

AEP Confidential - Moderate Risk - Approved - WR-60-5000 - 03/28/1972 - act_na_aep_wr-60-5000_3.dwf

1933
Surface of Water Elevation 578.2
Normal Pool Elevation 577.9

...KING UPSTREAM ALONG LEFT BANK OF KANAWHA RIVER FROM
...AS INLAND AT THE MOUTH OF CABIN CREEK

APPENDIX B

WRD - Pile Inspection Report

CABIN CREEK
PILE INSPECTION REPORT

SUBMITTED TO:
NAVEEN KUMAR GANJI, PE
WSP USA Inc.
NAVEENKUMAR.GANJI@WSP.COM
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SUBMITTED BY:
CLAY HOGER, PE (TX), P.Eng., RPEQ
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June 22, 2026

WOOD RESEARCH AND DEVELOPMENT
10466 SUNNYSIDE ROAD SE
BOX 70, JEFFERSON, OR 97352
(503) 385-8379
WOODRANDD.COM

REPORT DOCUMENTATION PAGE

1. REPORT DATE	2. REPORT TYPE	3. REVISION
June 22, 2026	Pile Inspection Report	Rev. 2
4. PROJECT NUMBER	**5. CONTRACT NUMBER**	**6. WRD CONTACT PERSON**
9141S	**P156079US001**	Clay Hoger, PE (TX, OR, ID) hogerec@woodrandd.com

7. PROJECT NAME

Cabin Creek Pile Inspection Report

8. CLIENT NAME AND ADDRESS	9. CLIENT CONTACT PERSON(S)
WSP USA INC. 3348 Peachtree Rd NE, Suite 1100 Atlanta, Georgia 30326	Naveen Kumar Ganji, PE (GA, TN, TX, VA) naveenkumar.ganji@wsp.com 770-492-8105
10. TITLE AND SUBTITLE	**11. AUTHOR(S)**
Cabin Creek Pile Inspection Report	Clay Hoger, PE (TX, OR, ID)

12. EXECUTIVE SUMMARY

Wood Research and Development (WRD) was commissioned by WSP to perform an inspection and structural assessment of the timber piles at the AEP Cabin Creek Ash Pond, Former Cabin Creek Power Plant, West Virginia. This report summarizes the findings of the inspection and the key data required for WSP's geotechnical slope stability analysis.

Visual assessments found that all identifiable piles exhibit complete section loss by decay at and above water level. At ground level, the shoulders of the piles revealed the possible original diameters of an average of 9.36 inches, which indicates the piles were originally ordered for a 10 inch diameter.

Stress wave times across the piles were recorded to determine remaining specific gravity (and strength). Testing of this nature into heavily decayed wood simply confirmed the same. In combination with the partial visual identification of the original pile diameters, and the almost certainty that decay by fungal colony cannot occur underwater where oxygen does not exist, it has been concluded that the piles underground (underwater) remain in sound condition.

Ground level is likely to have changed over time, but the historical drawings indicate the piles are approximately 31-32 feet into ground. Using pulse echo technology, the pile depths were measured to be an average of 32.9 feet.

Requests were made for transverse shear design values for the purpose of geotechnical modelling of the embankment. Published shear values for wood are parallel to grain shear values. Transverse shear values are much higher and are typically not reached before some other mode of failure. Piles inground subject to lateral loads are subject to bending stresses, typically for the full length of the pile, meaning the shear stress parallel to grain will exist and therefore govern over transverse shear stress. Wood design values for strength properties such as shear are based on a statistical assessment of empirical test data where the lower fifth percentile value is used then factored downwards to include a safety factor and convert the value based on the duration of loading during testing compared to a normal duration (10 years) for design purposes. The NDS (2018) does not publish design values for White Oak piles. Mean shear parallel to grain values with standard deviation were adopted from widely cited sources and converted into an allowable design value of 405 psi. This value must then be adjusted for duration of load based on the assumptions made by the geotechnical model. If at rest prior to a landslide event, an allowable shear value of 730 psi can be used. If there is constant active pressure, an allowable shear value of 365 psi must be used. Factored shear values are also included if the geotechnical model follows Load and Resistance Design practices.

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1. INTRODUCTION

Wood Research and Development (WRD) was commissioned by WSP to perform an inspection and structural assessment of the timber piles at the Cabin Creek Ash Pond, Former Cabin Creek Power Plant, West Virginia, owned by American Electric Power Company, Inc. (AEP). The assessment was initiated as part of WSP's work to remediate the Cabin Creek Ash Pond and assess the stability of the adjacent embankment of the Kanawa River.

1.1 REFERENCED DOCUMENTS

The following documents were reviewed by this assessment:

- Potentiometric Map, Drawings 1 and 3 of 5, January 27, 2026, by WSP.
- Extracts of historic plan view and permit drawings of 1956, as supplied by WSP.
- Keu Plan of Boiler & Turbine Room Extension, #62909-3002-1, April 21, 1948, by American Gas & Electrical Service Corporation.

1.2 APPLICABLE STANDARDS

Any future structural analysis of the timber structure should be in accordance with standard practice and methodologies outlined in the following documents:

- National Design Specification (NDS) for Wood Construction (2018) and its Supplement.
- ASTM D2395 – Standard Test Methods for Density and Specific Gravity (Relative Density) of Wood and Wood-based Materials.
- ASTM D4442 – Standard Test Methods for Direct Moisture Content Measurement of Wood and Wood-Based Materials.

1.3 SCOPE OF WORK

WSP required an assessment of the pile depths, diameters, condition and shear strengths of the existing timber pile to inform their geotechnical calculations. To evaluate the piles, WRD

proposed a statistical approach to mirror that which is used to grade lumber based on prevailing naturally occurring defects (knots, slope of grain etc.). The inspection included the following tasks, as accessible:

- Photographs.
- Identification and location of visible damage.
- Measurements of the pile diameters.
- Collection of assay samples for laboratory analysis.
- Determination of pile depths using Pile Echo Technology (PET)
- Determination of pile condition by collecting stress wave times (SWT) perpendicular to grain.
- Development of transverse shear strengths.

Assay samples were analyzed by WRD's IAS accredited testing laboratory (TL-193) to establish the specific gravity (SG) and moisture content (MC) in accordance with ASTM D2395 and ASTM D4442, respectively, and to assess microscopic anatomy to identify the wood species.

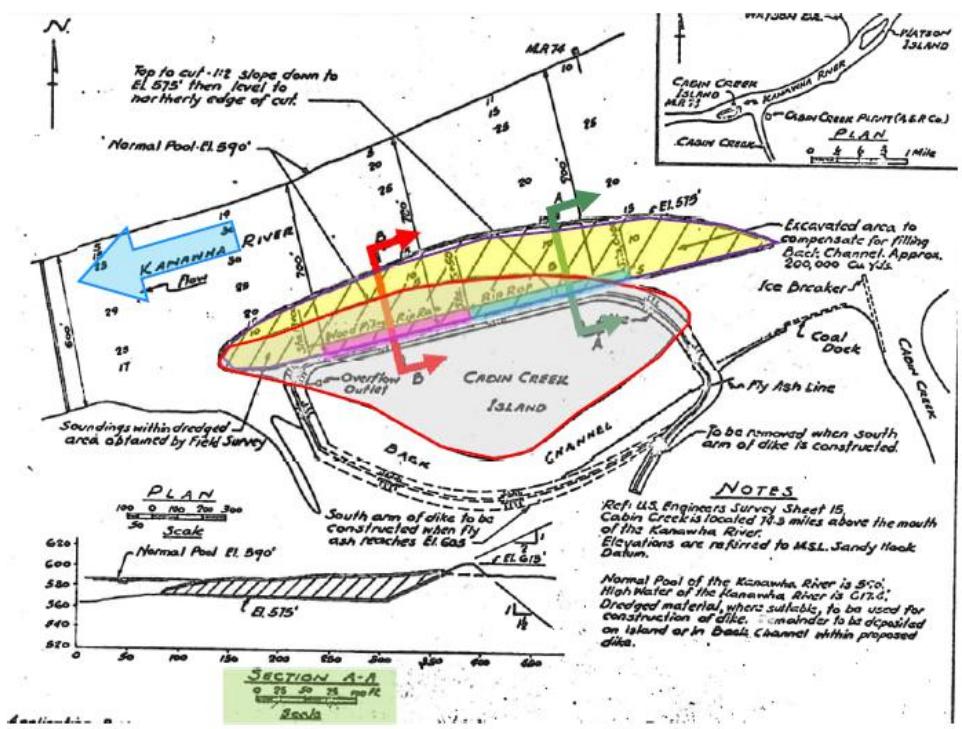


Figure 1-1: Plan view extract of historic drawings of 1956.

The plan view of the site presented in **Figure 1-1** was provided by WSP. Sections A-A and B-B are presented in **Figure 1-2**. The historic plans indicated that piles only existed in the zone designated by Section B-B. Prior to WRD's inspection, WSP representatives visited the site to verify that piles could not be observed in the zone designated by Section A-A or along the convergence of Cabin Creek with the Kanawa River.

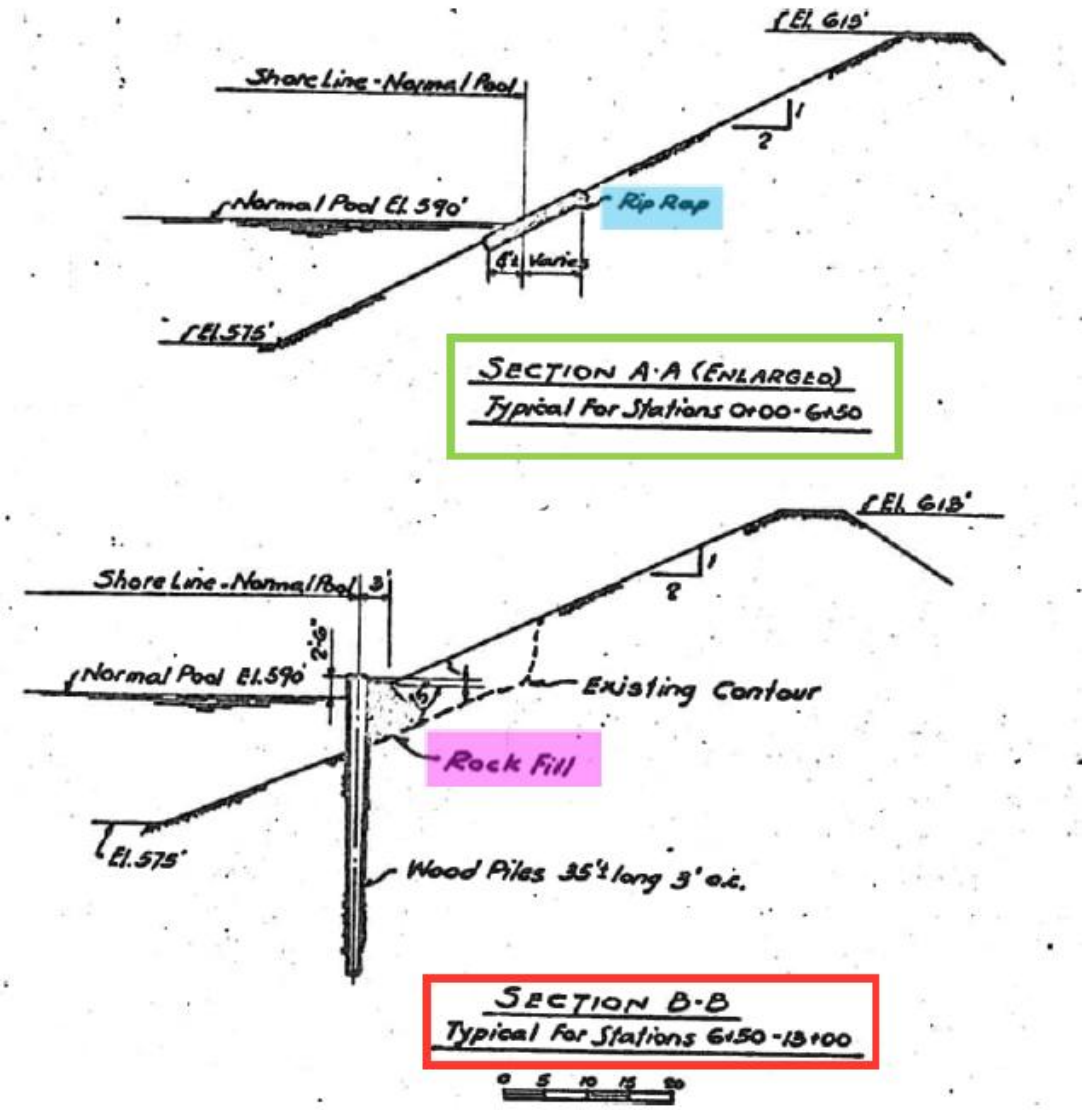


Figure 1-2: Section A-A and Section B-B extract of historic drawings of 1956.

1.4 LIMITATIONS

Figure 1-3 is a photograph of the piles, as supplied by WSP prior to WRD's inspection. As can be seen from the photograph, most piles have deteriorated significantly, and with extreme section loss. The piles are in water and the proximity of the piles to the embankment indicates shallow water depth, leaving little wood above ground to assess.

Figure 1-3: Photograph of the piles supplied prior to the WRD inspection.

The environmental requirements of AEP stipulated that no contaminants were permitted to enter the water. This meant that the piles could not be prepared for testing by cutting into the wood because of the potential for (creosote) preservative treatment to be released.

The PET testing typically requires petroleum jelly or lithium grease between the accelerometer (the receiver) and the pile to make a perfect couple so that the stress wave can be measured accurately. Neither could be used because of AEP's environmental requirements. The lack of grease did not preclude the testing but may have confounded some data. Since the impact of not using grease cannot be determined without speculation, no adjustments were made to the data.

Assessment of SWT measured perpendicular to grain gives meaningful information on the reductions of SG of the wood and therefore reductions in strength and stiffness properties can be quantified. In all cases, the piles were tested for SWT on pile segments where significant section loss had already occurred. The section loss extended to ground level in all cases. Turbidity was also a concern for AEP's environmental requirements, so digging into the river bed to identify original pile diameters and collect SWT below the splash zone could not be carried out.

1.5 METHODOLOGY

The inspection was performed using industry-recognized inspection practices for wood structural elements, incorporating visual examination, non-destructive testing, and laboratory analysis as appropriate for the structure and access conditions. Inspection procedures focused on identifying indicators of deterioration, including surface checking, fungal decay, insect activity, mechanical distress, moisture intrusion, and loss of section. Non-destructive testing tools, such as PET and SWT, are included in this report as standard practices for evaluating depth and internal fiber conditions.

Observations presented herein represent conditions identified during the inspection period and may not capture deterioration developing after the assessment. Evaluation and comments are based on field observations, professional judgement, industry standards, and established guidelines for timber inspection and preservation.

1.6 CONDITION STATE RATING ANALYSIS

The Condition State Rating (CSR) system in **Table 1-1** has been developed by WRD, through timber inspection experience, to clearly describe the condition of the elements inspected.

The CSR system assigns qualitative ratings, on a scale of 1 to 5, to timber members based on visual indicators of deterioration, biological activity, moisture exposure, test data, and anticipated progression of decay mechanisms. Ratings consider factors such as material loss, moisture exposure, biological activity, mechanical distress, and remaining effective cross-section. Using these observations, along with professional judgment and established timber assessment guidance, an estimated remaining service life is projected to reflect the expected period during which the member can continue to function as intended under typical service conditions. Remaining life estimates are inherently approximate and assume continued normal exposure and maintenance. Accelerated deterioration may occur if moisture, alterations, or environmental conditions change.

Table 1-1: Condition State Rating Descriptions

Condition State Rating	Subjective Rating	Estimated Remaining Life Span		Description
1	Good	100%	80 Years	Elements are in like-new condition and free of defects.
2	Fair	80%	64 Years	Free of defects affecting structural performance, integrity, and durability. Deterioration of a minor nature in the protective coating and/or parent material is evident.
3	Poor	30%	24 Years	Defects affecting durability/serviceability which may require monitoring and/or remedial action or inspection by a structural engineer. Component or element shows marked and advancing deterioration including loss of protective coating and minor loss of section from the parent material is evident. Intervention is normally required.
4	Very Poor	5%	Up to 4 Years	Defects affecting the performance and structural integrity of the structure which require urgent action as determined by a detailed structural engineering inspection. A component or element shows advanced deterioration, loss of section from the parent material, signs of overstressing or evidence that it is acting differently to its intended design mode or function.
5	Unsafe	1%	Less Than 2 Years	Structure should be closed. Structural integrity is severely compromised, and the structure must be taken out of service until a structural engineer has inspected the structure and recommended the required remedial action.

2. BACKGROUND INTO WOOD SCIENCE

2.1 WOOD AS A CONSTRUCTION MATERIAL

As a building material, wood has some unique characteristics. An understanding of these properties is important when inspecting existing timber structures and determining remaining strength for structural and other assessments.

2.1.1 THE VARIABILITY OF WOOD

As an organic material, wood is subject to a high level of variability, dependent on species and growth conditions. All physical properties of wood can vary, including strength, stiffness, weight, and dimensions. Additionally, wood is anisotropic, meaning the mechanical properties vary in the different directions as the tree grows (longitudinal, radial, and tangential). Naturally occurring defects, such as knots, can also affect mechanical properties. In contrast to concrete and steel, the effect of temperature on wood is negligible but it is highly affected by moisture and humidity which causes dimensional changes and can lead to biological degradation over time since decay causing fungal colonies require higher moisture content (greater than 24% by weight) to continue.

2.1.2 MOISTURE CONTENT AND HYGROSCOPICITY

Wood is hygroscopic, meaning that it can absorb and expel moisture from and to its surroundings. Changes in MC can affect other physical properties, such as dimensions, strength, and susceptibility to decay. Wood tends toward an equilibrium moisture content (EMC) which depends on the surrounding exposure conditions (temperature and humidity) and is relatively independent of species. Every timber structure undergoes daily and seasonal fluctuations which means the EMC can only be estimated.

The EMC of a timber element varies throughout the structure. Members which are sheltered from rain, exposed to sun, and exposed to airflow will tend to have a lower EMC. Elements which are directly exposed to water from rain, condensation, or leaking utilities may have a significantly higher EMC. An EMC above about 22% begins to support fungal decay, and

decay is most severe when MC is above the fiber saturation point (approximately 30%, see **Section 2.1.3** below). Elements which are fully saturated, for example the buried and permanently submerged pile segments, are deprived of oxygen (typically less than 21%) and will not decay.

2.1.3 MOISTURE INDUCED DIMENSIONAL CHANGE

Moisture in wood exists in two forms; the first is free water that is contained within the hollow cell cavities (lumina). Free water may be liquid, vapor, or ice, depending on the temperature. The second is bound water or water that is chemically linked with the cell walls through hydrogen bonding.

At low MC, most moisture is stored as bound water; as additional moisture is absorbed, it quickly bonds with the cellulose and hemicellulose which make up the cell walls; this is known as bound water. At higher MC, the cell walls become fully saturated, and additional moisture remains in the lumina as free water. The point at which the cell walls are saturated, but no free water exists, is known as the fiber saturation point (FSP). FSP commonly occurs between 26 and 34%.

Below FSP, reducing MC results in shrinkage of the wood while increasing MC results in swelling. This is a key consideration when detailing timber structures as it is necessary to allow the wood to shrink and swell as the MC fluctuates. Above FSP, fluctuations in MC do not result in dimensional changes.

The strength and stiffness of wood tend to increase as the MC reduces to below FSP; these changes are relatively small when considering the range of MC fluctuations for typical structural members, but they may need to be considered when wet-service conditions are anticipated. At elevated MC (above 24%), decay can occur; this can drastically reduce mechanical properties. More information on decay is presented in **Section 2.2**, below.

2.2 WOOD DETERIORATION

Wood deteriorates for a variety of reasons, adversely affecting the wood properties. The causes of deterioration can be classified as either biotic (living) agents or physical (nonliving) agents. The physical agents include chemical agents (e.g., ferric degradation), ultraviolet light,

erosion, abrasion, and impact damage. Biotic agents include fungi, bacteria, insects, marine borers, and birds. Often, these agents can compound each other, with one form of deterioration creating conditions that allow another form of deterioration to flourish. For example, decay of wood can lead to more insect infestation which can promote potential for woodpeckers, these events are all part of nature's cycle of life. Decay typically allows a primary breakdown of the wood minute structure, which in turn allows insects more freedom to invade the timber elements and decayed wood is easier to chew. This process is the necessary typical life cycle in the forest but does not work well in a timber structure.

2.2.1 WOOD DETERIORATION DUE TO BIOTIC AGENTS

Biotic organisms that attack wood include bacteria, fungi, insects (such as termites), and marine borers. As living organisms, they require certain conditions for survival – such as moisture, oxygen, specific temperature ranges, and food, which is usually wood. When the basic necessary living conditions are available, biotic agents of wood deterioration are free to proliferate, but if any one of them is removed the wood is safe from further biotic attack. Note that fungal decay requires an MC above ~24%; however, other biotic organisms like termites do not require elevated MC.

2.2.2 FUNGAL DECAY

As described above, fungal colonies require certain conditions for survival, including appropriate levels of moisture, oxygen, and temperature, as well as a source of food (the wood itself). If any one of those conditions is removed, fungal decay is stopped.

Fungal decay is the most common form of wood deterioration and 82% of all timber bridge degradation is due to fungal decay. When exposed to favorable levels of moisture, oxygen, and temperature, most types of wood become an attractive food source for a variety of decay-producing fungi. The precise conditions vary according to the species of wood and the species of fungi, but some general rules apply.

Fungi require moderate temperatures. Growth progresses most rapidly between 50°F and 95°F. Outside of this range, growth significantly slows; below 35°F or above 100°F, growth will

cease. Moisture content (measured as a percentage of the dry weight of the wood) typically needs to be above 22% for fungal growth. Moisture content control is the best means of extending life for timber bridge structures. Lastly, there must be sufficient oxygen; if the wood is fully saturated with water, the oxygen supply (>21%) will be cut off, and fungal growth will stop. It is for this reason that the assessment of the pile conditions below the splash zone and in ground where oxygen does not exist, the piles can be deemed to be free of decay zones caused by fungal colonies. Conversely, where the piles are exposed at the splash zone and above the water level, the available oxygen has allowed the fungal colony to proliferate, consuming much of the pile segments in these zones.

If any of the conditions noted above are not met, fungal decay will cease. However, it is important to note that this typically does not kill the fungus. The fungus goes dormant but remains in the wood. As soon as the favorable conditions return, fungal growth and decay will resume.

Decay fungi may be generally classified into two categories by the appearance on the wood surface:

1. **Brown Rot |** Appears darker and can crack across the grain. Brown rot fungi attack the cellulose in the wood fibers. The brown color is due to the remaining lignin (the binder which holds the cellulose structure together), which is not consumed by the fungi. The wood tends to shrink, collapse, and crush easily. It may crack across the grain forming small chunks or cubes, which is a sign of advanced decay.
2. **White Rot |** Appears lighter in color and does not crack across the grain until severely degraded. In contrast to brown rot, white rot consumes both lignin and cellulose and leaves the surface appearing generally intact, but with little or no significant mechanical strength. The surface of the decayed wood tends to have a "white" appearance. Wood does not tend to crack, shrink, or collapse. It often feels soft or spongy.

Dry rot is a common type of decay fungi in which the wood becomes brown and crumbly (frass left over after the fungal colony has moved on) and has an apparent dry condition. However, dry rot is a misnomer, because the wood must have some moisture in it to decay, although it may become dry later. A few fungi have water-conducting strands (hyphae) which can carry water, usually from the soil or water sources near the wood, into the areas where the wood elements are located. These elements are moistened and subsequently wood rots that would otherwise be dry and not decay.

Interior decay damage can occur even when some precaution has been taken. Surface treated wood material can form cracks, which extend beyond the treated surface into untreated core material. Water can also get into the core of "protected" wood by the fungi hyphae. In either case water enters the core material and provides the adequate conditions for decay fungi to live. It is known that as little as a 10% loss in SG due to decay can lead to a loss of 75% in bending strength and 80% in compression perpendicular to grain strength.

2.2.3 EFFECTS OF DETERIORATION ON THE PROPERTIES OF WOOD

As decay-causing fungi attack wood, cellulose and/or lignin are removed from the material. This causes a reduction in SG and breaks down the integrity of the cellular structure. The reduced SG has a direct correlation to reductions in mechanical properties.

The primary effects of fungi attack on wood can be characterized by the following points[1]:

- Change of color.
- Change of odor.
- Decreased weight.
- Decreased strength.
- Decreased stiffness.
- Increased hygroscopicity.
- Increased combustibility.
- Increased susceptibility to insect attacks.

Fungal decay negatively impacts nearly all mechanical properties of wood. Bending strength is typically more affected than shear strength. Strength and stiffness may be significantly reduced even in the early stages of decay. Mechanical properties may be reduced by 10% before decay is detectable. By the time SG is reduced by 10%, strength losses may be up to 80%. At this stage, decay is usually not visually detectable without magnification. By the time decay is visible to the naked eye, significant damage has already taken place.

There are no accepted methods for estimating the degree of strength-reduction based on visual assessment of decayed wood. If decay is visible to the naked eye, it is safest to assume that that portion of wood has no structural capacity.

A variety of non-destructive test (NDT) methods, including SWT and resistance micro-drilling, may be utilized to determine the extent of incipient decay and thereby estimate its remaining strength. These methods typically measure one property of the wood, such as stiffness or hardness, which can be correlated to the decay-related reduction in SG. The reduced SG can then be correlated to reductions in other mechanical properties. These methods for testing and analysis are often proprietary and developed through extensive empirical testing.

2.3 NON-DESTRUCTIVE TEST EQUIPMENT

2.3.1 STRESS WAVE TIMES

The first NDT method utilized in the inspection was the Electronic Pulse Highlight and Outline Diagnostic (EPHOD®) compression SWT test system. This analysis consists of sending a compression stress wave through a wood member and measuring the wave's time-of-flight. Because sound dense wood transmits energy more rapidly than deteriorated or decayed wood, increased wave travel times may indicate internal decay or other structural discontinuities.[5]

The EPHOD® SWT device consists of a transducer pair connected to a precise timer. Each half of the transducer pair detects very fine impulses. A compression wave is introduced into the wood material by striking it with a hammer embedded with one half of the transducer pair, which is then detected by the second half of the transducer pair. The technician measures the distance between the transducer pair, and the time-of-flight is recorded by the EPHOD® device in millionths of a second (microseconds). The average wave propagation velocity is then calculated based on the modulus of elasticity (MOE) parallel or perpendicular to grain, the species and specific gravity (SG) of the wood member, and the strength of the material. Other adjustment factors may also be applied to the base SWT reading, which depend on temperature, moisture content, the presence of preservative treatment, and other properties. For example, frozen wood transmits energy more quickly, so the SWT times must be adjusted appropriately. The SWT results are normalized for a 12-inch gauge length.

When the adjusted time-of-flight is below 150 to 250 microseconds (µs) for softwoods, or 250 to 350-µs for hardwoods, the element is in sound condition with little to no loss of strength. SWT ranging between 300 and 700-µs are not of concern for structural integrity at the time of testing – but it can indicate the onset of decay. Where SWT range from 700 to 1000-µs (shown in

yellow on Figure 2-1), the element can carry its own dead weight and an unknown live load at the localized area where the reading is taken. When the SWT values exceed 1000-µs (shown in red on **Figure 2-2**) the element is unable to carry its own dead weight at that localized area. SWT readings greater than 2200-µs are indicative of cavities within the element. When SWT values exceed 3300-µs, the element can no longer support its own dead weight. Elements with SWT values exceeding 3300-µs often have other criteria impacting performance, such as fasteners or adjacent members holding the element in place, even though the member is likely severely decayed throughout.

Since internal damage is rarely distributed uniformly throughout the cross section of a timber member, multiple readings are taken in various directions at a single test location. The orientation of each reading is reported using the points on a clock face. For example, 3-9 represents a horizonal reading through the 3 o'clock and the 9 o'clock points. 12-6 represents a vertical reading through the 12 o'clock and the 6 o'clock points. This is illustrated in **Figure 2-2**.

SWT testing is recognized by the USDA Forest Products Laboratory, *Wood and Timber Condition Assessment Manual*, as a reliable tool for screening timber for internal defects.[5]

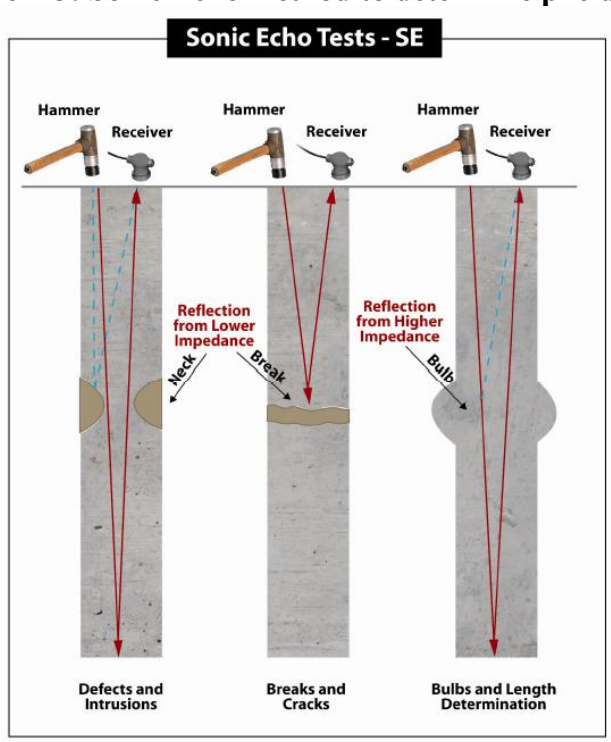
Figure 2-2: Stress wave time results key and configuration.

2.3.2 PILE ECHO TECHNOLOGY

The second non-destructive test method utilized in the inspection was the Foundation Test Gauge (FTG-1) PET test system. The FTG-1 device adopts the sonic echo (SE) method which is a low strain integrity test conducted from the top (or upper side) of the pile, as shown in **Figure 2-3**. The SE method is a low strain integrity test conducted from the top (or upper side) of the shaft as shown in the figure below. Test equipment includes a 3-pound hammer (instrumented or mechanical), receiver (accelerometer) mounted on the top (or upper side) of the pile, and a data

acquisition platform (tablet). The test involves hitting the pile top with a 3-lb hammer to generate wave energy that travels to the bottom of the foundation. The wave reflects off irregularities (cracks, necks, bulbs, soil intrusions, voids, etc.) and/or the bottom of the foundation and travels back along the foundation to the top. The receiver measures the vibration response of the foundation to each impact. The receiver does not track the wave as it travels down the pile; rather, it records the local motion at the pile head caused by wave interactions, including the initial impact and subsequent reflections from impedance changes along the pile. The data collection platform acquires, processes, and displays the receiver's output. Pile length and integrity are evaluated by identifying and analyzing the arrival times, direction, and amplitude of reflections measured by the receivers in time. Analysis of the length determination and the integrity evaluation of a foundation with the SE method is based on the identification and evaluation of reflections. Test results are analyzed in the time domain for the SE test.[6]

Figure 2-3: Sonic Echo method to determine pile depth.



The SE test method is sensitive to changes in the shaft impedance (shaft area, velocity, mass density), which causes the reflections of the compression wave energy. Compression wave energy (hammer impact energy) reflects differently from increased shaft impedance than from

decreased shaft impedance. This phenomenon allows the type of reflector to be identified. Soil intrusions, checks, breaks, cracks, cold joints, poor quality material, and similar defects are identified as reflections that correspond to a decrease in the shaft impedance. Increases in the shaft cross-section or the competency of surrounding materials such as bedrock and stiffer soil strata are identified as reflections corresponding to increases in the shaft impedance. A decrease in impedance is indicated by a downward initial break of a reflection event in an SE record and frequency peaks positioned in a record such that a peak could be extrapolated to be near 0 Hz in the mobility plot. Conversely, an increase in shaft impedance is identified by an upward initial break for an SE reflector. When length to diameter ratio exceeds 20:1 to 30:1 for piles in stiffer soils/bedrock, the attenuation of compression wave energy is high and bottom echoes are weak or unidentifiable in SE test results. If the pile is embedded in a material which has similar impedance to that of the pile, it may not be possible to see a reflection from the bottom of the shaft.

SE data is analyzed by picking the troughs in the wave form. The first trough representing the initial impact should be selected, along with the subsequent trough deemed to represent the shaft bottom (or other significant reflection). Based on these picks, the shaft depth will be automatically calculated and displayed at the top of the waveform panel as shown in **Figure 2-4** below. If multiple picks (three or more) are made at locations where multiple echoes appear in the waveform, the depth value will be averaged.

The determination of the pile lengths from the recorded waveforms depends on the assumed stress wave velocity within the timber, which is not directly measured by the equipment and may vary based on material properties, moisture content, and in-service condition. Pile depths are evaluated using two fundamental relationships: the wave speed derived from the material properties, $V = \sqrt{E/\rho}$, and the time-of-flight method to determine depth, $D = Vt/2$.

Based on the identified species of White Oak (see Section 3.5), which has a green MOE value of 1,250 ksi[8] and a specific gravity of 0.60, the moisture content of the wood at fiber saturation point (32% MC), the expected wave speed to be adopted is 11,000 feet per second.

Figure 2-4: Identifying shaft depth using the PET software (example only).

Finally, wave behavior and signal interpretation depend on the selection of frequency filter ranges. The applied filter setting represents a band-pass frequency range used to process the recorded PET signal data, where the lower cutoff (F1) and upper cutoff (F2) define the range of frequencies retained for interpretation. In this context, the filter removes low-frequency components that may be associated with signal drifting, background noise, or energy unrelated to the pile response, as well as high-frequency components that are often dominated by impact noise, surface wave effects, and localized irregularities at the pile head. Higher-frequency content is more sensitive to near-surface conditions, pile head variability, and intermediate impedance changes. Lower-frequency content more strongly reflects the overall pile response. The resulting filtered signal therefore emphasizes the frequency content most representative of axial wave propagation within the pile.

2.3.2.1 HARMONIC BEHAVIOR

Interpretation of the PET data also requires consideration of harmonic behavior, particularly for long, slender piles where the fundamental response may be weak or absent. This harmonic behavior can produce early-time oscillations and apparent reflection features that do not

correspond to the true pile toe. While the fundamental frequency is associated with the full pile length, higher order harmonics correspond to shorter wavelengths that are more sensitive to near-surface conditions and intermediate impedance changes. Under conditions of high attenuation and strong soil-pile coupling, the fundamental frequency may be dampened, resulting in a response dominated by higher order harmonics.

In wood, stress-wave frequency is linked to both modulus of elasticity and density through the wave-speed relationship $V = \sqrt{E/\rho}$. Additionally, wave propagation velocity is governed by the basic wave equation, $V = f\,\lambda$, where f is frequency and λ is wavelength. This equation is used to determine the overall pile depth through the time-of-flight relationship, $D = Vt/2$. Furthermore, where the fundamental frequency is calculated as, $f = V / (2D)$, higher order harmonic behavior, which occurs at n-integers, is calculated as $f_n = n\,V / (2D)$. When combined with the lab measured range of densities and moisture contents, the fundamental frequency and harmonic frequencies vary, as shown in **Table 2-1**.

Table 2-1: Typical PET frequency ranges at 40ft depth with a wave speed of 11,000 fps.

Frequencies	Frequency (Hz)
1st Harmonic	138
2nd Harmonic	275
3rd Harmonic	413
4th Harmonic	550
5th Harmonic	688
6th Harmonic	825
7th Harmonic	963
8th Harmonic	1100
9th Harmonic	1238
10th Harmonic	1375

Interpretation of the results was therefore carried out with careful consideration of the presence of higher-order harmonic frequencies inherent to PET testing due to the presence of very deep piles. Care was taken to avoid misidentification of second- and third-order harmonic

responses, which can produce apparent reflections corresponding to fractional pile lengths (e.g., one-half or one-third of the true depth). Accordingly, early-time oscillations and higher-frequency features were not interpreted as pile toe reflections, and emphasis was placed on lower-frequency behavior more representative of the fundamental response.

2.3.2.2 DISCLAIMER

It is noted that PET interpretation is an inherently iterative and detail-oriented process, requiring evaluation of multiple filter ranges and waveform characteristics for each individual record. While the analysis presented herein reflects a comprehensive review consistent with the project scope and schedule, the volume of data and complexity of the response necessitate a degree of professional judgment in prioritizing interpretation efforts. Accordingly, the findings presented herein represent the current best interpretation based on available information, and additional review or refinement of individual records may result in further clarification or adjustment of specific observations. The right is therefore reserved to supplement or amend these interpretations should additional analyses be undertaken.

3. INSPECTION FINDINGS

3.1 SITE CONDITIONS

The inspection focused on the piles observed in the zone represented by Section B-B located in **Figure 1-1**. Access to this zone was via two staircases. A densely vegetated area divided Section B-B into two subzones B1 and B2, located in the west and east ends of Section B-B, respectively. It was difficult to locate all piles, and it is possible that some have experienced sufficient section loss to now be buried.

3.2 PILE CONDITIONS

A total of 58 piles could be identified in Section B-B, but this does not preclude the existence of other piles in this zone. Note that measurements and testing took place on a total of 47 piles. All piles were found to exhibit advanced deterioration above water level, and all piles have significant section loss in the splash zone. Classification of condition state of the pile segments at or above the splash zone is consistent, in that they would all be deemed "unsafe" with a CSR of 5. All pile segments at or above the splash zone show sign of complete section loss and at best, advanced deterioration by fungal decay.

All but Pile 6 of subzone B2 have generally experienced decay commencing in the sapwood, with only portions of the heartwood remaining. Photographs of the piles are presented in **Appendix B**. **Figure 3-1** shows a photograph of Pile 18 of Subzone B2 which shows only the heartwood remaining above water and which is exhibiting advanced decay. The moisture content of the pile is sufficiently high enough to promote the growth of vegetation on the remaining pile segment. Pile 6 is the anomaly, with only the sapwood partially remaining above water, see **Figure 3-2**.

Some piles could no longer be seen above water. It is understood from historical documents that the piles were installed with a spacing of 3-feet on center. If this were so, some piles no longer exist above ground level.

Generally, all piles that could be identified above ground revealed their possible original diameters at ground level, where the shoulders of the remaining sapwood can be seen, as seen for Pile 16 of Subzone B1, shown in **Figure 3-3**.

Despite the significant section loss, SWT and PET testing were conducted where practicable.

Figure 3-1: Typical pile conditions with only decayed heartwood remaining above water.

Figure 3-2: Plan view photograph of Pile 6 of Subzone B2 with only the sapwood remaining above water.

Figure 3-3: Plan view photograph of Pile 16 of Subzone B2 showing the shoulder of the pile at ground level.

3.3 STRESS WAVE TIME RESULTS

The results of the SWT tests are summarized in **Table 3-1** and **Table 3-2** for Subzone B1 and B2, respectively. The raw SWT values are adjusted based on the measured gauge length, which is measured as the distance between the strike and the accelerometer. In many cases, the decay resulted in areas of piles separated by large cracks or by section loss, such as Pile 9 of Subzone B1, shown in **Figure 3-4**. It is not possible to test for SWT across discontinuities, and so SWT testing took place across the isolated segments where possible.

The adjusted values are colored as described in **Section 2.3.1**. Generally, all piles exhibit some level of decay, which is obvious by visual inspection of the piles. The testing for SWT values took place at or just below water level, which is still considered the splash zone, in pile material where section loss has already occurred. It was not possible to test below ground level where the potential full diameters of the piles exist due to concerns for raised turbidity.

Figure 3-4: Pile 9 of Subzone B1 with large vertical separation between remaining segments (left and right sides)

Table 3-1: Stress wave times for piles in Subzone B1.

Zone	Pile	SWT Gauge Length (in)	Raw SWT (µs)	Adjusted SWT (µs)	Notes
B1	1				Too punk to test
B1	2	8	891	1403	
B1	3				Major section loss, no reading
B1	4	4.5	355	984	
B1	5	4	251	786	
B1	6	1	76	920	
B1	7	7.75	734	1201	
B1	8	7	465	855	
B1	9	7	465	855	
B1	10	5	276	704	
B1	11				Too much section loss to test
B1	12	6	525	1100	
B1	22	6	653	1356	

Table 3-2: Stress wave times for piles in Subzone B2.

Zone	Pile	SWT Gauge Length (in)	Raw SWT (µs)	Adjusted SWT (µs)	Notes
B2	1	2	189	1151	
B2	2	4	394	1215	
B2	3	4	235	738	
B2	5	7	874	1557	
B2	6				Cannot test, annulus only
B2	7	6	367	784	
B2	8	6	780	1610	
B2	9	4	278	867	
B2	10	6	719	1488	
B2	11	6	484	1018	
B2	12	8	782	1240	

B2	13	6	278	606	
B2	14	6	650	1350	
B2	15	5	320	810	
B2	16	7	254	494	
B2	17	7	394	734	
B2	18	7	492	902	
B2	19				Cannot test, too punky
B2	20	4	297	924	
B2	21	11.75	838	954	
B2	22	8	618	994	
B2	23				Cannot test, too punky
B2	24	7	970	1721	
B2	25	8	522	850	
B2	26	7	465	855	
B2	27	7	659	1188	
B2	28	5	604	1491	
B2	33	4	417	1284	
B2	36	6	331	712	

3.4 PILE ECHO TECHNOLOGY TEST RESULTS

With careful consideration of the calculated wave propagation characteristics, intermediate impedance changes, dynamic inflection behavior, expected fundamental frequency ranges, influence of higher-order harmonic behavior, the depth results of the PET tests were determined and summarized in **Table 3-3** and **Table 3-4** for Subzones B1 and B2, respectively. Full PET records are presented in **Appendix** C. On average, the accelerometer was positioned at or just below level, for an average of 1 foot above ground level. The pile depths in **Table 3-3** and **Table 3-4** deduct this distance from the PET reading to obtain the pile depths in ground.

Figure 1-2 indicates a design length of the piles to be 35 feet with the pile butts to be located 2.5 feet above normal water. While no water level was measured, the water level at the time of the inspection, between wet and dry seasons, could be considered 'normal'. Ground level was approximately 1 foot below water level. Ground level is likely to have changed over time, but the historical drawings indicate the piles are approximately 31-32 feet into ground. The

average inground depth of all piles is 32.9 feet as measured by the PET (standard deviation of 3.32 feet and coefficient of variation of 10.1%), which matches well with the historical design depth.

Table 3-3: PET test data for Subzone B1.

Subzone	Pile	PET Reading (ft)	Pile Depth Inground (ft)
B1	1	36.8	35.8
B1	2	40.4	39.4
B1	3	37.9	36.9
B1	5	27.2	27.2
B1	6	35.7	34.7
B1	7	35.2	34.2
B1	8	38.5	37.5
B1	9	29.6	28.6
B1	10	32.1	31.1

Table 3-4: PET test data for Subzone B2.

Subzone	Pile	PET Reading (ft)	Pile Depth Inground (ft)
B2	4	35.3	34.3
B2	5	29.4	28.4
B2	6	28.9	27.9
B2	7	32.0	31.0
B2	8	32.3	31.3
B2	9	39.0	38.0
B2	10	31.4	30.4
B2	11	33.0	32.0
B2	12	30.7	29.7
B2	13	29.3	28.3
B2	14	33.6	32.6
B2	15	34.7	33.7
B2	16	33.9	32.9
B2	18	33.1	32.1
B2	20	35.6	34.6
B2	21	36.0	35.0
B2	22	36.3	35.3
B2	23	36.5	35.5
B2	24	28.5	27.5
B2	25	34.4	33.4
B2	26	31.7	30.7
B2	27	40.6	39.6
B2	28	31.1	30.1
B2	31	31.9	30.9
B2	33	33.6	32.6
B2	34	35.5	34.5
B2	36	37.9	36.9

3.5 PILE DIAMETERS

As noted in Section 3.1, all piles that could be identified above ground generally revealed their possible original diameters at ground level, where the shoulders of the remaining sapwood can be seen, as seen for Pile 16 of Subzone B1, shown in **Figure 3-3**. Measuring the shoulders was difficult without disturbance of the river bed resulting in raised turbidity levels. In most cases, the full shoulders of the piles could not be observed, so the recorded diameters are approximate only.

The average diameter for all piles measured is 9.36 inches (standard deviation of 1.25 inches, coefficient of variation of 13.4%), as presented in **Table 3-5** and **Table 3-6**.

Table 3-5: Measured pile diameters of Subzone B1.

Subzone	Pile	Measured Pile Diameter (in)	Notes
B1	1	10	
B1	2	10	
B1	3	9	Cannot see full shoulder
B1	6	10	
B1	7	9	
B1	8	9	
B1	9	10	
B1	10	10	

Table 3-6: Measured pile diameters of Subzone B2.

Subzone	Pile	Measured Pile Diameter (in)	Notes
B2	1	8	Cannot see shoulder
B2	2	11	
B2	5	8	Cannot see shoulder
B2	6	11	Annulus remaining
B2	7	8	Cannot see shoulder
B2	8	9	Cannot see shoulder
B2	9	9	Cannot see shoulder
B2	10	9	Cannot see shoulder
B2	11	10	
B2	12	10	Cannot see full shoulder
B2	13	9	Cannot see full shoulder
B2	14	9	Cannot see full shoulder
B2	15	10	Cannot see full shoulder
B2	16	9	Cannot see full shoulder
B2	17	7	Cannot see full shoulder
B2	18	9	Cannot see full shoulder
B2	19	8	Cannot see full shoulder
B2	20	7	Cannot see full shoulder
B2	21	11	Cannot see full shoulder
B2	22	9	Cannot see full shoulder
B2	23	11	
B2	24	7	Cannot see full shoulder
B2	25	10	Cannot see full shoulder
B2	26	13	
B2	27	11	Cannot see full shoulder
B2	28	10	Cannot see full shoulder

Subzone	Pile	Measured Pile Diameter (in)	Notes
B2	29	9	Cannot see full shoulder
B2	30	10	
B2	32	8	Cannot see full shoulder
B2	34	8	Cannot see full shoulder
B2	36	10	Cannot see full shoulder

3.6 LABORATORY ANALYSIS

A total of 4 assay samples were collected from various randomly selected piles while onsite then sent to the Wood Research and Development (WRD) independent Testing Laboratory in Jefferson, OR (TL-193) for the purposes of confirming the timber species and assessing the as-received SG and moisture content (MC). The results of these analyses are presented herein.

3.6.1 METHODOLOGY AND EQUIPMENT

Table 3-7 and **Table 3-8** summarize the test methods and measuring equipment for the testing of the collected core samples. All testing was performed in adherence to applicable ASTM standards under the supervision of the WRD laboratory supervisor at WRD's IAS certified, TL-193 testing facility in Jefferson, OR. All equipment was calibrated by 3rd-party calibration agencies according to WRD Laboratory ISO 17025 standards. The results of testing presented in this report apply only directly to the samples tested.

Table 3-7: List of test methods utilized.

Test Method	Description/Purpose
ASTM D2395	Specific Gravity
ASTM D4442	Moisture Content
Microscopic Anatomy	Species Identification

Table 3-8: List of calibrated test equipment utilized.

WRD Equipment No.	Description
WRD 201	Psychrometer
WRD 206	6” Digital Calipers
WRD 210	Gram Scale
WRD 001	Electric Drying Oven

3.6.2 RESULTS SUMMARY

A total of 4 assay samples were received by the laboratory in Jefferson OR on May 18, 2026, by the laboratory manager, Anthony Davis. Three of the samples were divided into 2 sub-samples making a total of 7 individual sub-samples for testing, via ASTM D2395/D4442 to determine the SG/MC of each sub sample. The results of each sub sample were averaged to establish point-estimator values for SG and MC.

3.6.3 WOOD SPECIES IDENTIFICATION

The assay samples collected were analyzed by WRD laboratory technicians to determine the species of the wood as well as to identify if any evidence of treatment was present. The timber species was determined by using light microscopy to observe specific minute features of the cellular structure of the wood. The species of the assay samples is determined by observation of unique combinations of cellular features. The primary references utilized by WRD in species identification are the Inside Wood database (Wheeler, EA 2011), Forest Products Laboratory (FPL) US Department of Agriculture (USDA) Wood Handbook (FPL 2010), and Identifying Wood by Bruce Hoadley.

All assay samples were identified to be White Oak (*Quercus spp*). The features that lead to the identification of White Oak include but are not limited to:

1. A wood ring porous vessel arrangement with abundant tyloses deposits.
2. A dendritic latewood pore pattern.
3. Presence of broad multiseriate rays.
4. Presence of simple perforation plates with alternate inter-vessel pitting.

These features are shown in **Figure 3-5**.

Figure 3-5: Species ID report identifying cellular features of White Oak.

Project Name :	Cabin Creek WV
Project Number :	9141S
Specimen Number :	Pile Material
Specimen Location :	N/A
Date :	27-May-26

Species Identification Results			
Common Name :	White Oak Group		
Scientific Name :	*Quercus spp.*		
Analyzed by :	AD	DT	

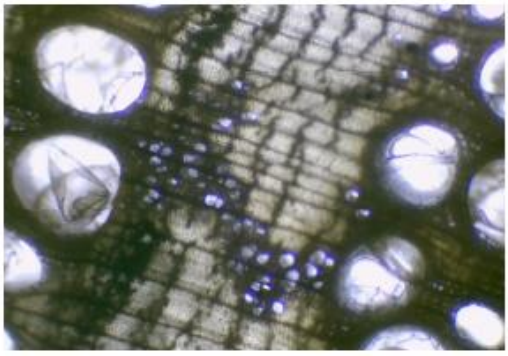


Figure A: Transverse View 1 | Investigation of the transverse face identifies a wood ring porous vessel arrangement with abundant tyloses deposits.

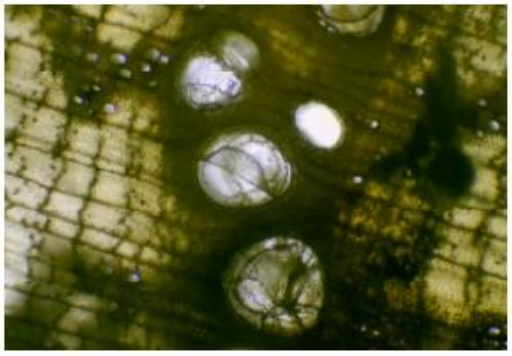


Figure B: Transverse View 2 | Additional investigation of the transverse face demonstrates a dendritic latewood pore pattern.

Figure C: Tangential View 1| Observation of the tangential face confirms the presence of broad multiseriate rays.

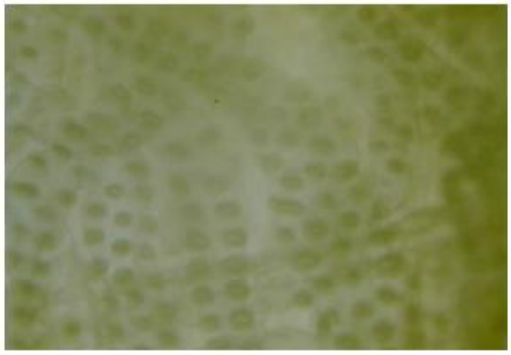


Figure D: Radial View 1 | Observation of the radial face identifies the presence of simple perforation plates with alternate intervessel pitting.

3.6.4 SG & MC TESTING RESULTS

Testing was conducted on the 4 assay samples in accordance with ASTM D2395/D4442 to establish the SG/MC of each sample. The core samples were tested in the as-received condition. Three of the samples were separated into 2 sub-samples for a total of 7 sub samples that were tested. The results of both SG and MC along with important sample information is presented in **Table 3-9**.

It is important to consider the context of the results summary that includes all samples. The dataset presents the average values for SG and MC based upon the testing of the 7 sub-samples. The results are point-estimator averages and do not represent statistically significant design values.

The results of MC testing found that all samples exhibited relatively high moisture contents with an average MC of 27.6%, indicating that fungal decay was possible. The average SG of 0.60 matches the published value for unseasoned (green) White Oak[8].

The detailed data tables for SG/MC testing for all samples, including results of each sub-sample test, are presented in **Appendix A**.

Table 3-9: Summary of results of SG/MC testing on 7 sub samples.

Sample ID	Specific Gravity	Moisture Content
1	0.52	29%
2	0.52	27%
3	0.71	34%
4	0.60	28%
5	0.71	23%
6	0.56	28%
7	0.59	26%
Average	0.60	27.6%
Standard Deviation	0.08	3.37
Median	0.59	27.7

4. SHEAR DESIGN VALUES

Reference design values for timber piles are presented in the NDS (2018) Supplement for typical species used as piles in the United States, for which White Oak is not one of them. These values are in allowable stress design (ASD) format and have been derived from an empirical test cohort to establish strengths at failure loads. This development of design values for wood is because wood is a naturally occurring material, with each tree growing in different patterns. Strength reducing naturally occurring defects, such as knots, grain density and slope of grain, cannot be controlled when machining lumber and therefore the published ASD strength values take a statistical approach by adopting the lower fifth percentile of the test data, which is then factored downwards to incorporate a safety factor, an adjustment for duration of load from test duration to "normal" duration of 10 years (which is representative of structures accommodating live loads, see more information below), and other features.

The published values do not account for the service conditions relevant for shear, which are duration of load, C_D, (other than a 10-year duration), temperature, C_t, and condition treatment, C_{ct}. Adjustments for the latter two service conditions are not relevant to the piles at Cabin Creek because the service temperature does not exceed 100° F and it is unlikely the piles were dried prior to installation (drying large dimension timber and logs can take many months).

Load duration requires careful consideration. Creep is a time-dependent deformation of wood under load. If the load is sufficiently high and the duration of load is long, failure (creep–rupture) will eventually occur. The time required to reach rupture is commonly called duration of load. If the soil is at rest until a landslide event occurs, the duration of load during the land slide event could be near instantaneous and C_D could be up to a value of 1.8-2.0. If, however, the embankment is unstable and model considers constant active pressure against the piles, the duration of load would be considered constant, much in the same way dead load or self-weight is constant, and C_D would be 0.9. The NDS (2018) provide a graphical representation of C_D with respect to different durations of load, as shown in **Figure 4-1**.

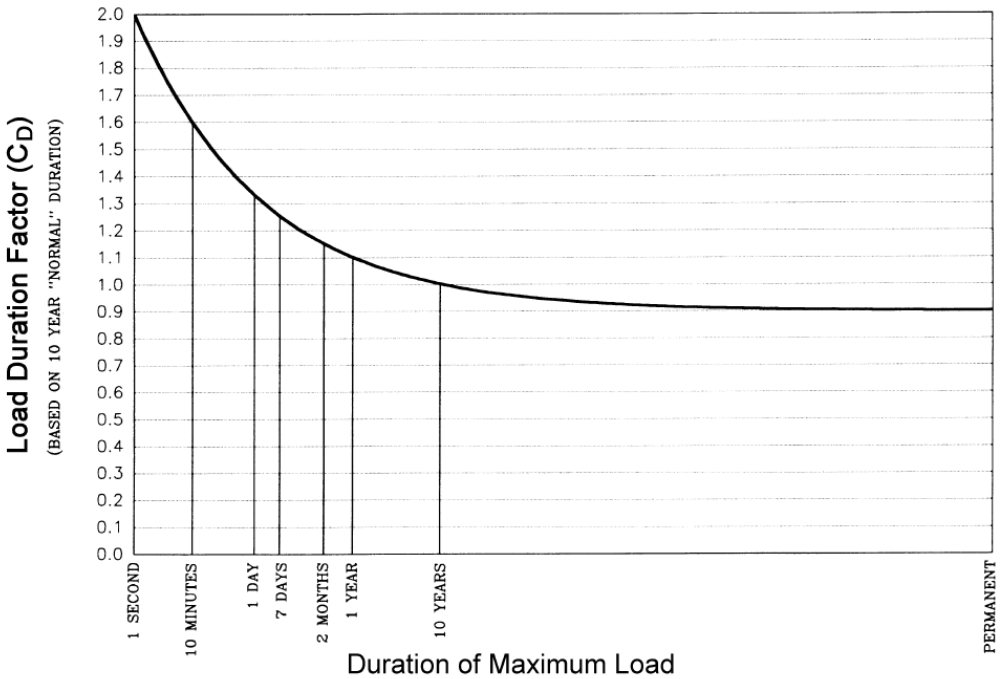


Figure 4-1: Load duration factor, C_D, for various durations of load.

Several publications offer average values for shear parallel to grain for White Oak. The Wood Handbook[8] by the FPL is regarded as the preeminent publication of wood science for construction purposes in the United States, which states an average shear parallel to grain strength of 1250 psi, which is considerably higher than lower fifth percentile strength values in the NDS (2018) Supplement. However, adopting an average value is non-conservative and not recommended.

Without published design values for White Oak piles, it is necessary to take a statistical approach to developing allowable design values. It is not appropriate to select design values of White Oak for sawn lumber presented elsewhere in the NDS (2018) Supplement because the process of machining lumber to square or rectangular sizes for typical construction intercepts the wood fibers because the fibers are generally not parallel to the cut face, and the slope of grain means fibers are discontinuous. This discontinuity of fibers results in reduced strength, particularly in tensile zones. Timber piles are not sawn and therefore need not be penalized with reduced design values. As a comparison, **Table 4-1** presents design values as published by the NDS (2018) Supplement for Douglas-fir Larch as Grade Dense No. 1 as Post and Timbers (size

classification) and piles. Generally, the design values for the piles are higher than the sawn lumber and note the tension parallel to grain value for piles is not published.

Table 4-1: Comparison of design values of Post and Timbers and Piles of Douglas-fir Larch as presented in the NDS (2018) Supplement.

Design Value	Post and Timbers Grade – Dense No. 1	Piles Grade – N/A
Bending	1,550 psi	2,050 psi
Tension parallel to grain	1,000 psi	N/A
Shear parallel to grain	170 psi	160 psi
Compression parallel to grain	1,100 psi	1,300 psi
Compression perpendicular to grain	625 psi	490 psi
Modulus of elasticity	1,700,000 psi	1,700,000 psi
Minimum modulus of elasticity	620,000 psi	690,000 psi

Bodig & Jayne[4] gives the same mean value of the shear parallel to grain strength of White Oak of 1250 psi and with a standard deviation of 250 psi. By considering a normal distribution of test values, the following can be derived:

$$\text{PTL} = \hat{X} - Ks = 0.05$$

Where PTL is the parametric tolerance level (the population exceeding the lower fifth percentile, 0.05) and K is the one-sided tolerance limit for normal distribution. $\hat{X}$ is the mean and s is the standard deviation. ASTM D2915[1] is commonly used to determine allowable design values from test data, and based on a typical 75% confidence interval, the value for K for the total 47 piles considered is 1.818 (Table 3 of D2915, interpolated). The result is a lower fifth percentile value in shear parallel to grain of 931 psi. ASTM D2899[2] gives the adjustment factor of 1/2.3 to convert the lower fifth percentile value to a reference design value of 2.1. The resulting reference design value in shear parallel to grain is:

$$F_v = 405\ psi$$

This value is appropriate if the soil is expected to yield, allowing the pile to deform. This value requires adjustment for duration of load as discussed above.

It is understood that WSP requires shear strength values perpendicular to the pile cross-section for their assessments. This would be relevant to a shear plane condition and without the soil yielding around the pile to allow it to laterally deform. As noted above, published design values for shear strength are considered parallel to grain. Because wood is highly orthotropic, it is very difficult to fail in shear perpendicular to the grain. Attempts to obtain shear failure perpendicular to the grain usually result in failure in another failure mode, such as compression perpendicular to the grain, which is based on a deformation limit, it is not a strength criterion. Adopting a transverse shear value is nonconservative and not recommended. Piles inground subject to lateral loads are subject to bending stresses, typically for the full length of the pile, meaning the shear stress parallel to grain will exist and therefore govern over transverse shear stress.

If the geotechnical model considers load and resistance factored design (LRFD) principles, the allowable shear values discussed above should be factored by 2.88 (Table 6.3.1, NDS 2018) to obtain applicable factored resistance values. The load duration factor, C_D, does not apply to LRFD design; the appropriate time effect factor, λ, shall be applied to the LRFD value in lieu of C_D, as either:

- For soil at rest prior to landslide event $\lambda = 1.0$
- For active soil pressure $\lambda = 0.6$

To interpret the discussion above, **Table 4-2** summarizes the shear design values to be used, rounded to the nearest 5 psi.

Table 4-2: Summary of shear design values.

Shear value for design	Earth Behavior	ASD Design	LRFD Design
Adjusted shear Parallel to grain	At rest	730 psi	1165 psi
	Active	365 psi	700 psi

5. CONCLUSION

Wood Research and Development (WRD) was commissioned by WSP to perform an inspection and structural assessment of the timber piles at the AEP Cabin Creek Ash Pond, Former Cabin Creek Power Plant, West Virginia. This report summarizes the findings of the inspection and the key data required for WSP's geotechnical slope stability analysis.

Visual assessments found that all identifiable piles exhibit complete section loss by decay at and above water level. At ground level, the shoulders of the piles revealed the possible original diameters of an average of 9.36 inches, which indicates the piles were originally ordered for a 10 inch diameter.

SWT across the piles were recorded to determine remaining SG (and strength). Testing of this nature into heavily decayed wood simply confirmed the same. In combination with the partial visual identification of the original pile diameters, and the almost certainty that decay by fungal colony cannot occur underwater where oxygen does not exist, it has been concluded that the piles underground (underwater) remain in sound condition.

Ground level is likely to have changed over time, but the historical drawings indicate the piles are approximately 31-32 feet into ground. Using pulse echo technology, the pile depths were measured to be an average of 32.9 feet.

Requests were made for transverse shear design values for the purpose of geotechnical modelling of the embankment. Published shear values for wood are parallel to grain shear values. Transverse shear values are much higher and are typically not reached before some other mode of failure. Piles inground subject to lateral loads are subject to bending stresses, typically for the full length of the pile, meaning the shear stress parallel to grain will exist and therefore govern over transverse shear stress. Wood design values for strength properties such as shear are based on a statistical assessment of empirical test data where the lower fifth percentile value is used then factored downwards to include a safety factor and convert the value based on the duration of loading during testing compared to a normal duration (10 years) for design purposes. The NDS (2018) does not publish design values for White Oak piles. Mean shear parallel to grain values with standard deviation were adopted from widely cited sources and converted into an allowable design value of 405 psi. This value must then be adjusted for duration of load based on the assumptions made by the geotechnical model. If at rest prior to a landslide event, an allowable

shear value of 730 psi can be used. If there is constant active pressure, an allowable shear value of 365 psi must be used. Factored shear values are also included if the geotechnical model follows Load and Resistance Design practices.

WRD thanks WSP for the opportunity to contribute to their work and looks forward to discussing these results further as necessary.

6. REFERENCES

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7. APPENDIX A – MC & SG TEST RESULTS

Moisture Content - ASTM D4442 (Method B - Oven Drying Secondary)

Specific Gravity - ASTM D2395 (Method E) / ASTM D2395 (Method A)

Project #:	9099S-02	Equipment Used:			Oven Temp (F): 217
Project Name:	Barber Dam	Parameter	Equip ID #	Cali Due:	RH (%): 43
Date:	5/26/2026	Mass:	WRD210	March 20267	Lab Temp (F): 64
Recorded By:	AD	Dimensions:	WRD206	March 2027	
Checked By:	CTM	Oven Temp:	WRD001	March 2027	Start Date/Time: 5/26/2026 10:00
		Lab RH/T/Dp:	WRD201	October 2027	End Date/Time: 5/27/2026 12:00
		Date:	5/26/2026	5/27/2026	5/27/2026
		Time:	10:00	8:00	12:00

Sample ID	Origin	L_i (in)	W_i (in)	H_i (in)	Initial Vol.	$Mass_i$ (g)	M_2	M_3	M_4	M_O	L_O	W_O	H_O	Final Vol.	SG	MC
S1	N/A	0.880	0.889	0.343	0.268	2.563	1.995	1.989			0.882	0.86	0.307	0.233	0.52	29
S2	N/A	0.871	0.337	0.351	0.103	1.101	0.871	0.869			0.91	0.333	0.338	0.102	0.52	27
S3	N/A	0.787	0.433	0.345	0.118	1.554	1.162	1.162			0.783	0.406	0.316	0.100	0.71	34
S4	N/A	0.992	0.428	0.491	0.208	2.344	1.833	1.835			0.977	0.404	0.475	0.187	0.60	28
S5	N/A	0.929	0.407	0.341	0.129	1.597	1.301	1.301			0.923	0.395	0.308	0.112	0.71	23
S6	N/A	0.769	0.388	0.347	0.104	1.068	0.836	0.836			0.762	0.375	0.32	0.091	0.56	28
S7	N/A	0.569	0.327	0.399	0.074	0.812	0.648	0.647			0.529	0.4	0.318	0.067	0.59	26
														Average:	0.60	27.58
														StDev:	0.08	3.37
														Median:	0.59	27.74

Notes:

Specific Gravity	The overall value of SPECIFIC GRAVITY was determined to be 0.60 based on the tests of 7 specimens in accordance with Specific Gravity ASTM D2395 (Method A - Volume by Measurement)
Moisture Content	The overall MOISTURE CONTENT was determined to be 27.58% based on the tests of 7 specimens in accordance with Moisture Content ASTM D4442 (Method B - Oven-drying Secondary)

8. APPENDIX B – PHOTOGRAPHS

8.1 SUBZONE B1 PILES

Pile 1

Pile 2

Pile 3

Pile 4

Pile 5

Pile 6

Pile 7

Pile 8

Pile 9

Pile 10

8.2 SUBZONE B2 PILES

Pile 1

Pile 5

Pile 6

Pile 7

Pile 8

Pile 9

Pile 10

Pile 11

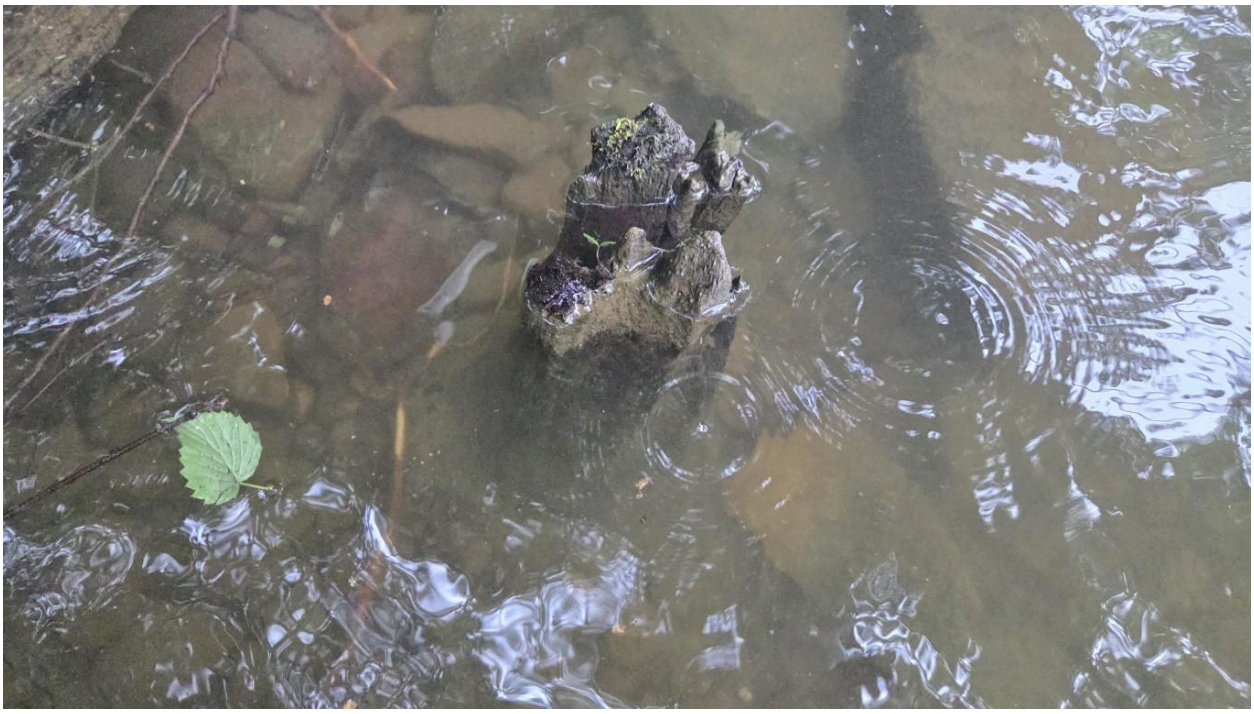
Pile 12

Pile 13

Pile 14

Pile 15

Pile 16



Pile 17

Pile 18

Pile 19

Pile 20

Pile 21

Pile 22

Pile 23

Pile 24

Pile 25

Pile 26

Pile 28

Pile 29

Pile 32

Pile 33

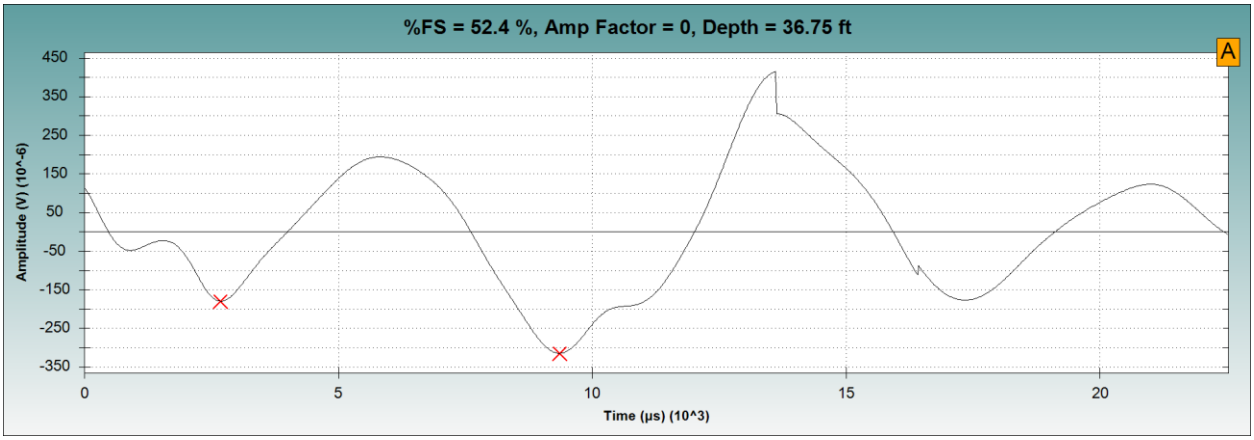
Pile 34

Pile 35

Pile 36

9. APPENDIX C – PULSE ECHO TECHNOLOGY (PET) TEST DATA

Pile 1, Zone B1



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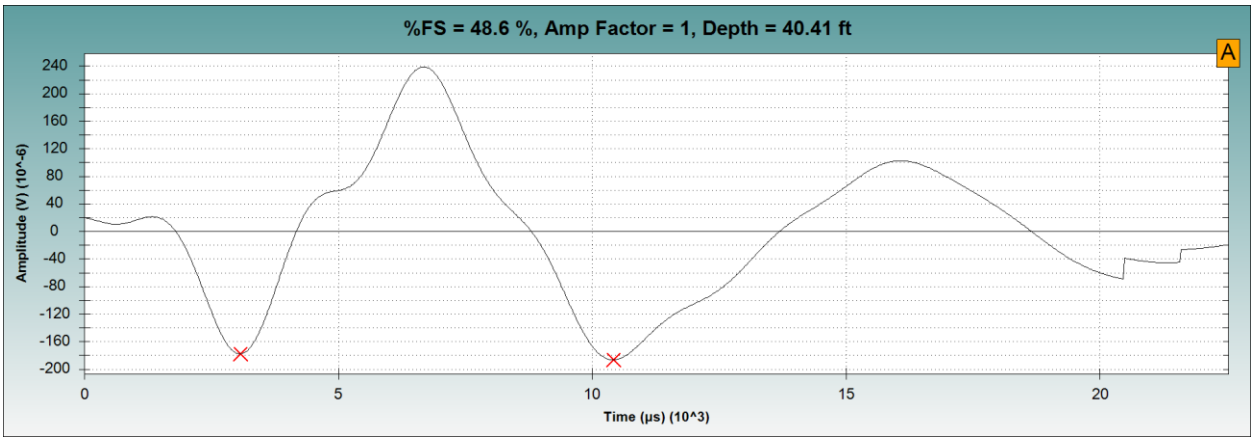

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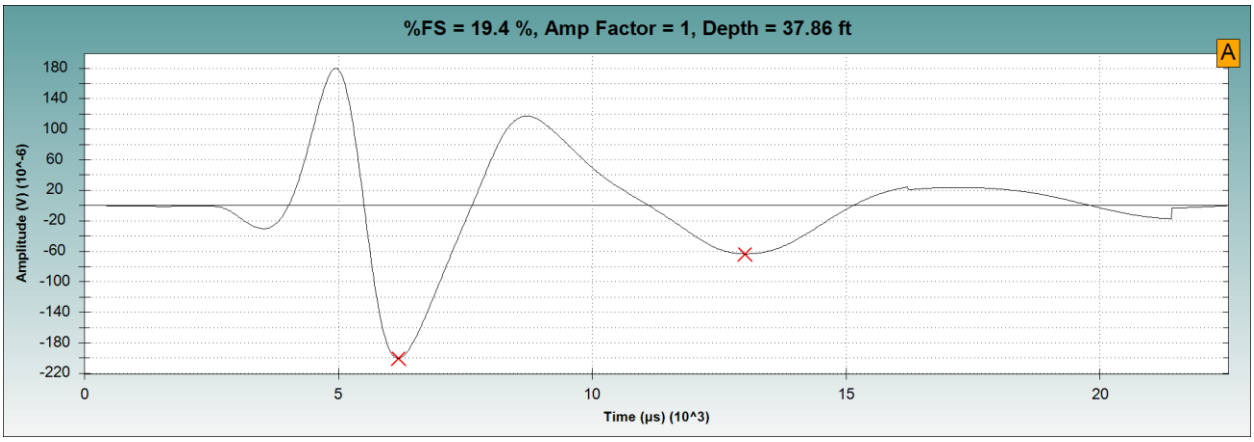

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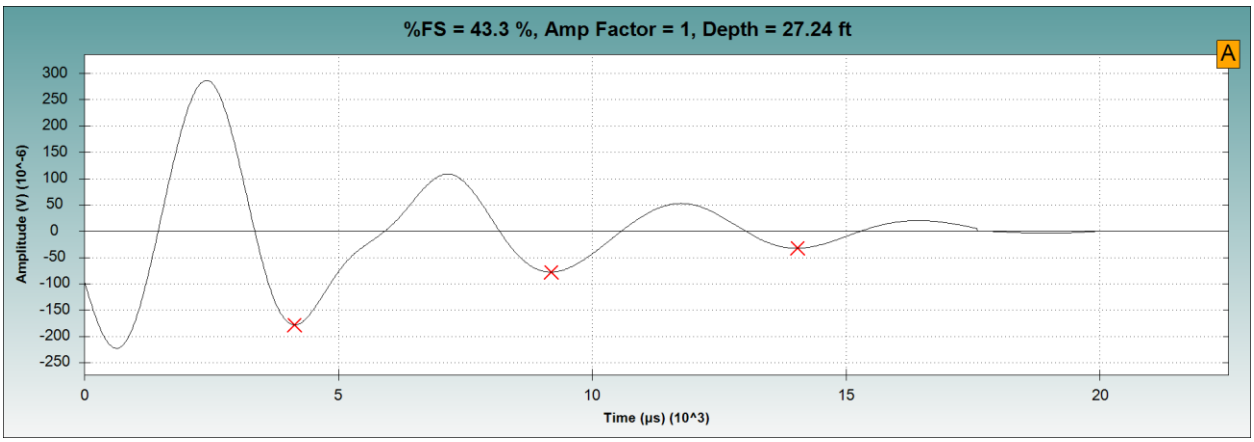
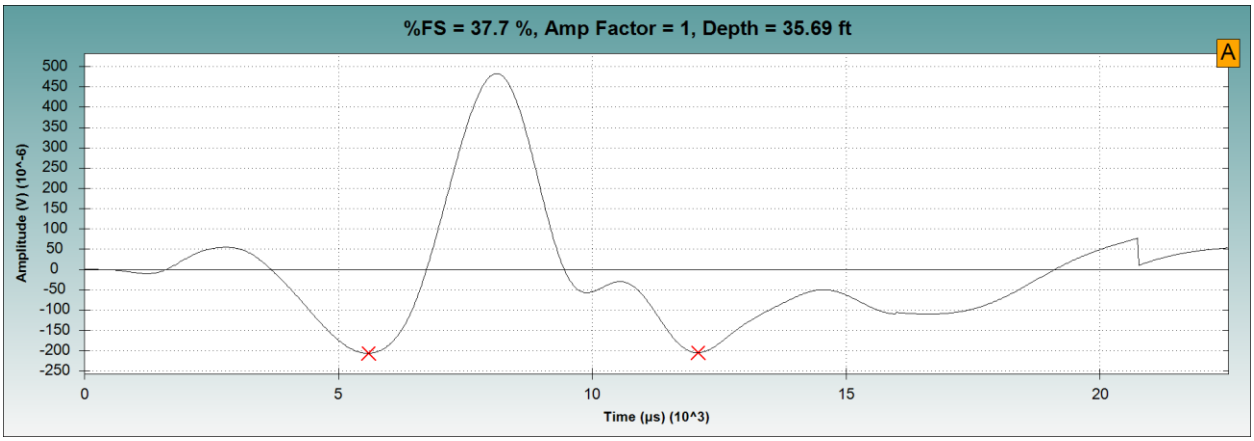


Pile 6, Zone B1



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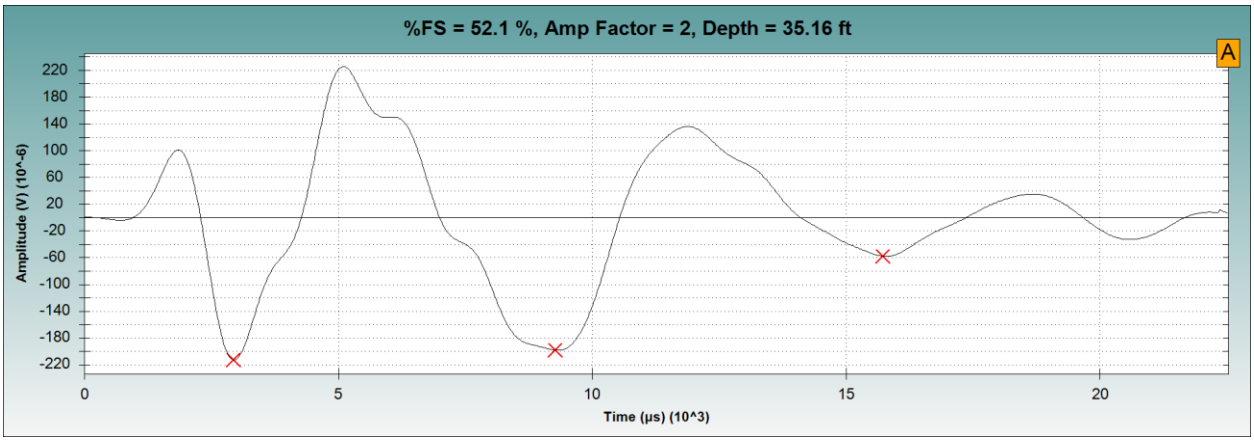

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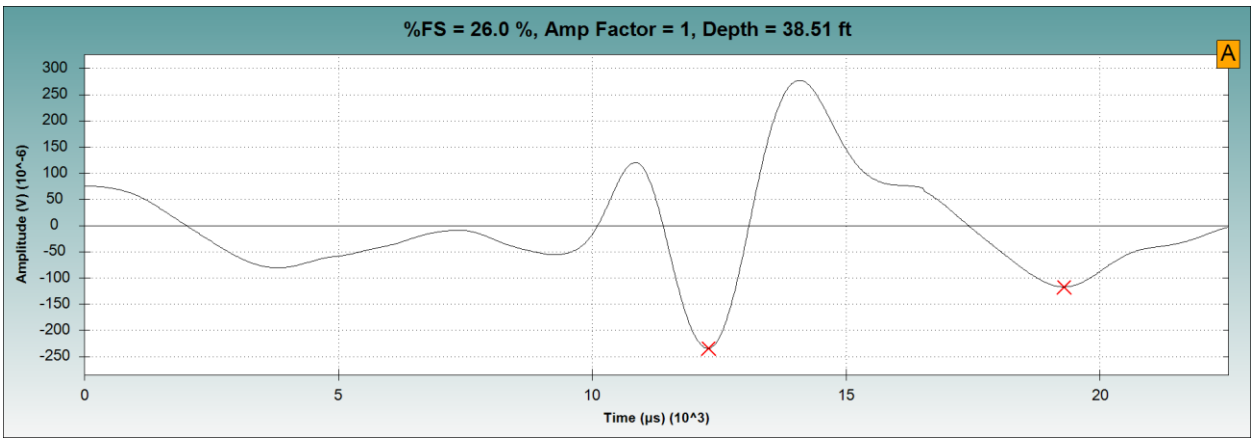

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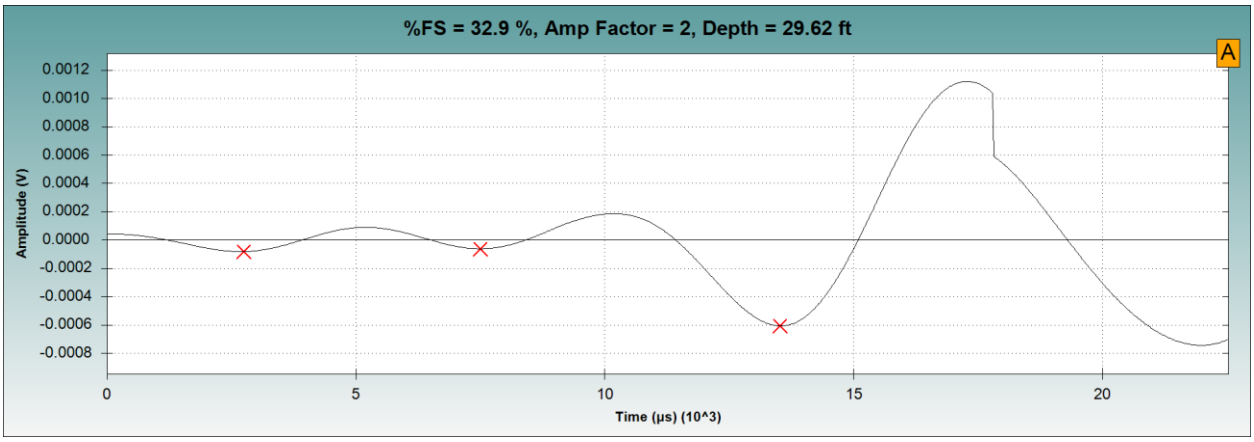

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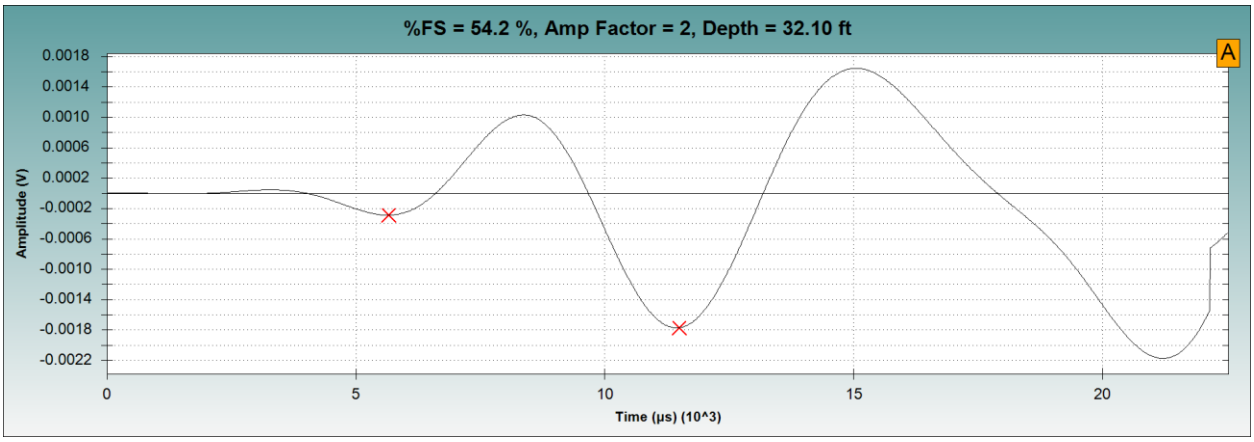

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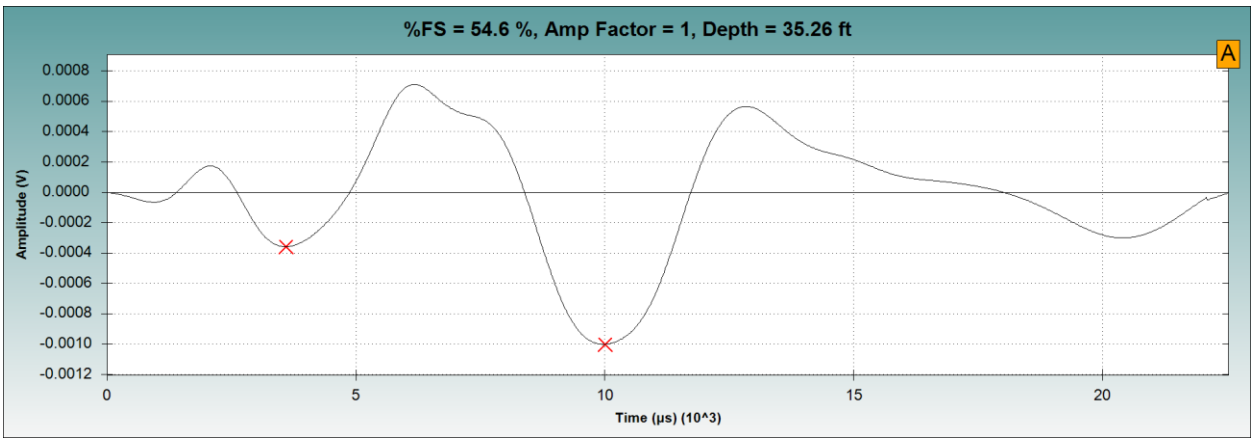

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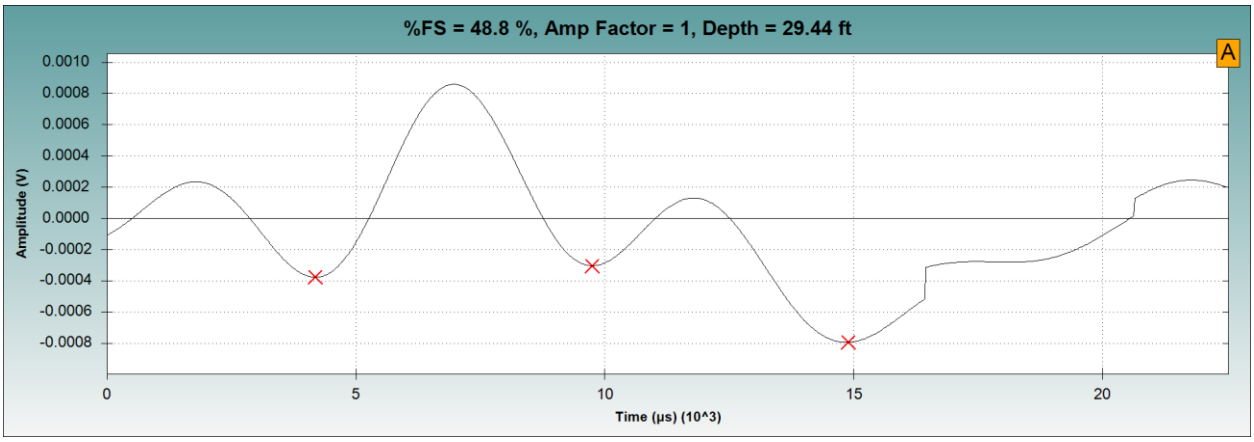

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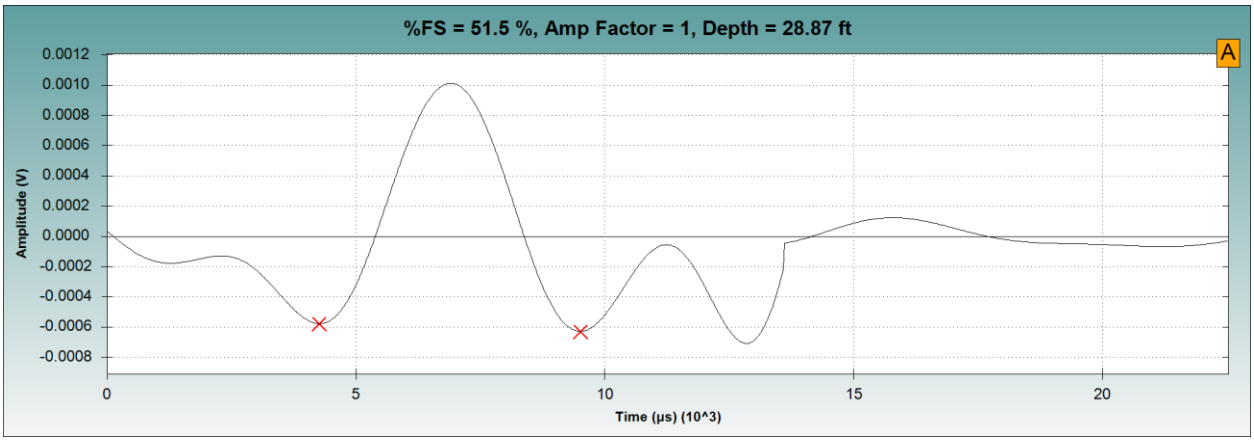

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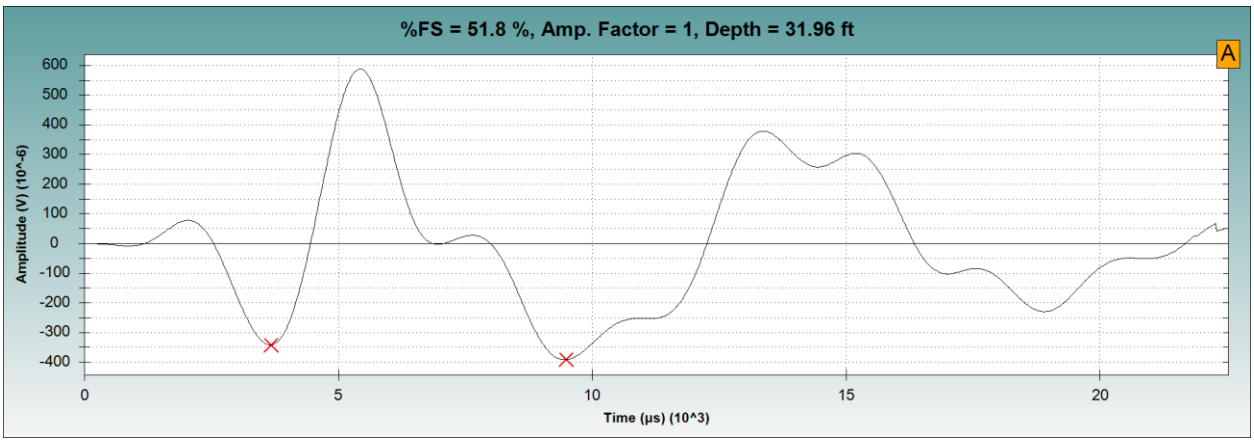

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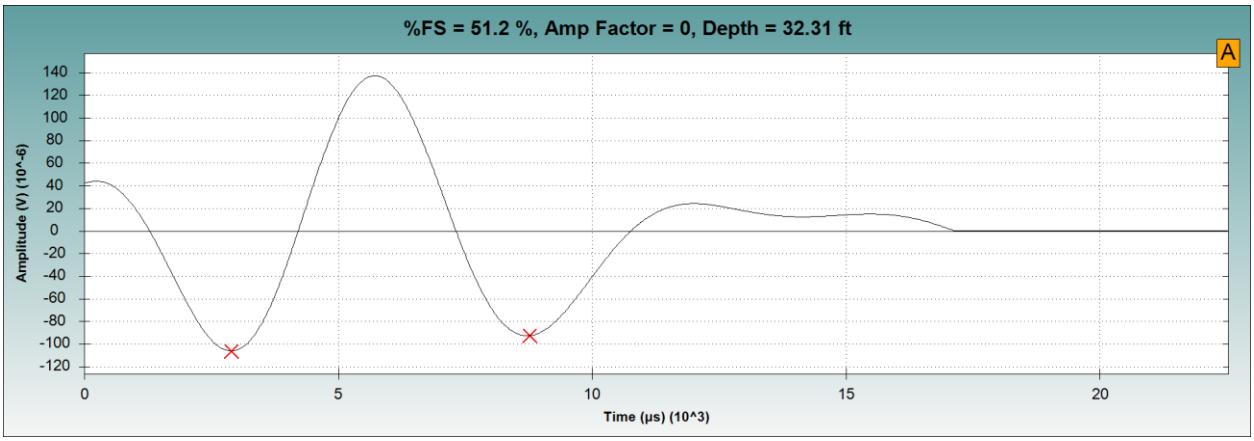

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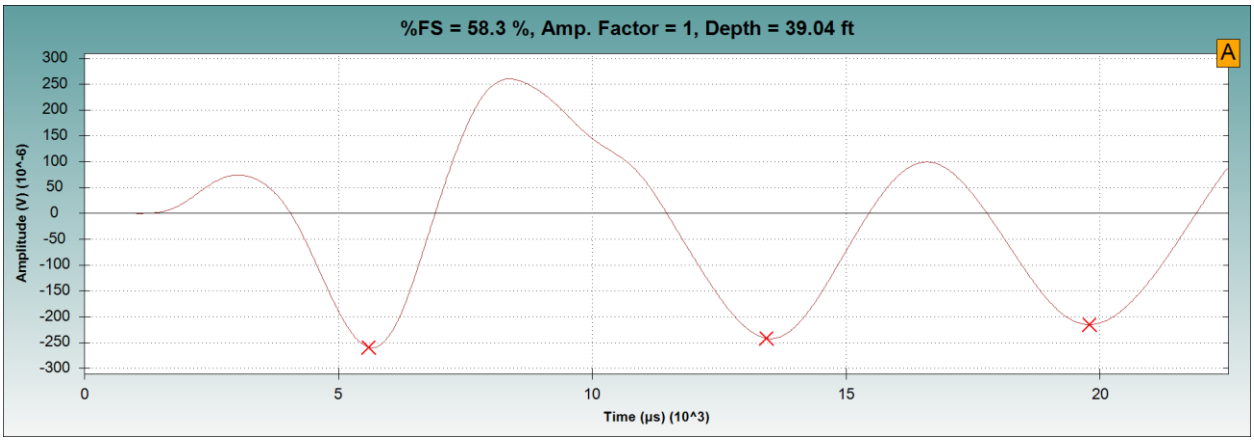

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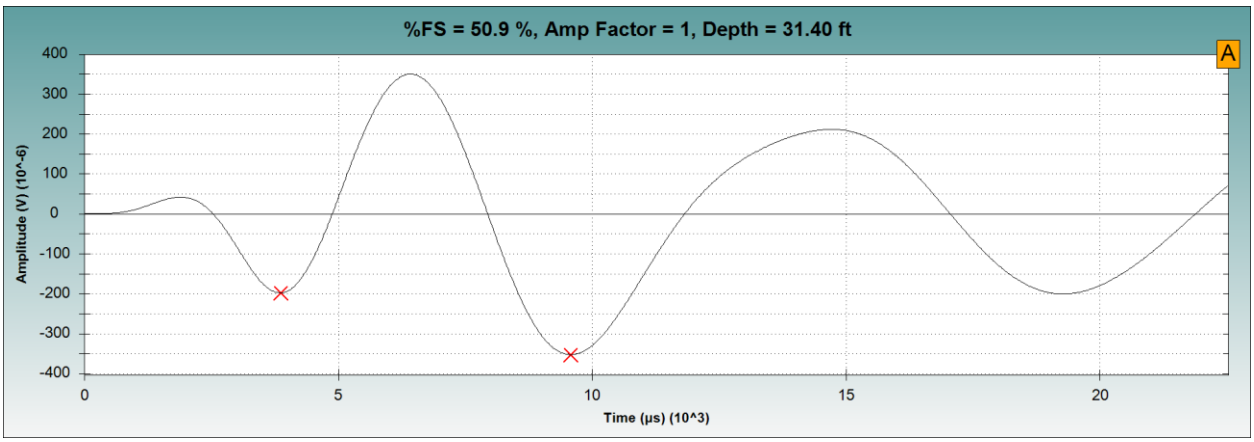

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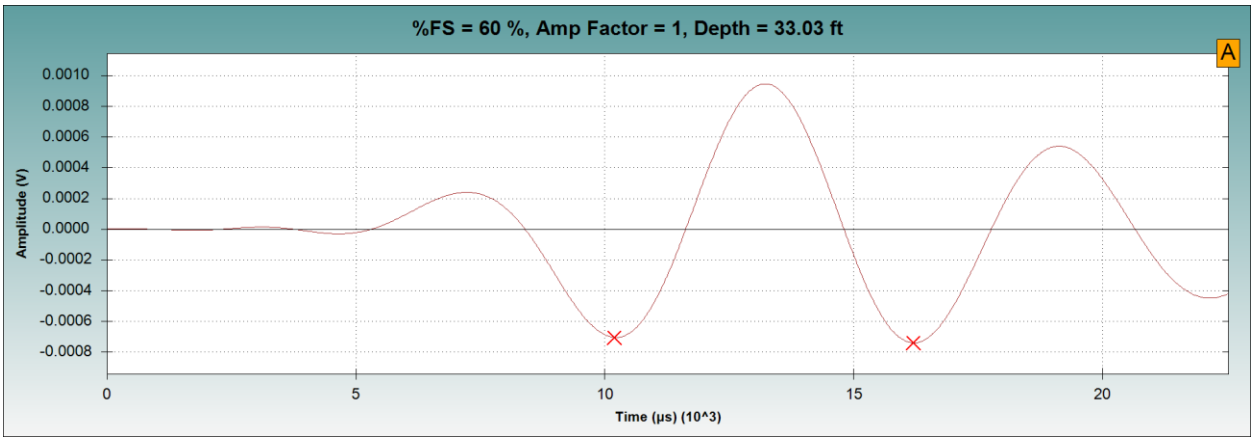

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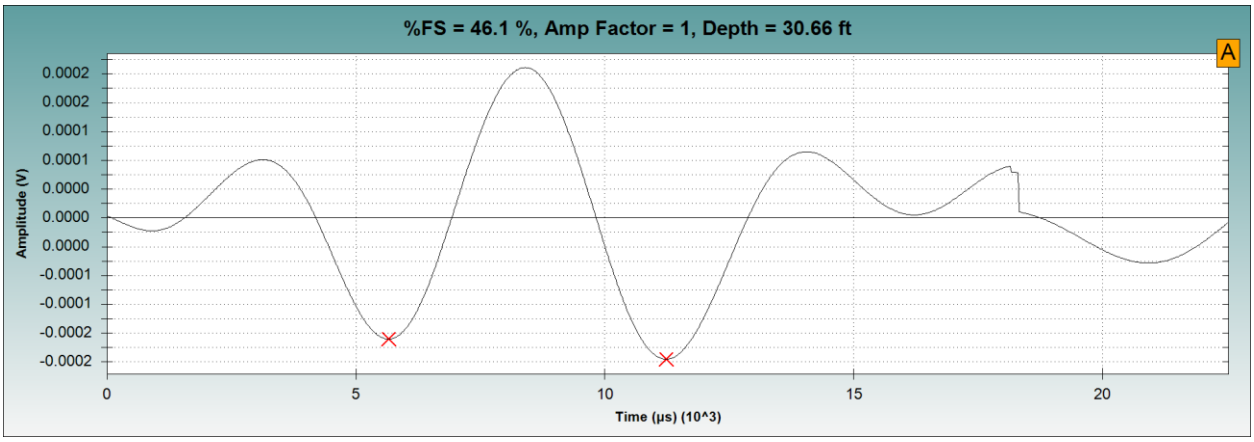

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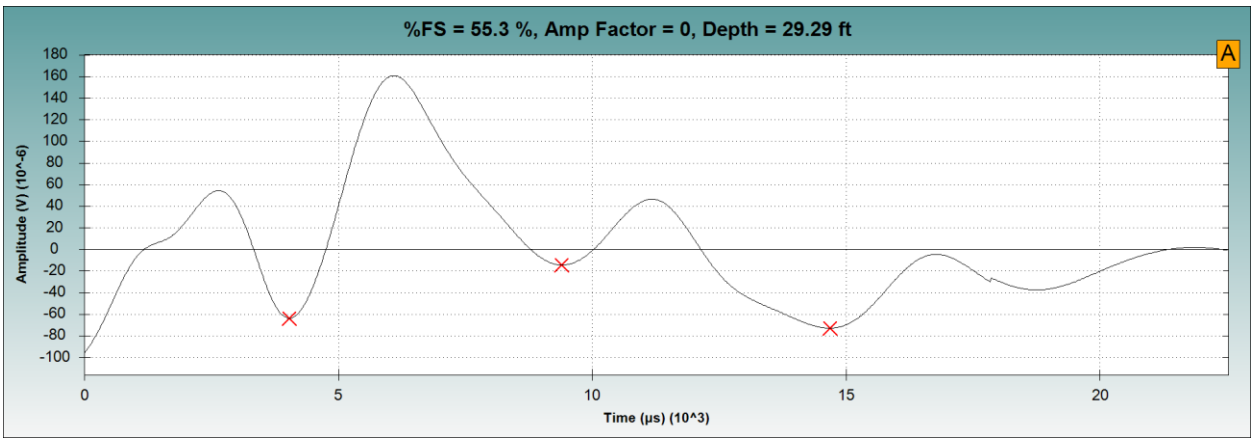

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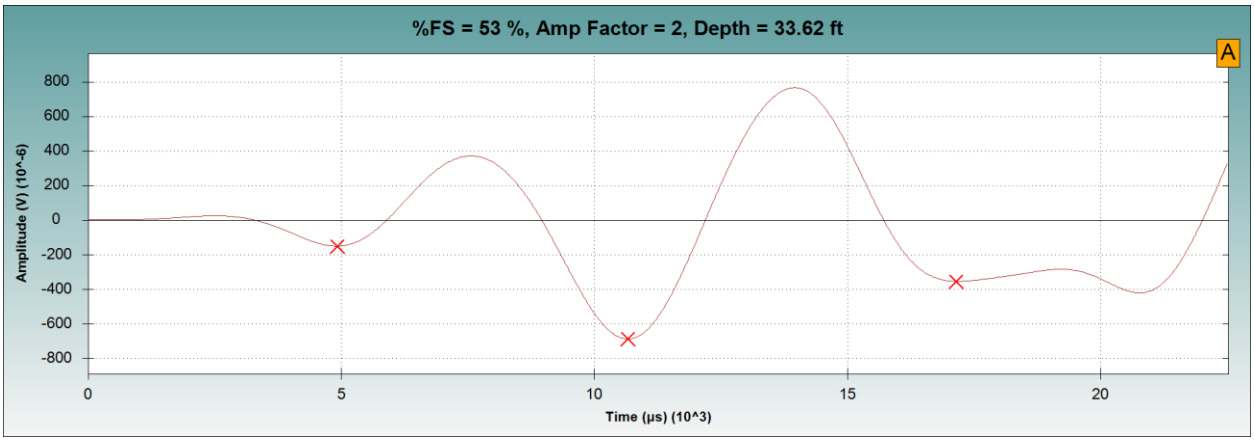

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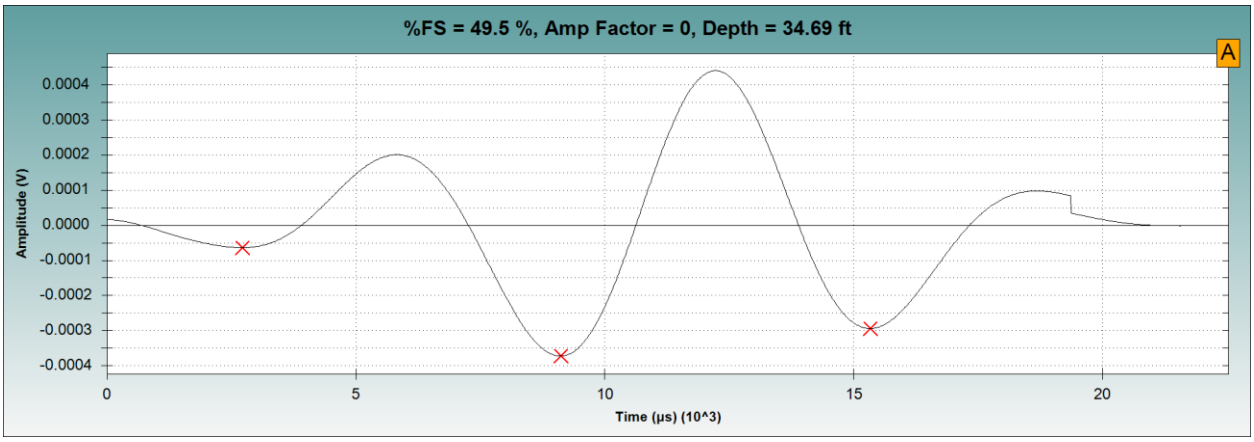

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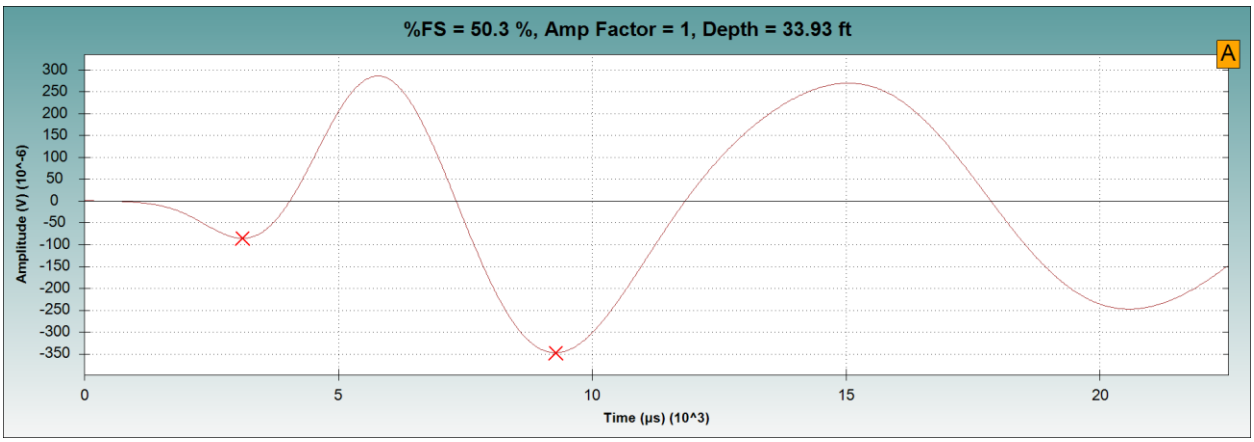

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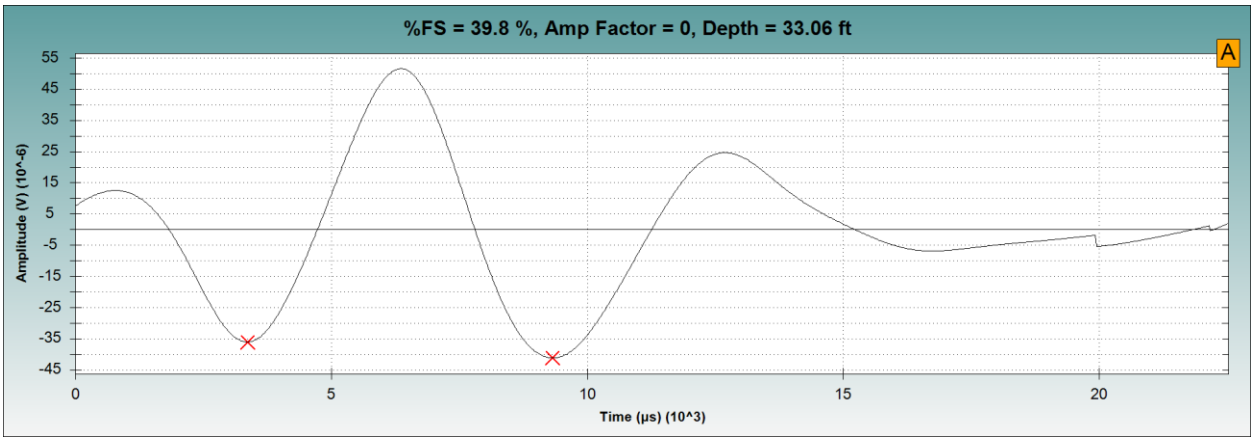

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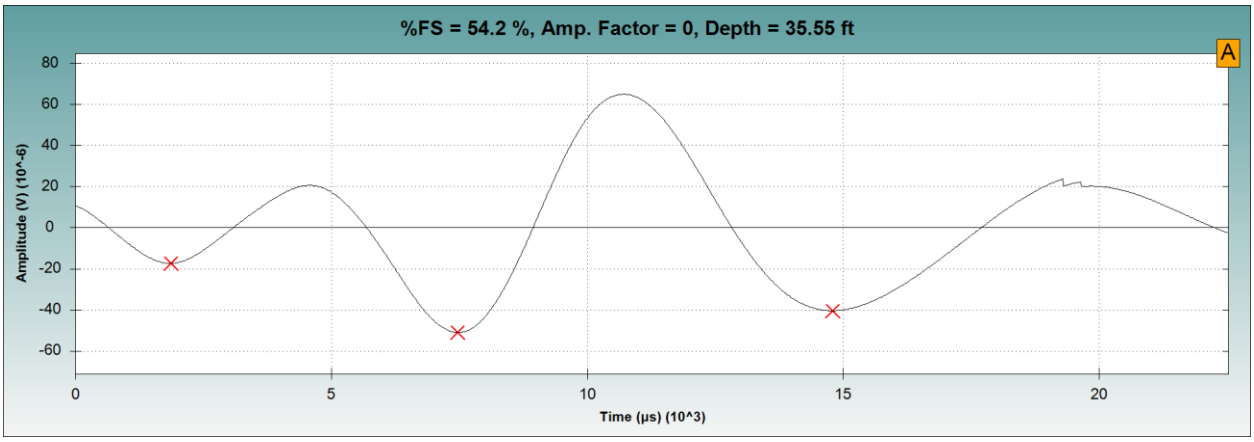

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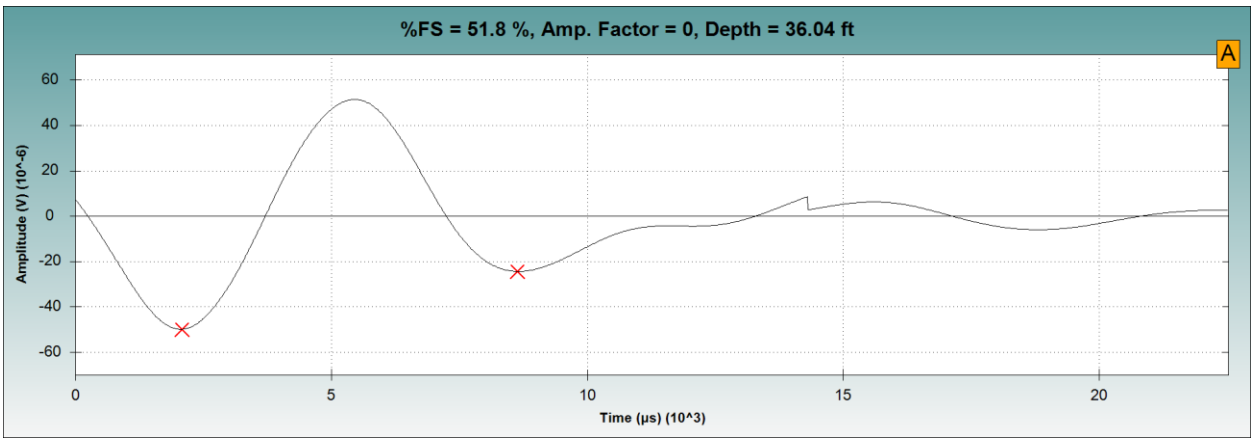

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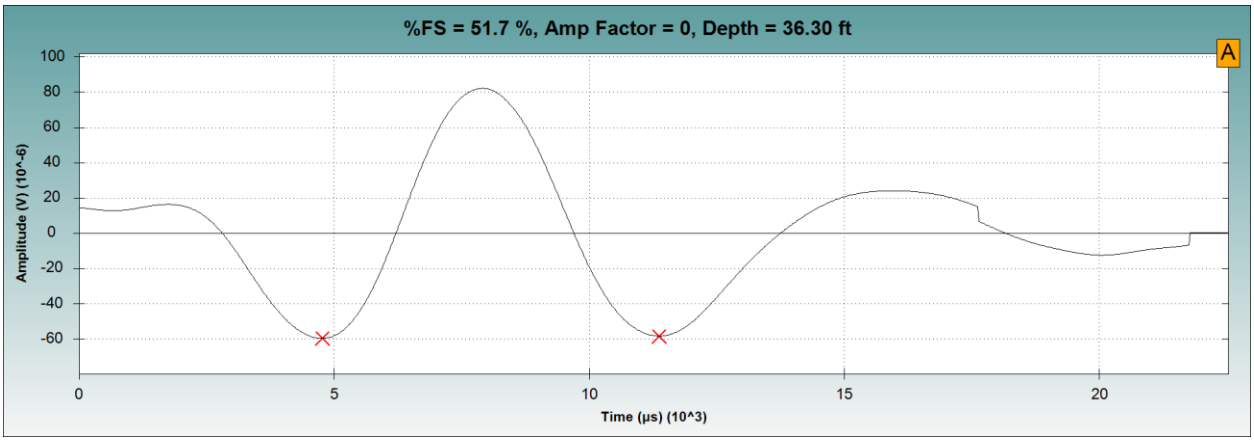

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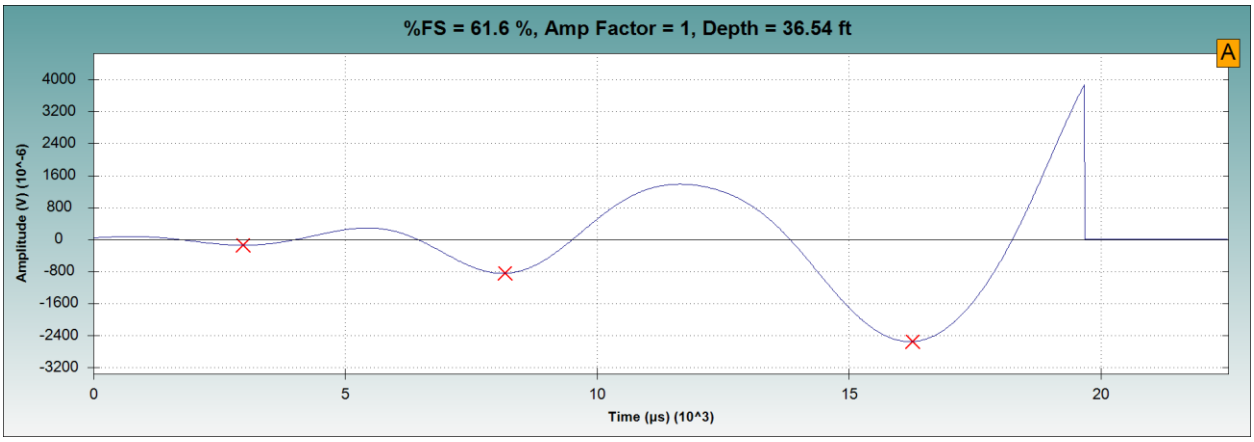

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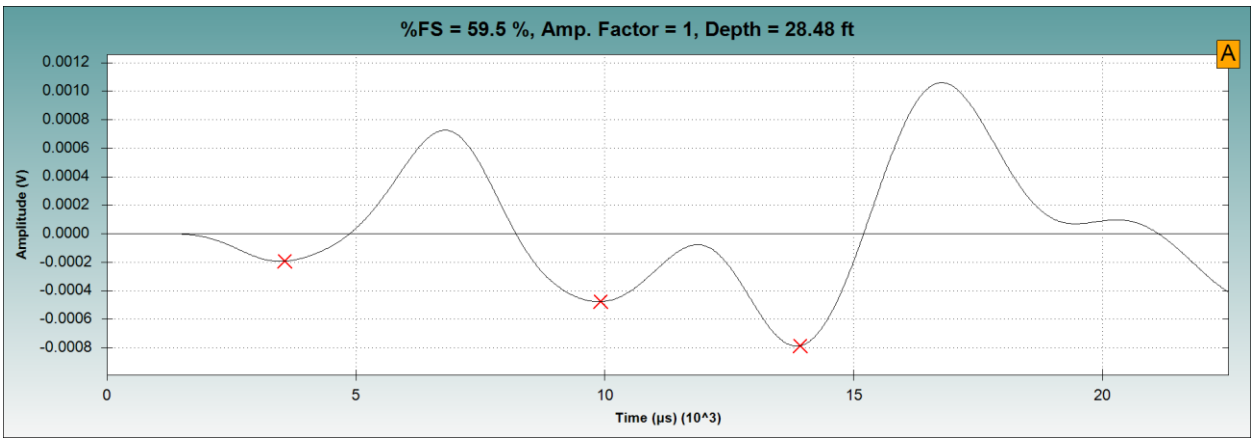

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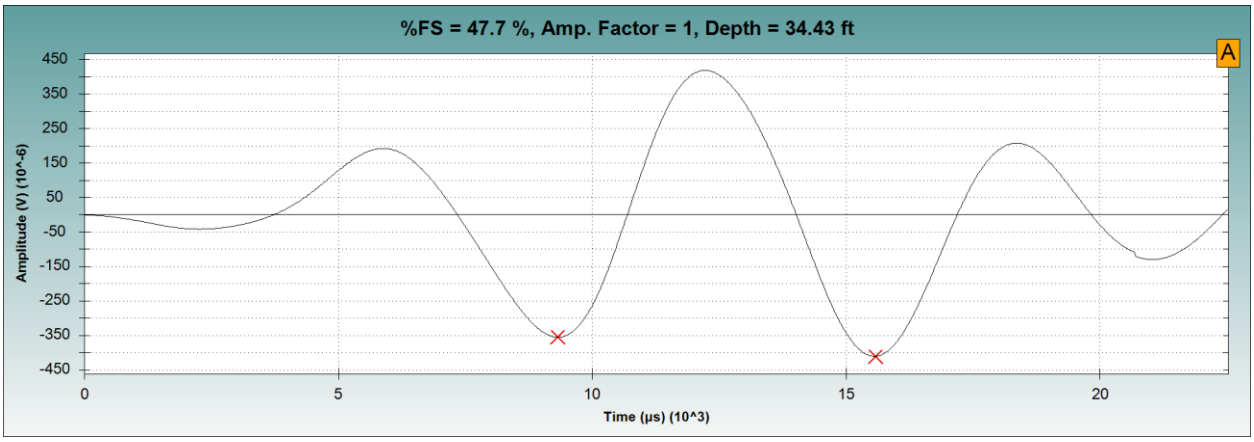

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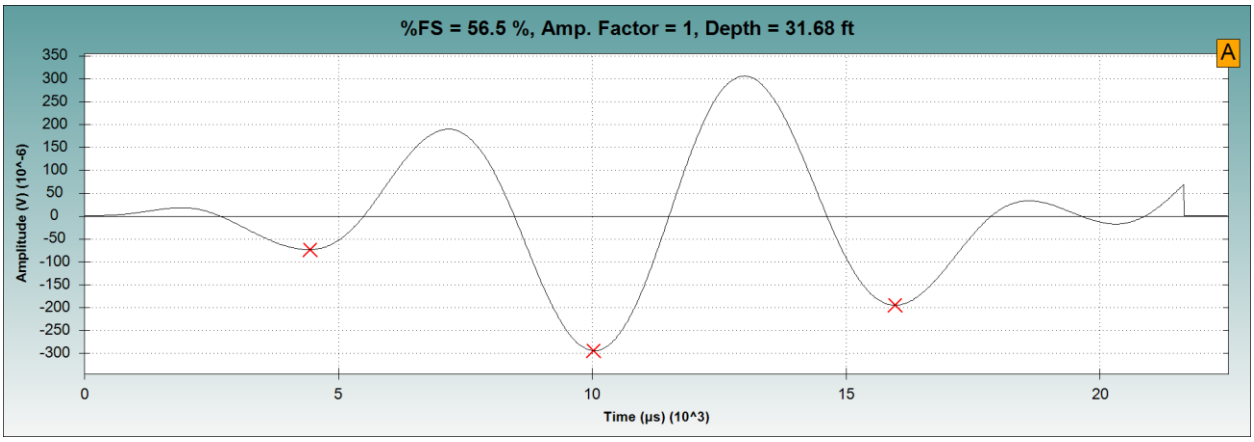

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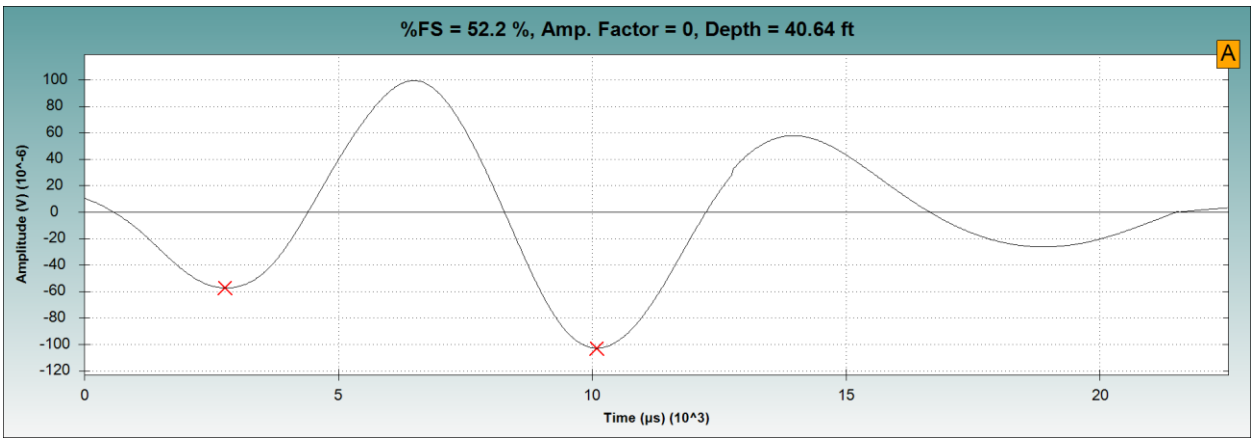

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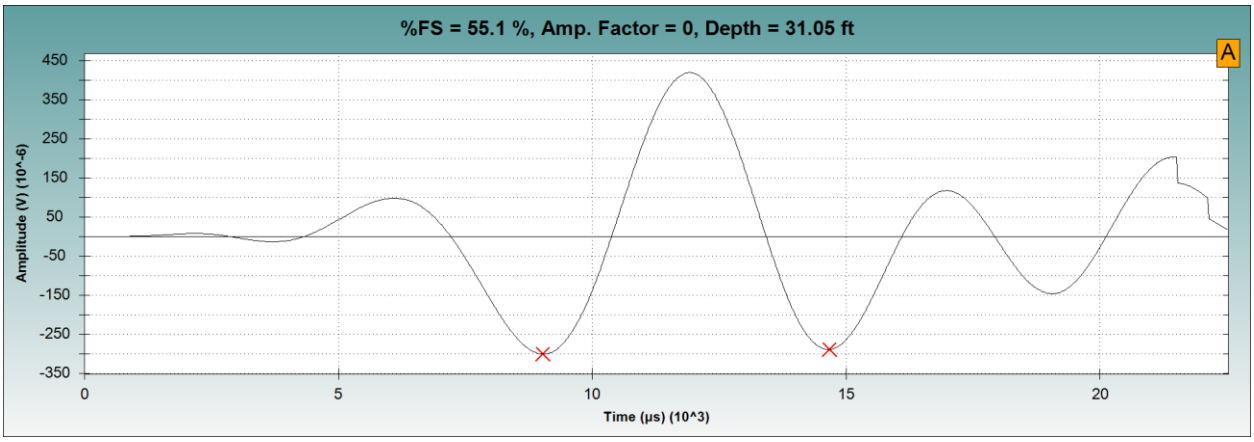

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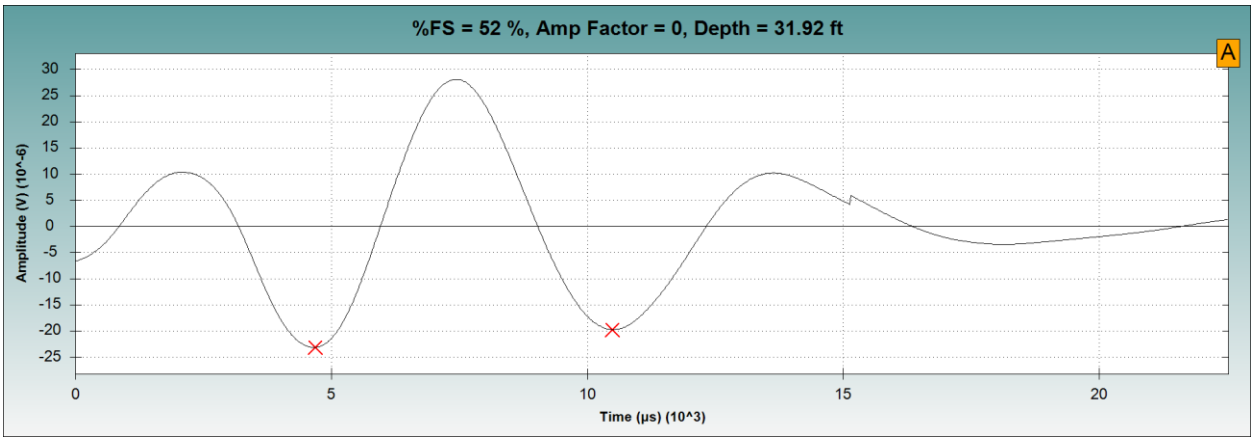

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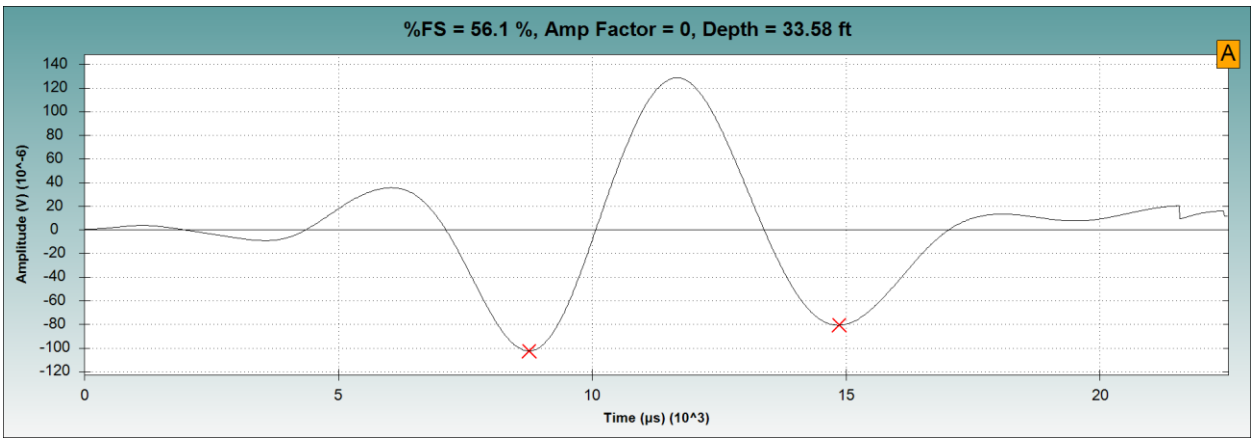

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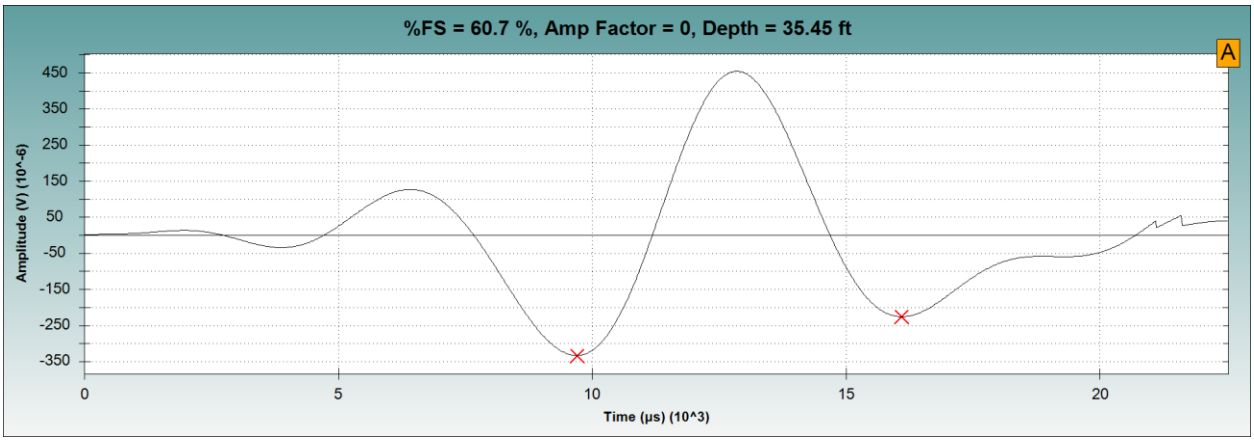

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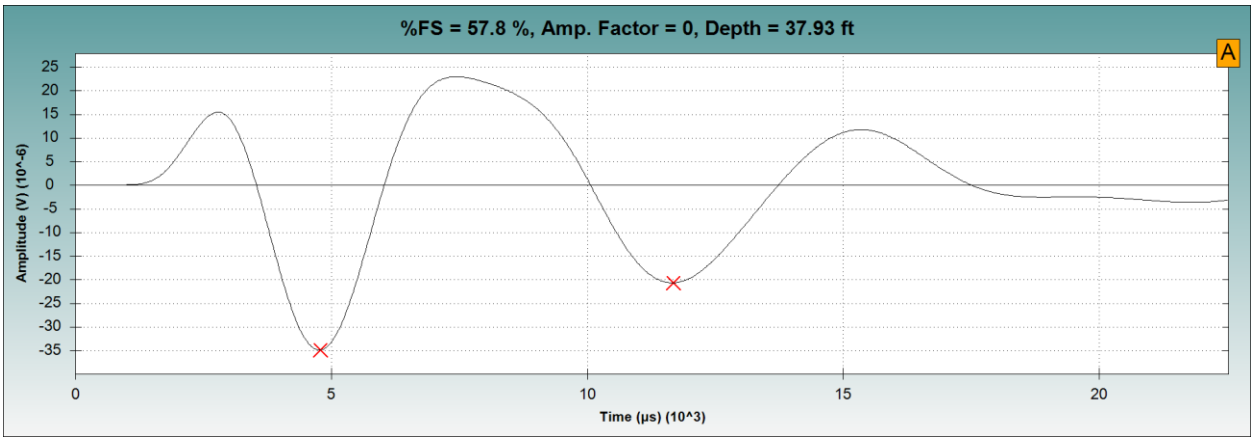


APPENDIX C

Geotechnical Strength Selection

APPENDIX C: GEOTECHNICAL STRENGTH SELECTION

DATE July 1, 2026 **Project No.** US0041868.3860
TO AEP
FROM WSP

1 INTRODUCTION

The following calculation package provides an assessment of the soil strength data collected for the former Cabin Creek Power Station (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 Cabin Creek Ash Pond site 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), Particle Size Distribution (ASTM D 6913), Specific Gravity (ASTM D854), and Atterberg Limits (ASTM D4318). Additionally, limited Direct Shear Tests 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 effective cohesion (c') and effective internal friction angle (φ') for each material type. Laboratory strength data were additionally used to calculate total (undrained) shear strength parameters, where appropriate, to support stability analyses under short-term loading conditions.

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 the φ'. The calculations followed procedures outlined in ASTM D1586 and supplemented by published correlations (Peck, Hanson & Thornburn, 1974).

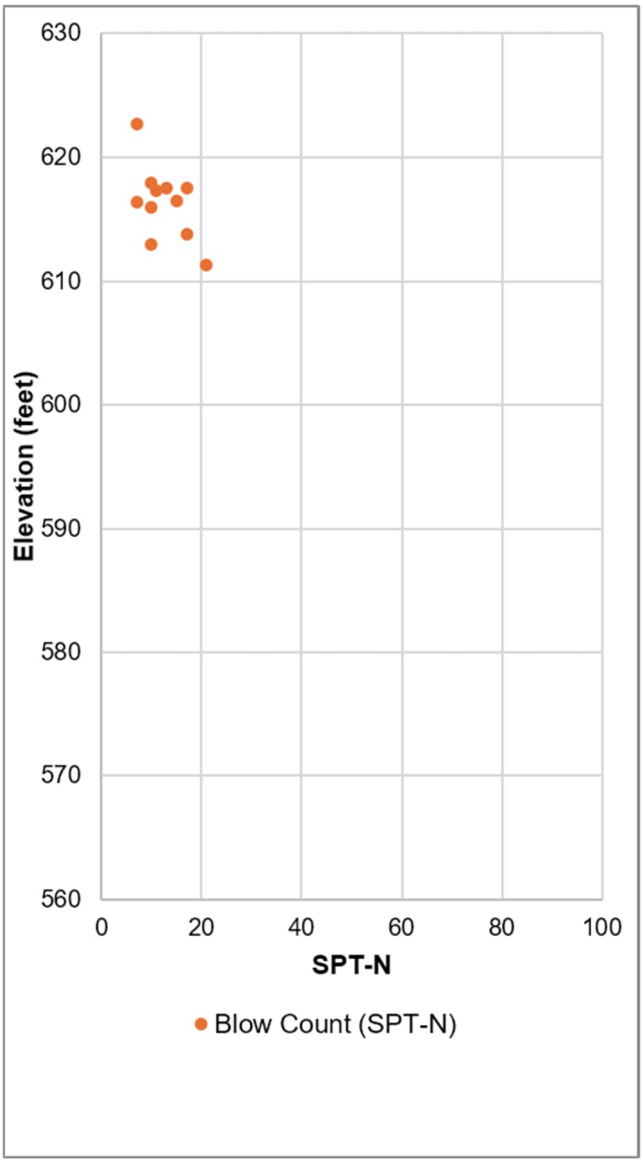
Alluvial and Earthfill strength parameters were developed using a comprehensive dataset from the Cabin Creek Ash Pond and adjacent sites.

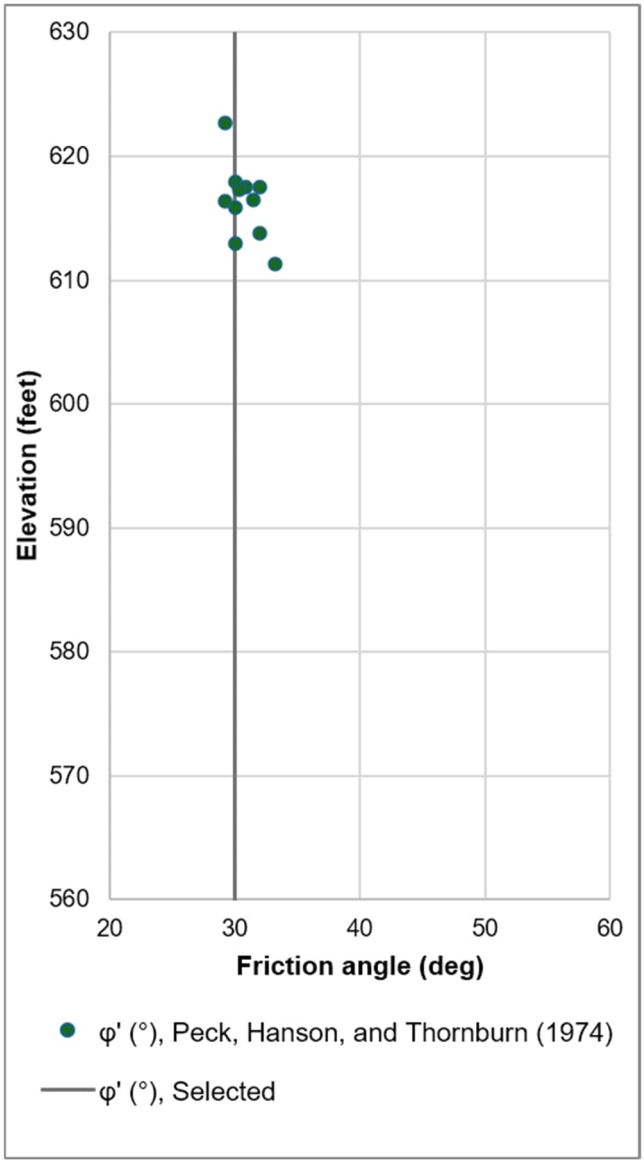
- For alluvial materials, this data indicates the presence of two distinct alluvial strata (Alluvium-1 and Alluvium-2), which were included in the data set to capture stratigraphic variability. This approach is considered appropriate given the generally consistent nature of the alluvial deposits across the region.
- For the earthfill materials, strength parameters were estimated based on the strength data from SPT values and properties of the native alluvial soils from which the embankment materials were derived, with consideration for compaction effects during placement. This approach is appropriate because the earthfill is composed predominantly of reworked site soils and dredged materials that reflect the similar strength characteristics as the underlying Alluvium-1, with minor modifications to account for placement conditions and density increases.

3.1 STRENGTH PARAMETERS – CCR-SOIL MIX (EMBANKMENT)

The following plots show the field-measured SPT values and the correlated strength data, φ' from the SPT values for the CCR-Soil Mix material from the embankment that range from 29° to 33°.

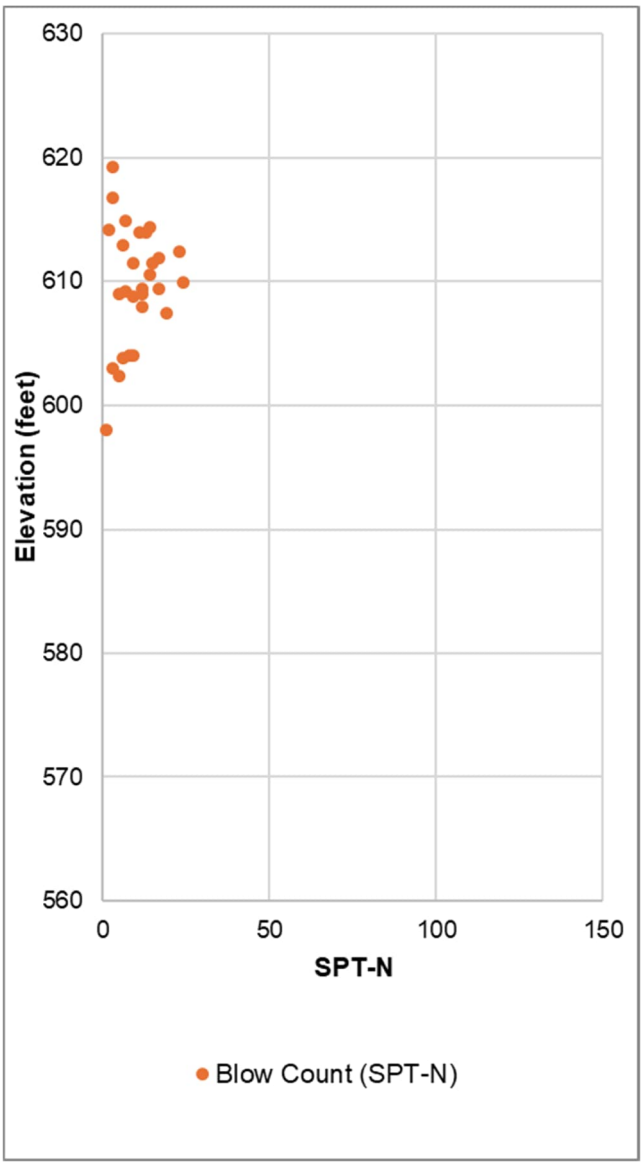
Based on an evaluation of the data, and WSP's experience at other sites, WSP selected representative strength parameters for the CCR-Soil Mix (Embankment) materials: φ' of 30° and c' of 0 psf.

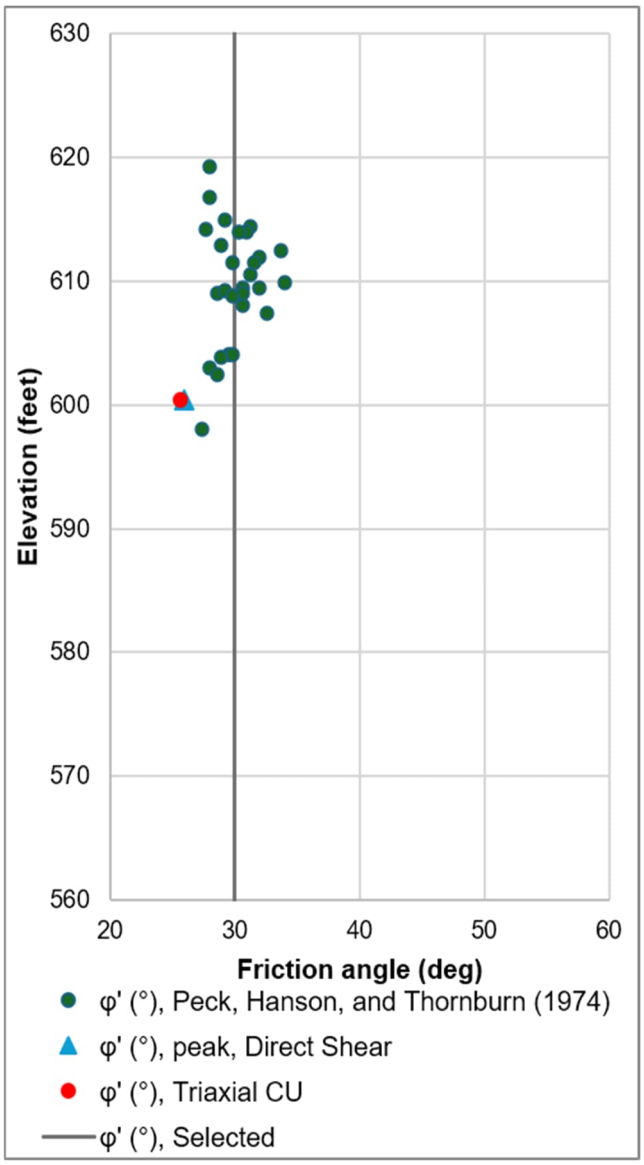




3.2 STRENGTH PARAMETERS – CCR (EMBANKMENT)

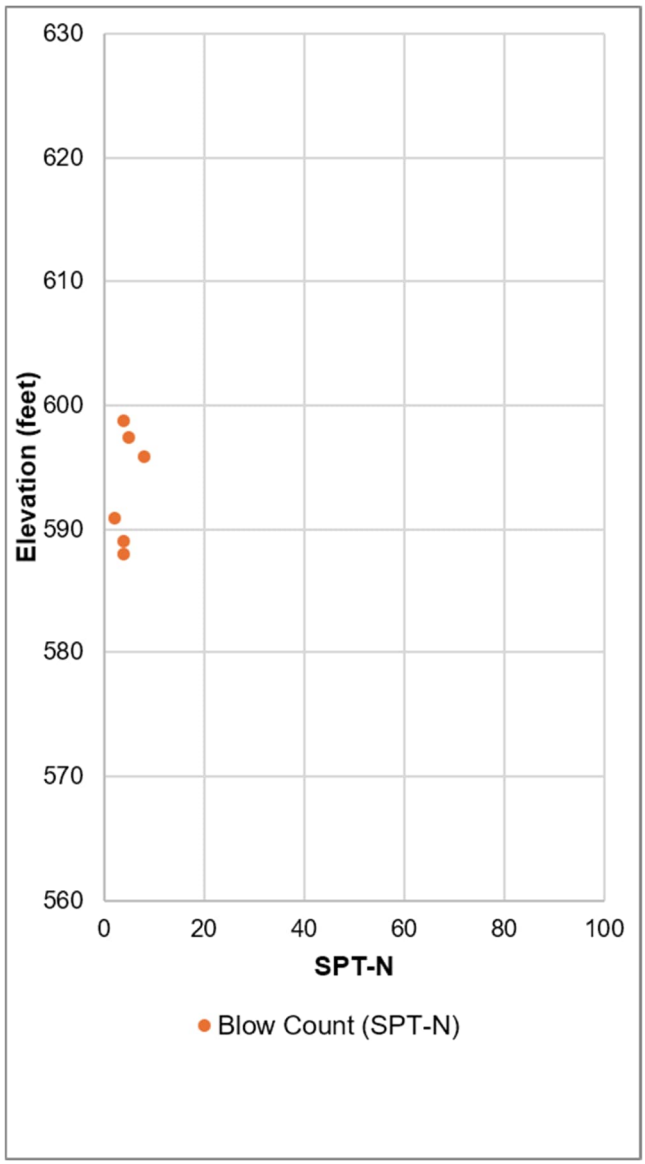
The following plots show the field-measured SPT values, laboratory strength data, and the correlated strength data from SPT values for the CCR material from the embankment. The effective shear strengths φ' ranged from 26° to 34°. Based on an evaluation of the data and WSP's experience at other sites, WSP selected representative strength parameters for the CCR (Embankment) materials: φ' of 30° and c' of 0 psf.

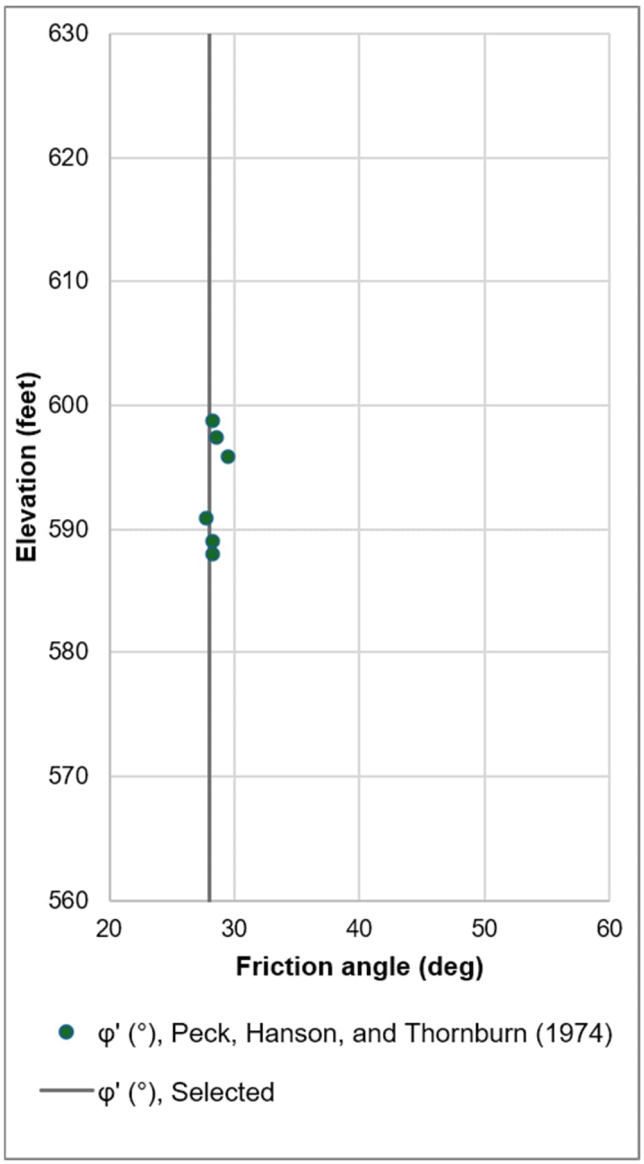




3.3 STRENGTH PARAMETERS – DREDGE SPOIL

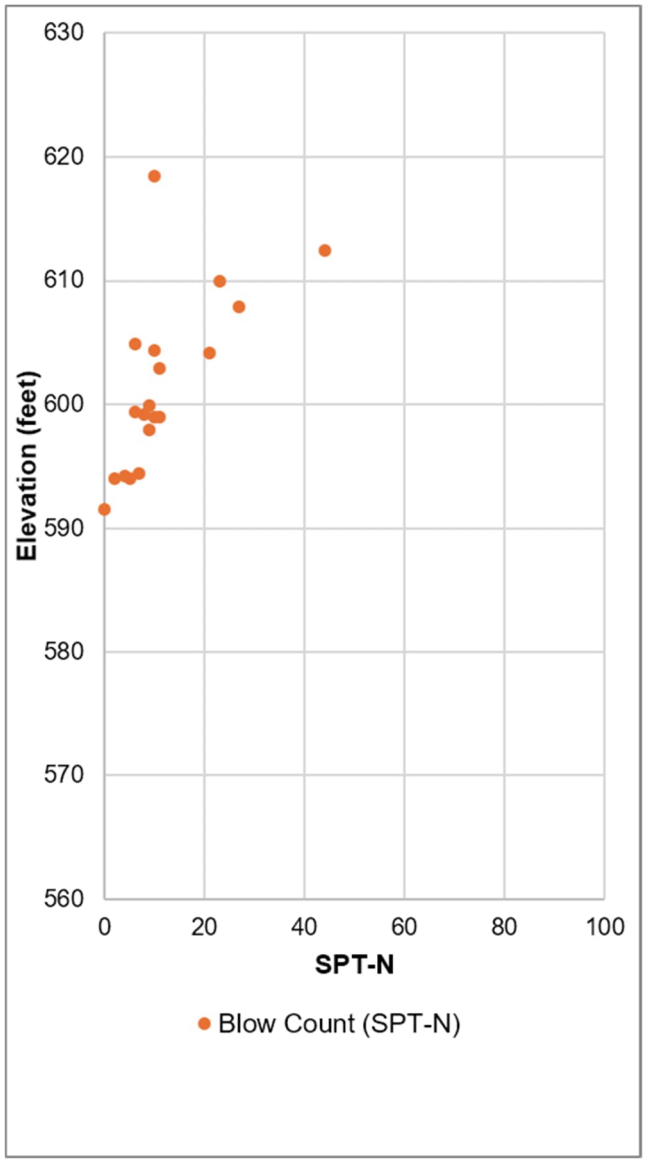
The following plots show the field measured SPT values and the correlated strength data from SPT values for the Dredge Spoil material. The effective shear strengths φ' ranged from 27.5° to 29.5°. Based on an evaluation of the data, and WSP's experience at other sites, WSP selected representative strength parameters for the Dredge Spoil materials: φ' of 28° and c' of 0 psf.

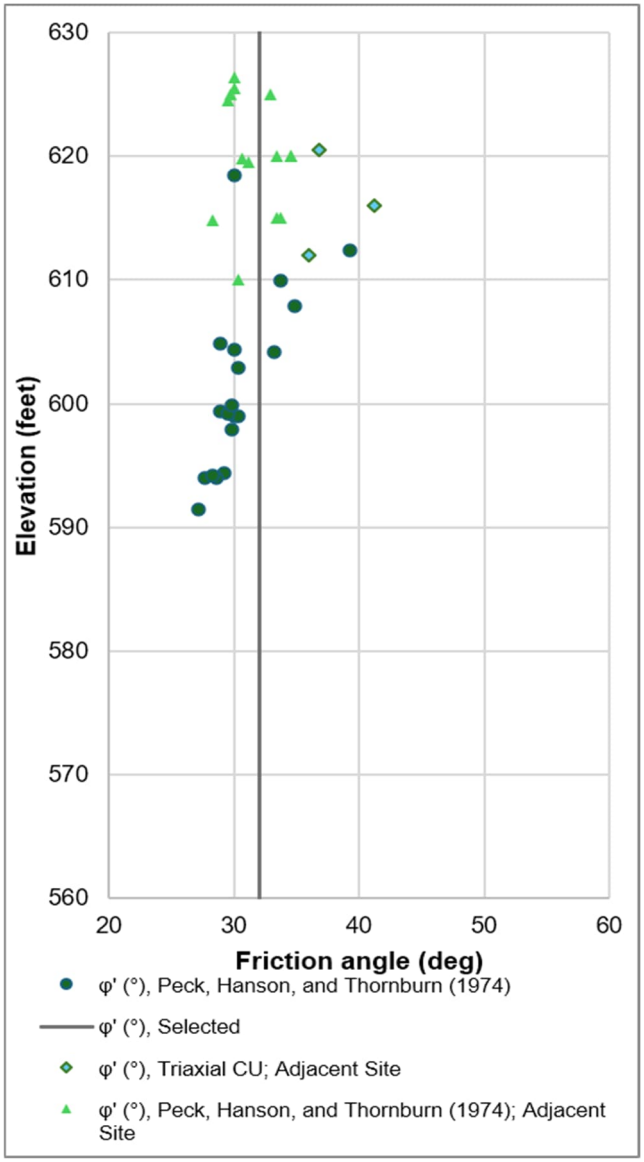




3.4 STRENGTH PARAMETERS - EARTHFILL

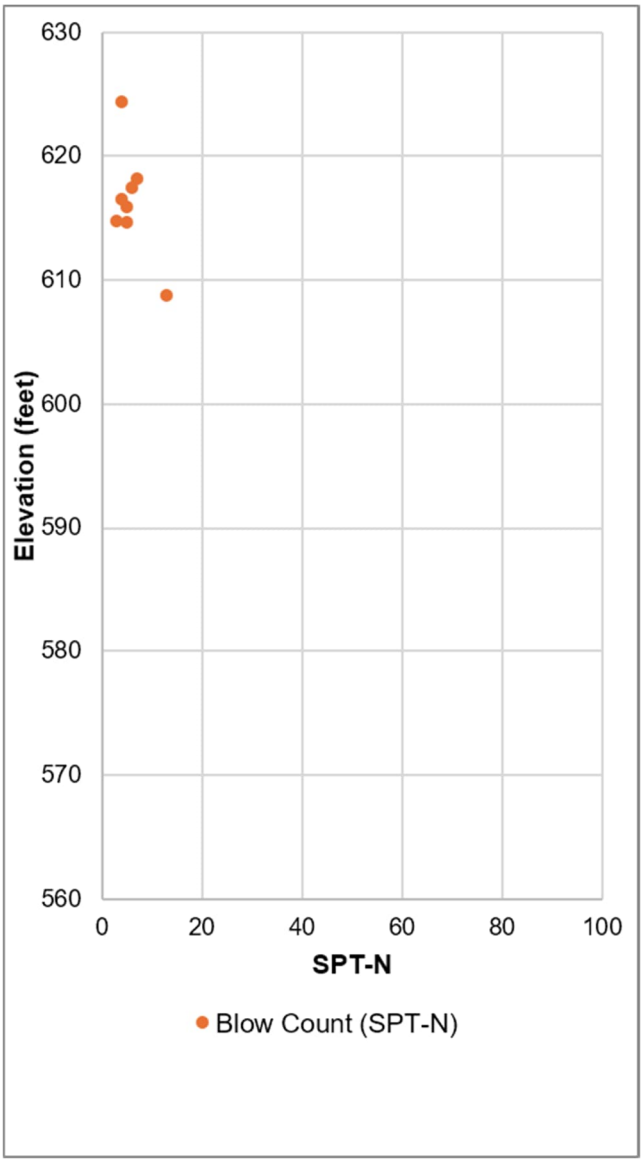
The following plots show the field-measured SPT values and the correlated strength data from SPT from Cabin Creek and adjacent sites and laboratory strength data from adjacent sites. The effective shear strengths φ' ranged from 27° to 41°. Based on an evaluation of the data, and WSP's experience at other sites, WSP selected representative strength parameters for the Earthfill materials: φ' of 32° and c' of 100 psf.

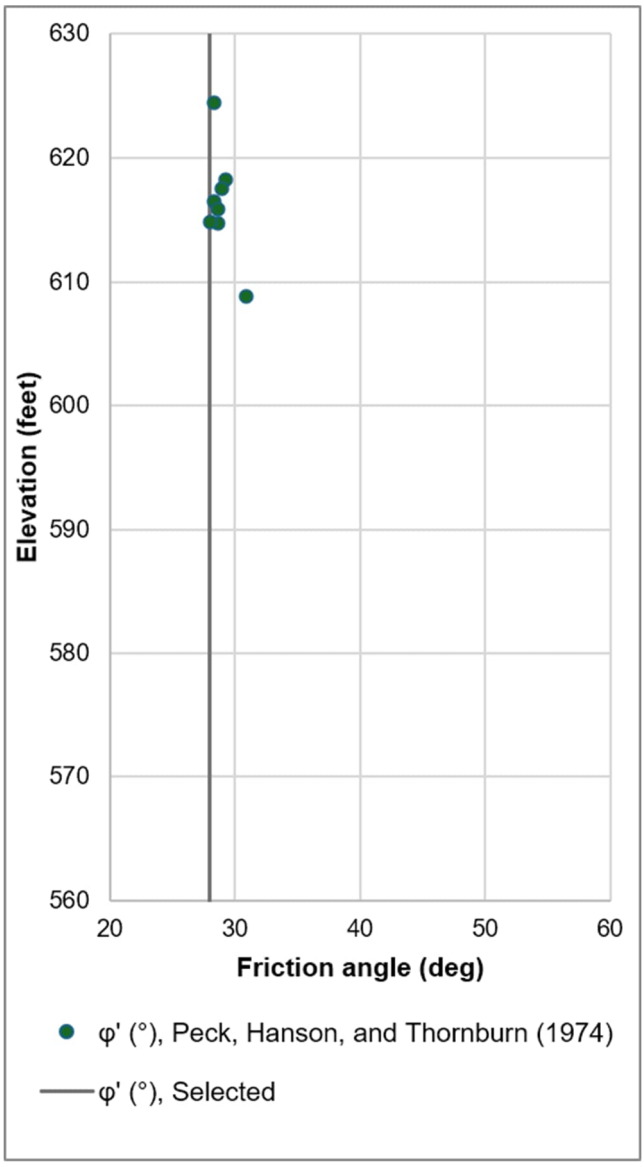




3.5 STRENGTH PARAMETERS – CCR-SOIL MIX (IMPOUNDED)

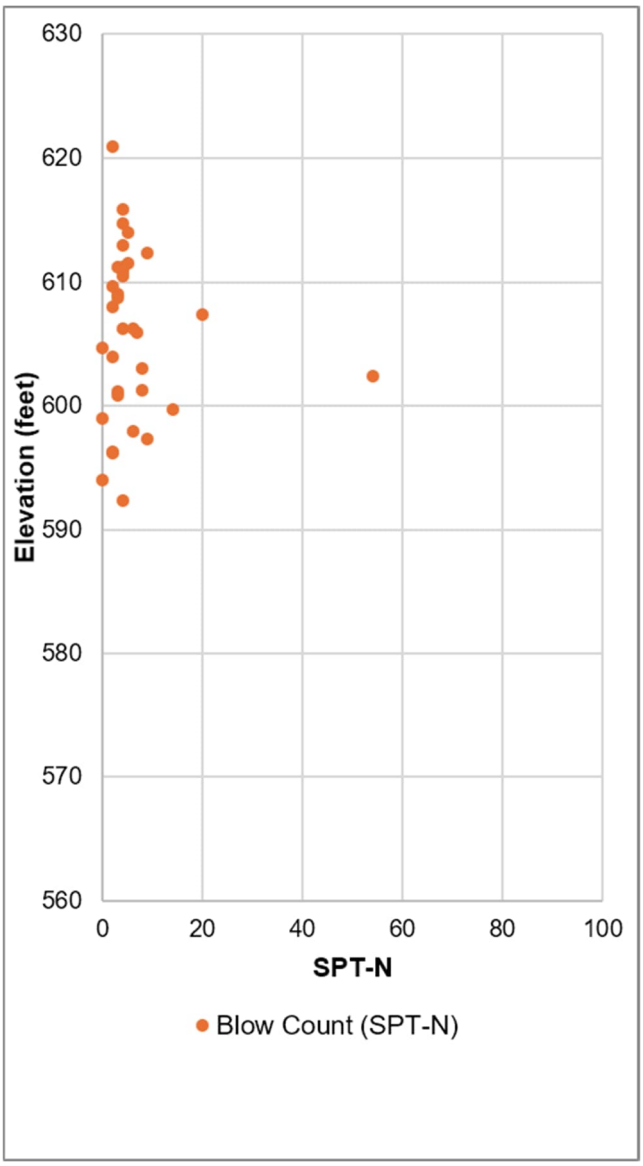
The following plots show the field-measured SPT values and the correlated strength data from SPT values for the CCR-Soil Mix material that was impounded on site. The effective shear strengths φ' ranged from 28° to 31°. Based on an evaluation of the data, and WSP's experience at other sites, WSP selected representative strength parameters for the CCR-Soil Mix (Impounded) materials: φ' of 28° and c' of 0 psf.

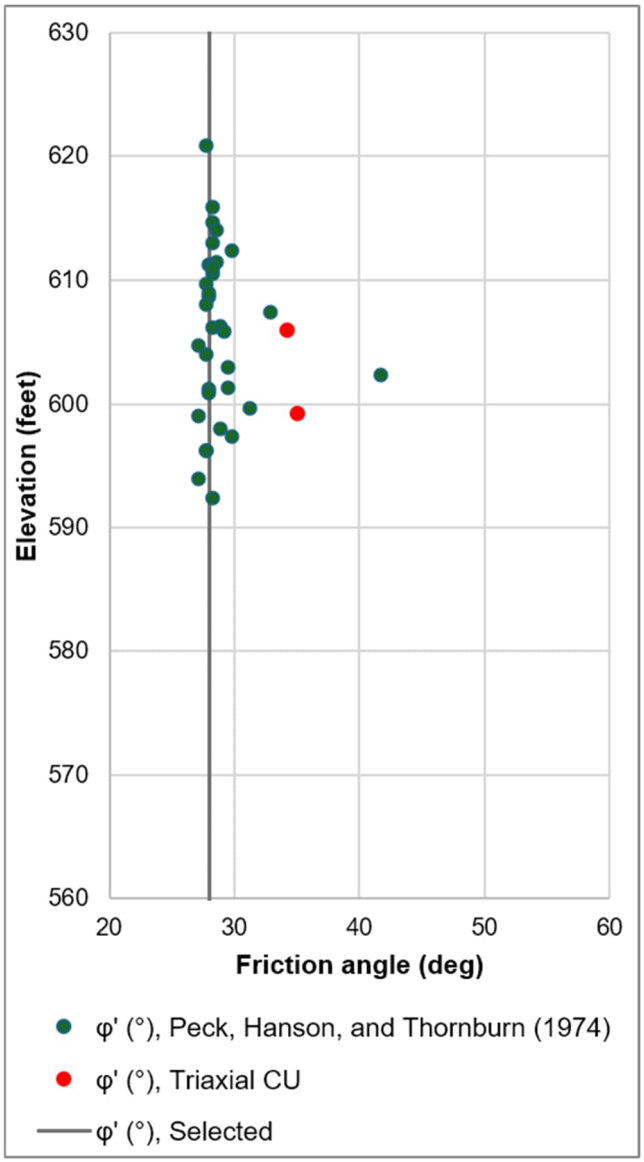




3.6 STRENGTH PARAMETERS – SLUICED CCR

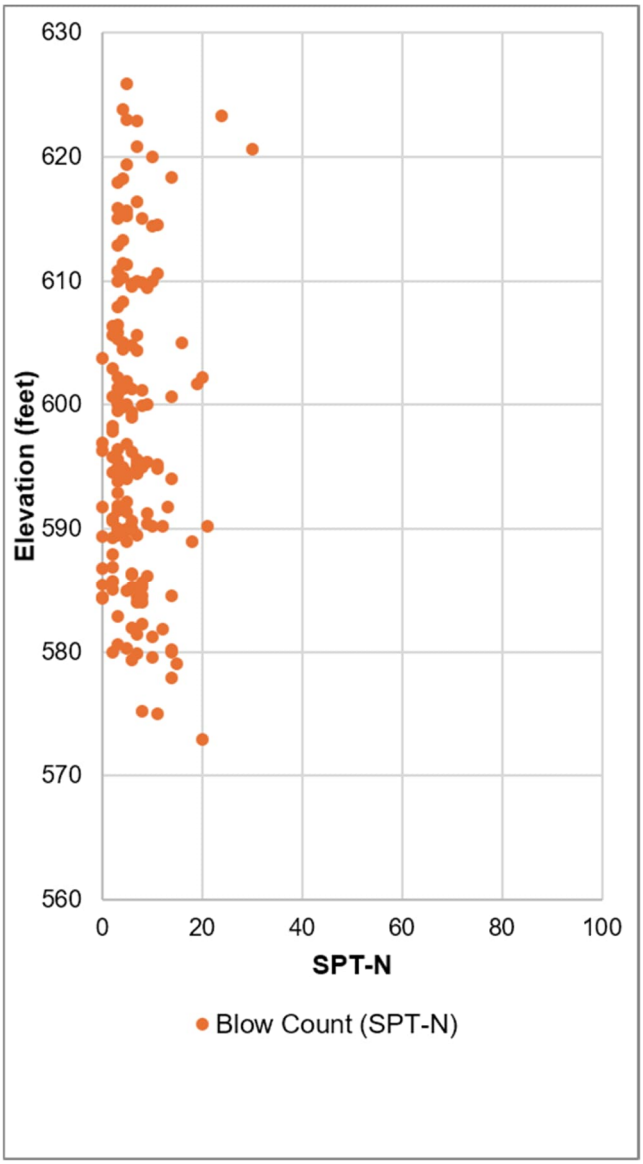
The following plots show the field-measured SPT values, laboratory strength data, and the correlated strength data from SPT values for the Sluiced CCR material. The effective shear strengths φ' ranged from 27° to 41.5°. Based on an evaluation of the data, and WSP's experience at other sites, WSP selected representative strength parameters for the Sluiced CCR materials: φ' of 28° and c' of 0 psf.

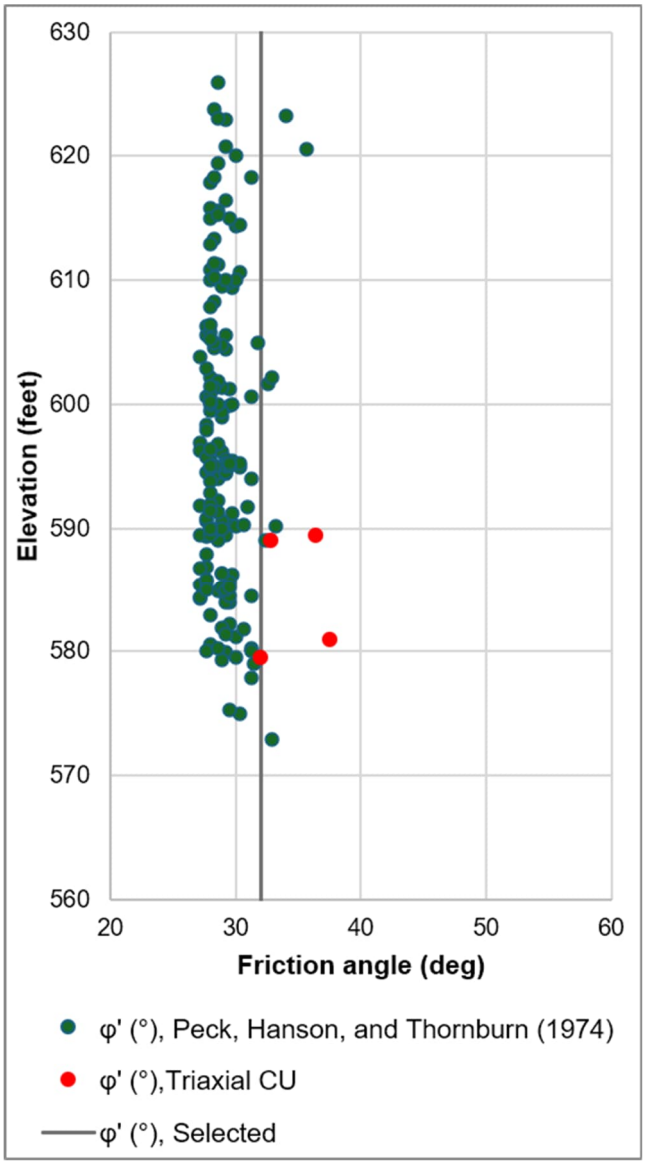


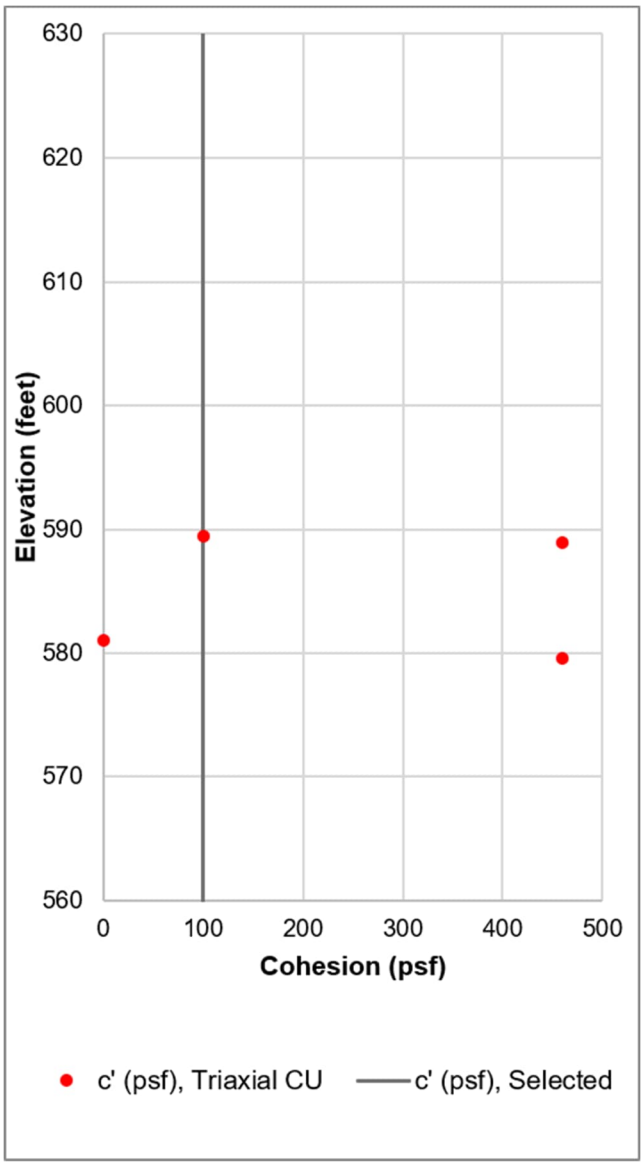


3.7 STRENGTH PARAMETERS - ALLUVIUM 1

The following plots show the field-measured SPT values, laboratory strength data and the correlated strength data from SPT values for Alluvium-1. The effective shear strengths φ' ranged from 27° to 38° and c' ranges from 0 to 460 psf.



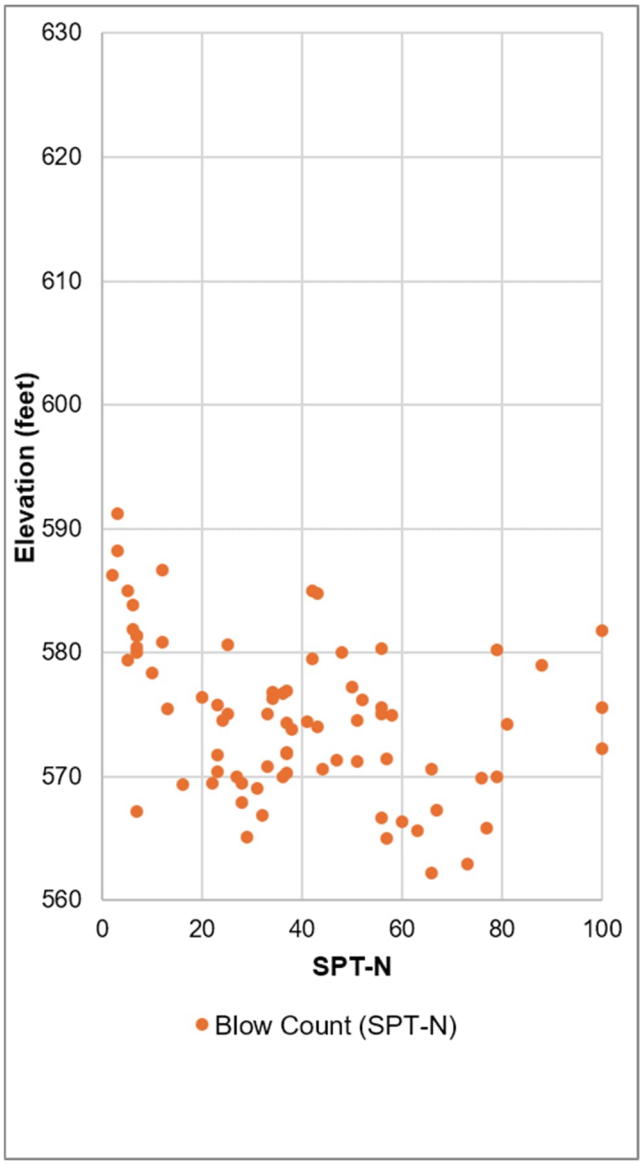


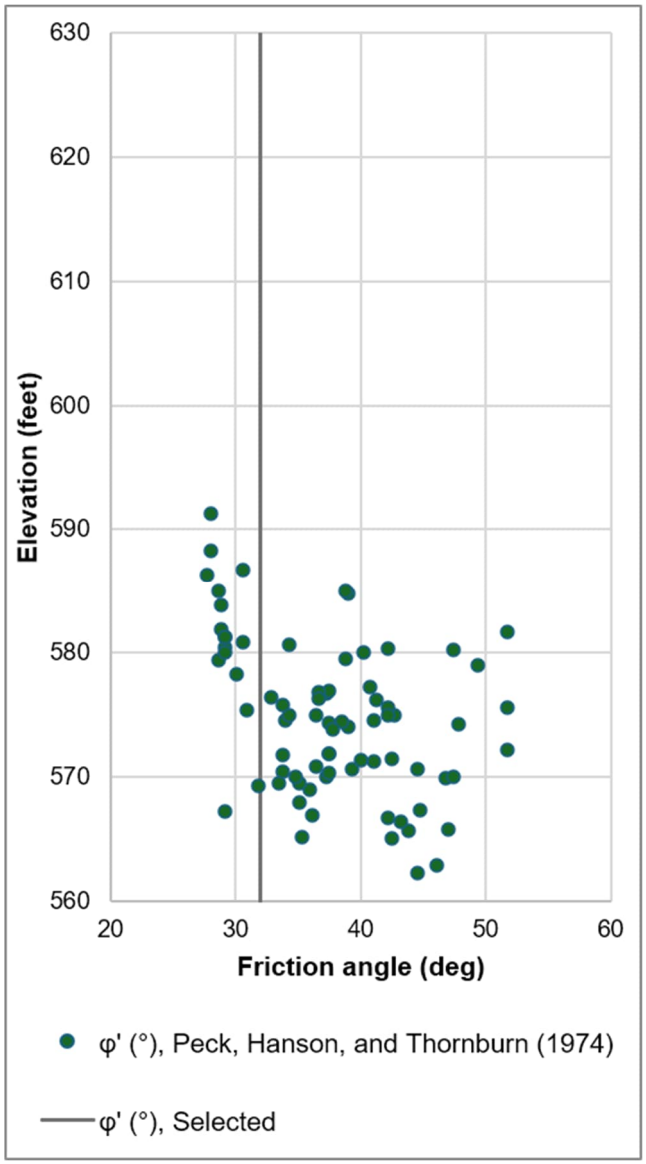


Based on an evaluation of the data and WSP's experience, WSP selected representative strength parameters for the Alluvium-1 material: φ' of 32° and c' of 100 psf. This φ' was selected based upon the shear strength observed from triaxial tests conducted on alluvial samples collected from the area. WSP considers these tests more representative than general correlations using SPT data.

3.8 STRENGTH PARAMETERS - ALLUVIUM 2

The following plots show the field-measured SPT values, and the correlated strength data from SPT values for Alluvium-2. The effective shear strengths φ' ranged from 25° to 51.5° and c' is 0 psf.





Based on an evaluation of the data and WSP's experience, WSP selected representative strength parameters for the Alluviaum-2 material: φ' of 32 degrees and c' of 0 psf.

4 SUMMARY OF STRENGTH PARAMETERS

A summary of the strength parameters based on the geotechnical information provided in the GDR and the interpretations discussed above is presented in Table 1 below.

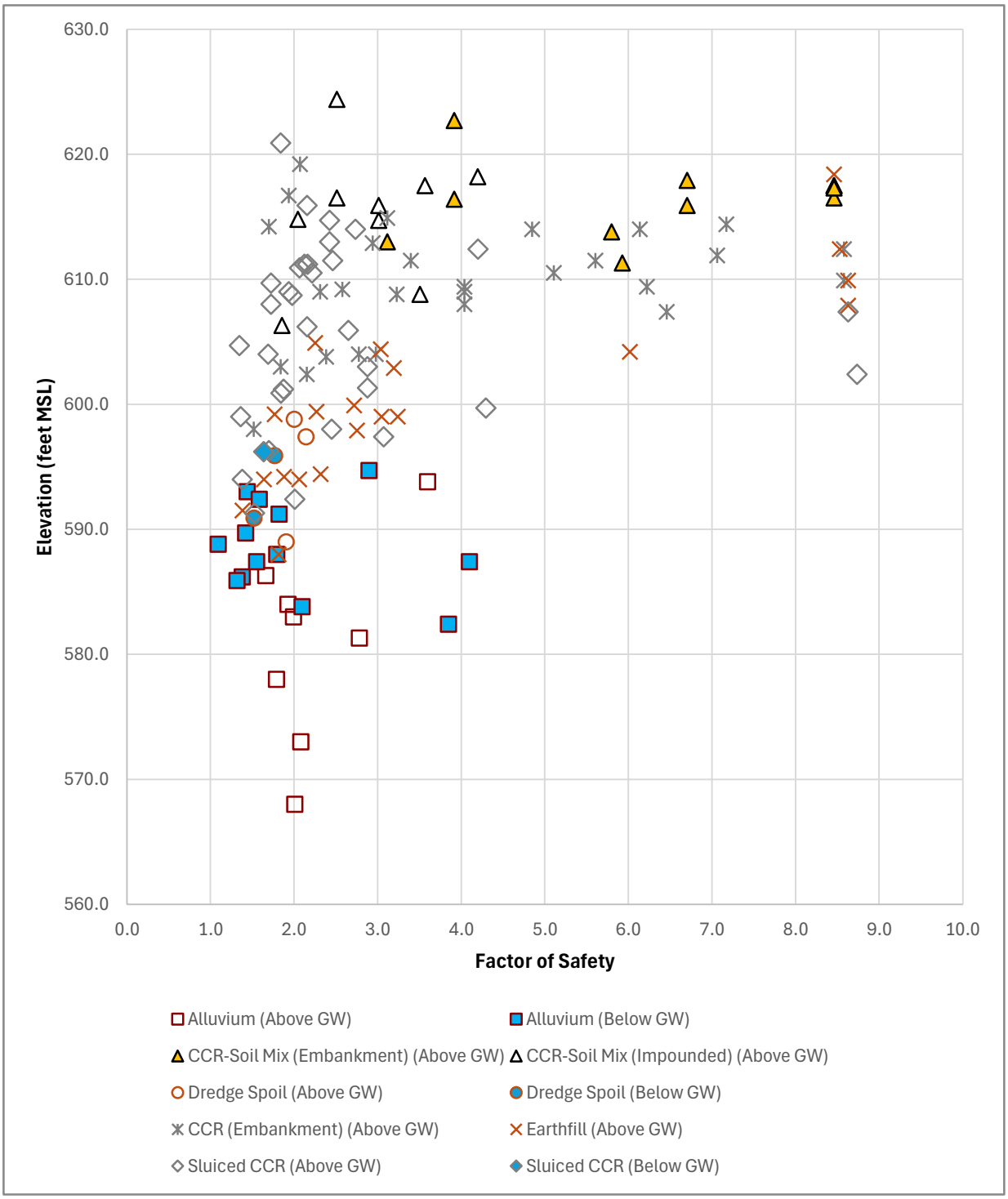
Table 1: Summary of Geotechnical Material Properties

Material	Unit Weight (pcf)	Drained Strength		Undrained Strength	
		Effective Internal Friction φ' (°)	Effective Cohesion c' (psf)	Total Internal Friction φ (°)	Total Cohesion c (psf)
Fill and Embankment Materials					
CCR-Soil Mix (Embankment)	85	30	0	-	-
CCR (Embankment)	85	30	0	-	-
Dredge Spoil	100	28	0	-	-
Earthfill	125	32	100	-	-
Coal Combustion Residuals					
CCR-Soil Mix (Impounded)	75	28	0	-	-
Sluiced CCR	75	28	0	-	-
Alluvium					
Alluvium-1	120	32	100	0	26
Alluvium-2	115	32	0	0	26

APPENDIX D

Liquefaction Assessment

APPENDIX D - LIQUEFACTION ASSESSMENT RESULTS
Cabin Creek Ash Pond, Former Cabin Creek Power Plant, Cabin Creek, 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 A-1

Groundwater Elevation=	597.2	feet
Ground Surface Elevation=	616.2	feet
Depth to Water=	19.0	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Impounded)	75	610.7	5%-35%
Sluiced CCR	75	596.5	≥35%
Alluvium	120	586.2	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
614.7	1.5	0.46	CCR-Soil Mix (Impounded)	75	112.5	113	1.00	0.099	5	14	5%-35%	4.3	0.75	16	0.167	0.299	3.01
611.2	5.0	1.52	Sluiced CCR	75	375	375	0.99	0.098	3	4	≥35%	2.4	0.75	10	0.117	0.209	2.13
608.7	7.5	2.29	Sluiced CCR	75	562.5	563	0.98	0.098	3	4	≥35%	1.9	0.75	9	0.108	0.193	1.98
606.2	10.0	3.05	Sluiced CCR	75	750	750	0.98	0.097	4	5	≥35%	1.7	0.80	10	0.117	0.209	2.16
601.2	15.0	4.57	Sluiced CCR	75	1125	1125	0.97	0.096	3	3	≥35%	1.4	0.85	9	0.100	0.180	1.87
596.2	20.0	6.10	Sluiced CCR	75	1500	1438	0.95	0.099	2	2	≥35%	1.2	0.95	7	0.090	0.162	1.64
591.2	25.0	7.62	Alluvium	120	2100	1726	0.94	0.114	5	4	≥35%	1.1	0.95	10	0.116	0.208	1.82
586.2	30.0	9.15	Alluvium	120	2700	2014	0.93	0.124	3	2	≥35%	1.0	0.95	8	0.095	0.171	1.38

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID A-2

Groundwater Elevation=	598	feet
Ground Surface Elevation=	618.0	feet
Depth to Water=	20.0	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Impounded)	75	615.5	5%-35%
Sluiced CCR	75	598.3	≥35%
Alluvium	120	588.0	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
616.5	1.5	0.46	CCR-Soil Mix (Impounded)	75	112.5	113	1.00	0.099	4	11	5%-35%	4.3	0.75	13	0.139	0.249	2.51
613.0	5.0	1.52	Sluiced CCR	75	375	375	0.99	0.098	4	6	≥35%	2.4	0.75	12	0.133	0.238	2.42
610.5	7.5	2.29	Sluiced CCR	75	562.5	563	0.98	0.098	4	5	≥35%	1.9	0.75	11	0.121	0.217	2.22
608.0	10.0	3.05	Sluiced CCR	75	750	750	0.98	0.097	2	2	≥35%	1.7	0.80	8	0.093	0.168	1.73
603.0	15.0	4.57	Sluiced CCR	75	1125	1125	0.97	0.096	8	8	≥35%	1.4	0.85	14	0.154	0.276	2.88
598.0	20.0	6.10	Sluiced CCR	75	1500	1500	0.95	0.095	6	6	≥35%	1.2	0.95	12	0.130	0.232	2.45
593.0	25.0	7.62	Alluvium	120	2100	1788	0.94	0.110	2	2	≥35%	1.1	0.95	7	0.088	0.158	1.44
588.0	30.0	9.15	Alluvium	120	2700	2076	0.93	0.120	6	5	≥35%	1.0	0.95	11	0.120	0.216	1.79

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID A-3

Groundwater Elevation=	596.2	feet
Ground Surface Elevation=	619.7	feet
Depth to Water=	23.5	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Impounded)	75	617.2	5%-35%
Sluiced CCR	75	600.5	≥35%
Alluvium	120	595.7	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
618.2	1.5	0.46	CCR-Soil Mix (Impounded)	75	112.5	113	1.00	0.099	7	19	5%-35%	4.3	0.75	21	0.232	0.416	4.20
614.7	5.0	1.52	Sluiced CCR	75	375	375	0.99	0.098	4	6	≥35%	2.4	0.75	12	0.133	0.238	2.42
611.2	8.5	2.59	Sluiced CCR	75	637.5	638	0.98	0.097	4	5	≥35%	1.8	0.75	11	0.118	0.211	2.16
609.7	10.0	3.05	Sluiced CCR	75	750	750	0.98	0.097	2	2	≥35%	1.7	0.80	8	0.093	0.168	1.73
604.7	15.0	4.57	Sluiced CCR	75	1125	1125	0.97	0.096	0	0	≥35%	1.4	0.85	5	0.072	0.129	1.35
599.7	20.0	6.10	Sluiced CCR	75	1500	1500	0.95	0.095	14	13	≥35%	1.2	0.95	21	0.227	0.407	4.30
594.7	25.0	7.62	Alluvium	120	2100	2006	0.94	0.098	10	8	≥35%	1.0	0.95	15	0.158	0.284	2.90
589.7	30.0	9.15	Alluvium	120	2700	2294	0.93	0.109	2	2	≥35%	1.0	0.95	7	0.086	0.155	1.42

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID A-4

Groundwater Elevation=	581.3	feet
Ground Surface Elevation=	616.3	feet
Depth to Water=	35.0	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Impounded)	75	607.0	5%-35%
Sluiced CCR	75	592.8	≥35%
Alluvium	120	581.3	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
614.8	1.5	0.46	CCR-Soil Mix (Impounded)	75	112.5	113	1.00	0.099	3	8	5%-35%	4.3	0.75	10	0.113	0.203	2.05
608.8	7.5	2.29	CCR-Soil Mix (Impounded)	75	562.5	563	0.98	0.098	13	16	5%-35%	1.9	0.75	18	0.191	0.343	3.51
606.3	10.0	3.05	CCR-Soil Mix (Impounded)	75	750	750	0.98	0.097	6	7	5%-35%	1.7	0.80	9	0.100	0.180	1.85
601.3	15.0	4.57	Sluiced CCR	75	1125	1125	0.97	0.096	8	8	≥35%	1.4	0.85	14	0.154	0.276	2.88
596.3	20.0	6.10	Sluiced CCR	75	1500	1500	0.95	0.095	2	2	≥35%	1.2	0.95	7	0.090	0.161	1.70
591.3	25.0	7.62	Sluiced CCR	75	1875	1875	0.94	0.094	1	1	≥35%	1.1	0.95	6	0.080	0.143	1.53
586.3	30.0	9.15	Alluvium	120	2475	2475	0.93	0.092	2	1	≥35%	0.9	0.95	7	0.086	0.154	1.66
581.3	35.0	10.67	Alluvium	120	3675	3675	0.89	0.088	10	6	≥35%	0.8	1.00	13	0.137	0.246	2.78

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., Ii, 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)
2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID	B-1			a_{max} (USGS Hazard tool)				Corrections to SPT		
Groundwater Elevation=	584	feet		10% in 50 yrs	0.054	Lat	38.197944	C_N=	see each cell	*Overburden*
Ground Surface Elevation=	619.0	feet		5% in 50 yrs	0.085	Long	-81.486306	C_E=	0.8	*Energy Ratio*
Depth to Water=	35.0	feet - below ground surface		2% in 50 yrs	0.153			C_B=	1.05	*Borehole Diameter*
M_w=	5.97	*Moment magnitude of earthquake at the Site*						C_R=	see each cell	*Rod Length*
MSF=	1.79	*Magnitude Scaling Factor*						C_S=	1	*Sampling Method*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*						Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Impounded)	75	616.5	5%-35%
Sluiced CCR	75	594.5	≥35%
Dredge Spoil	100	587.3	≥35%
Alluvium	120	584.0	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
617.5	1.5	0.46	CCR-Soil Mix (Impounded)	75	112.5	113	1.00	0.099	6	16	5%-35%	4.3	0.75	18	0.197	0.353	3.57
614.0	5.0	1.52	Sluiced CCR	75	375	375	0.99	0.098	5	7	≥35%	2.4	0.75	14	0.150	0.269	2.74
611.5	7.5	2.29	Sluiced CCR	75	562.5	563	0.98	0.098	5	6	≥35%	1.9	0.75	12	0.134	0.241	2.46
609.0	10.0	3.05	Sluiced CCR	75	750	750	0.98	0.097	3	3	≥35%	1.7	0.80	9	0.105	0.188	1.94
604.0	15.0	4.57	Sluiced CCR	75	1125	1125	0.97	0.096	2	2	≥35%	1.4	0.85	7	0.091	0.162	1.69
599.0	20.0	6.10	Sluiced CCR	75	1500	1500	0.95	0.095	0	0	≥35%	1.2	0.95	5	0.072	0.129	1.36
594.0	25.0	7.62	Sluiced CCR	75	1875	1875	0.94	0.094	0	0	≥35%	1.1	0.95	5	0.072	0.129	1.38
589.0	30.0	9.15	Dredge Spoil	100	2875	2875	0.93	0.092	4	3	≥35%	0.9	0.95	8	0.098	0.176	1.91
584.0	35.0	10.67	Alluvium	120	4075	4075	0.89	0.088	4	2	≥35%	0.7	1.00	8	0.095	0.171	1.93

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID	B-2			a_{max} (USGS Hazard tool)				Corrections to SPT		
Groundwater Elevation=	597.4	feet		10% in 50 yrs	0.054	Lat	38.197944	C_N=	see each cell	*Overburden*
Ground Surface Elevation=	625.9	feet		5% in 50 yrs	0.085	Long	-81.486306	C_E=	0.8	*Energy Ratio*
Depth to Water=	28.5	feet - below ground surface		2% in 50 yrs	0.153			C_B=	1.05	*Borehole Diameter*
M_w=	5.97	*Moment magnitude of earthquake at the Site*						C_R=	see each cell	*Rod Length*
MSF=	1.79	*Magnitude Scaling Factor*						C_S=	1	*Sampling Method*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*						Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Impounded)	75	623.4	5%-35%
Sluiced CCR	75	597.4	≥35%
Dredge Spoil	100	589.2	≥35%
Alluvium	120	585.9	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
624.4	1.5	0.46	CCR-Soil Mix (Impounded)	75	112.5	113	1.00	0.099	4	11	5%-35%	4.3	0.75	13	0.139	0.249	2.51
620.9	5.0	1.52	Sluiced CCR	75	375	375	0.99	0.098	2	3	≥35%	2.4	0.75	9	0.101	0.181	1.84
615.9	10.0	3.05	Sluiced CCR	75	750	750	0.98	0.097	4	5	≥35%	1.7	0.80	10	0.117	0.209	2.16
610.9	15.0	4.57	Sluiced CCR	75	1125	1125	0.97	0.096	4	4	≥35%	1.4	0.85	10	0.110	0.198	2.06
605.9	20.0	6.10	Sluiced CCR	75	1500	1500	0.95	0.095	7	7	≥35%	1.2	0.95	13	0.140	0.251	2.65
600.9	25.0	7.62	Sluiced CCR	75	1875	1875	0.94	0.094	3	3	≥35%	1.1	0.95	8	0.096	0.173	1.84
595.9	30.0	9.15	Dredge Spoil	100	2375	2281	0.93	0.096	8	6	5%-35%	1.0	0.95	8	0.095	0.170	1.77
590.9	35.0	10.67	Dredge Spoil	100	3375	2969	0.89	0.100	2	1	≥35%	0.8	1.00	7	0.085	0.153	1.52
585.9	40.0	12.20	Alluvium	120	5175	4457	0.85	0.098	0	0	≥35%	0.7	1.00	5	0.072	0.129	1.32

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID B-3

Groundwater Elevation=	588.9	feet
Ground Surface Elevation=	617.4	feet
Depth to Water=	28.5	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Impounded)	75	615.9	5%-35%
Sluiced CCR	75	590.7	≥35%
Alluvium	120	580.4	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
615.9	1.5	0.46	CCR-Soil Mix (Impounded)	75	112.5	113	1.00	0.099	5	14	5%-35%	4.3	0.75	16	0.167	0.299	3.01
612.4	5.0	1.52	Sluiced CCR	75	375	375	0.99	0.098	9	13	≥35%	2.4	0.75	21	0.230	0.413	4.20
607.4	10.0	3.05	Sluiced CCR	75	750	750	0.98	0.097	20	23	≥35%	1.7	0.80	30	0.468	0.838	8.63
602.4	15.0	4.57	Sluiced CCR	75	1125	1125	0.97	0.096	54	53	≥35%	1.4	0.85	30	0.468	0.838	8.74
597.4	20.0	6.10	Sluiced CCR	75	1500	1500	0.95	0.095	9	9	≥35%	1.2	0.95	15	0.162	0.291	3.07
592.4	25.0	7.62	Sluiced CCR	75	1875	1875	0.94	0.094	4	3	≥35%	1.1	0.95	9	0.105	0.188	2.01
587.4	30.0	9.15	Alluvium	120	2475	2381	0.93	0.096	17	13	≥35%	0.9	0.95	20	0.220	0.394	4.10
582.4	35.0	10.67	Alluvium	120	3075	2669	0.89	0.102	17	13	≥35%	0.9	1.00	20	0.219	0.392	3.85

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID E-1

Groundwater Elevation=	568	feet
Ground Surface Elevation=	618.0	feet
Depth to Water=	50.0	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Embankment)	85	614.0	5%-35%
CCR (Embankment)	85	611.2	≥35%
Earth Fill	125	586.3	≥35%
Alluvium	120	578.5	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
616.5	1.5	0.46	CCR-Soil Mix (Embankment)	85	127.5	128	1.00	0.099	15	38	5%-35%	4.1	0.75	30	0.468	0.838	8.46
613.0	5.0	1.52	CCR-Soil Mix (Embankment)	85	425	425	0.99	0.098	10	14	5%-35%	2.2	0.75	16	0.171	0.306	3.12
610.5	7.5	2.29	CCR (Embankment)	85	637.5	638	0.98	0.098	14	16	≥35%	1.8	0.75	24	0.278	0.499	5.11
608.0	10.0	3.05	CCR (Embankment)	85	850	850	0.98	0.097	12	13	≥35%	1.6	0.80	20	0.219	0.392	4.04
603.0	15.0	4.57	CCR (Embankment)	85	1275	1275	0.97	0.096	3	3	≥35%	1.3	0.85	8	0.099	0.177	1.84
598.0	20.0	6.10	CCR (Embankment)	85	1700	1700	0.95	0.095	1	1	≥35%	1.1	0.95	6	0.080	0.144	1.52
591.5	26.5	8.08	Earth Fill	125	3762.5	3763	0.94	0.093	0	0	≥35%	0.7	0.95	5	0.072	0.129	1.38
588.0	30.0	9.15	Earth Fill	125	4200	4200	0.93	0.092	4	2	≥35%	0.7	0.95	8	0.094	0.168	1.81
583.0	35.0	10.67	Alluvium	120	7200	7200	0.89	0.088	6	3	≥35%	0.5	1.00	8	0.098	0.176	1.99
578.0	40.0	12.20	Alluvium	120	7800	7800	0.85	0.084	3	1	≥35%	0.5	1.00	7	0.084	0.151	1.79
573.0	45.0	13.72	Alluvium	120	10800	10800	0.81	0.080	6	2	≥35%	0.4	1.00	8	0.093	0.167	2.08
568.0	50.0	15.24	Alluvium	120	11400	11400	0.77	0.076	4	1	≥35%	0.4	1.00	7	0.086	0.153	2.01

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID E-2

Groundwater Elevation=	594.4	feet
Ground Surface Elevation=	619.4	feet
Depth to Water=	25.0	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Embankment)	85	616.4	5%-35%
CCR (Embankment)	85	605.2	≥35%
Earth Fill	125	594.4	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
617.9	1.5	0.46	CCR-Soil Mix (Embankment)	85	127.5	128	1.00	0.099	10	26	5%-35%	4.1	0.75	28	0.371	0.664	6.70
614.4	5.0	1.52	CCR (Embankment)	85	425	425	0.99	0.098	14	20	≥35%	2.2	0.75	29	0.393	0.705	7.17
611.9	7.5	2.29	CCR (Embankment)	85	637.5	638	0.98	0.098	17	20	≥35%	1.8	0.75	28	0.385	0.690	7.06
609.4	10.0	3.05	CCR (Embankment)	85	850	850	0.98	0.097	12	13	≥35%	1.6	0.80	20	0.219	0.392	4.04
604.4	15.0	4.57	Earth Fill	125	1475	1475	0.97	0.096	10	9	≥35%	1.2	0.85	15	0.163	0.292	3.04
599.4	20.0	6.10	Earth Fill	125	2100	2100	0.95	0.095	6	5	≥35%	1.0	0.95	11	0.120	0.215	2.27
594.4	25.0	7.62	Earth Fill	125	2725	2725	0.94	0.094	7	5	≥35%	0.9	0.95	11	0.121	0.217	2.32

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID E-3

Groundwater Elevation=	597.9	feet
Ground Surface Elevation=	617.9	feet
Depth to Water=	20.0	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Embankment)	85	613.2	5%-35%
CCR (Embankment)	85	609.4	≥35%
Earth Fill	125	597.9	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
616.4	1.5	0.46	CCR-Soil Mix (Embankment)	85	127.5	128	1.00	0.099	7	18	5%-35%	4.1	0.75	20	0.216	0.388	3.92
612.9	5.0	1.52	CCR (Embankment)	85	425	425	0.99	0.098	6	8	≥35%	2.2	0.75	15	0.161	0.289	2.94
609.4	8.5	2.59	CCR (Embankment)	85	722.5	723	0.98	0.097	17	18	≥35%	1.7	0.75	27	0.338	0.606	6.22
607.9	10.0	3.05	Earth Fill	125	910	910	0.98	0.097	27	28	≥35%	1.5	0.80	30	0.468	0.838	8.63
602.9	15.0	4.57	Earth Fill	125	1535	1535	0.97	0.096	11	9	≥35%	1.2	0.85	16	0.171	0.306	3.19
597.9	20.0	6.10	Earth Fill	125	2160	2160	0.95	0.095	9	7	≥35%	1.0	0.95	14	0.146	0.261	2.75

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID E-4

Groundwater Elevation=	594	feet
Ground Surface Elevation=	619.0	feet
Depth to Water=	25.0	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Embankment)	85	616.8	5%-35%
CCR (Embankment)	85	604.6	≥35%
Earth Fill	125	594.0	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
617.5	1.5	0.46	CCR-Soil Mix (Embankment)	85	127.5	128	1.00	0.099	17	44	5%-35%	4.1	0.75	30	0.468	0.838	8.46
614.0	5.0	1.52	CCR (Embankment)	85	425	425	0.99	0.098	13	18	≥35%	2.2	0.75	27	0.336	0.603	6.14
611.5	7.5	2.29	CCR (Embankment)	85	637.5	638	0.98	0.098	15	17	≥35%	1.8	0.75	26	0.306	0.548	5.61
609.0	10.0	3.05	CCR (Embankment)	85	850	850	0.98	0.097	5	5	≥35%	1.6	0.80	11	0.125	0.225	2.31
604.0	15.0	4.57	CCR (Embankment)	85	1275	1275	0.97	0.096	8	7	≥35%	1.3	0.85	14	0.149	0.266	2.77
599.0	20.0	6.10	Earth Fill	125	1900	1900	0.95	0.095	10	8	≥35%	1.1	0.95	15	0.161	0.289	3.05
594.0	25.0	7.62	Earth Fill	125	2525	2525	0.94	0.094	5	4	≥35%	0.9	0.95	9	0.108	0.193	2.06

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID E-5

Groundwater Elevation=	594	feet
Ground Surface Elevation=	619.0	feet
Depth to Water=	25.0	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Embankment)	85	617.5	5%-35%
CCR (Embankment)	85	604.8	≥35%
Earth Fill	125	594.0	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
617.5	1.5	0.46	CCR-Soil Mix (Embankment)	85	127.5	128	1.00	0.099	13	33	5%-35%	4.1	0.75	30	0.468	0.838	8.46
614.0	5.0	1.52	CCR (Embankment)	85	425	425	0.99	0.098	11	15	≥35%	2.2	0.75	24	0.266	0.477	4.85
611.5	7.5	2.29	CCR (Embankment)	85	637.5	638	0.98	0.098	9	10	≥35%	1.8	0.75	17	0.185	0.332	3.40
609.0	10.0	3.05	CCR (Embankment)	85	850	850	0.98	0.097	12	13	≥35%	1.6	0.80	20	0.219	0.392	4.04
604.0	15.0	4.57	CCR (Embankment)	85	1275	1275	0.97	0.096	9	8	≥35%	1.3	0.85	15	0.159	0.286	2.98
599.0	20.0	6.10	Earth Fill	125	1900	1900	0.95	0.095	11	9	≥35%	1.1	0.95	16	0.171	0.307	3.24
594.0	25.0	7.62	Earth Fill	125	2525	2525	0.94	0.094	2	1	≥35%	0.9	0.95	7	0.086	0.154	1.64

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID E-6

Groundwater Elevation=	594.2	feet
Ground Surface Elevation=	624.2	feet
Depth to Water=	30.0	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Embankment)	85	624.0	5%-35%
CCR (Embankment)	85	615.3	≥35%
Earth Fill	125	594.2	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
622.7	1.5	0.46	CCR-Soil Mix (Embankment)	85	127.5	128	1.00	0.099	7	18	5%-35%	4.1	0.75	20	0.216	0.388	3.92
619.2	5.0	1.52	CCR (Embankment)	85	425	425	0.99	0.098	3	4	≥35%	2.2	0.75	10	0.114	0.204	2.07
616.7	7.5	2.29	CCR (Embankment)	85	637.5	638	0.98	0.098	3	3	≥35%	1.8	0.75	9	0.106	0.189	1.94
614.2	10.0	3.05	CCR (Embankment)	85	850	850	0.98	0.097	2	2	≥35%	1.6	0.80	8	0.092	0.165	1.70
609.2	15.0	4.57	CCR (Embankment)	85	1275	1275	0.97	0.096	7	6	≥35%	1.3	0.85	13	0.138	0.247	2.58
604.2	20.0	6.10	Earth Fill	125	1900	1900	0.95	0.095	21	18	≥35%	1.1	0.95	26	0.318	0.571	6.02
599.2	25.0	7.62	Earth Fill	125	2525	2525	0.94	0.094	8	6	5%-35%	0.9	0.95	8	0.092	0.166	1.77
594.2	30.0	9.15	Earth Fill	125	3150	3150	0.93	0.092	4	3	≥35%	0.8	0.95	8	0.097	0.174	1.88

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID E-7

Groundwater Elevation=	599.9	feet
Ground Surface Elevation=	619.9	feet
Depth to Water=	20.0	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
Earth Fill	125	618.7	≥35%
CCR (Embankment)	85	612.4	≥35%
Earth Fill	125	618.7	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
618.4	1.5	0.46	Earth Fill	125	187.5	188	1.00	0.099	10	21	≥35%	3.4	0.75	30	0.468	0.838	8.46
614.9	5.0	1.52	CCR (Embankment)	85	485	485	0.99	0.098	7	9	≥35%	2.1	0.75	16	0.171	0.306	3.12
612.4	7.5	2.29	CCR (Embankment)	85	697.5	698	0.98	0.098	23	25	≥35%	1.7	0.75	30	0.468	0.838	8.58
609.9	10.0	3.05	Earth Fill	125	1010	1010	0.98	0.097	23	22	≥35%	1.4	0.80	30	0.468	0.838	8.63
604.9	15.0	4.57	Earth Fill	125	1635	1635	0.97	0.096	6	5	≥35%	1.1	0.85	11	0.121	0.216	2.25
599.9	20.0	6.10	Earth Fill	125	2260	2260	0.95	0.095	9	7	≥35%	1.0	0.95	13	0.144	0.258	2.72

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID E-8

Groundwater Elevation=	589.8	feet
Ground Surface Elevation=	618.8	feet
Depth to Water=	29.0	feet - below ground surface
M_w=	5.97	*Moment magnitude of earthquake at the Site*
MSF=	1.79	*Magnitude Scaling Factor*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*

a_{max} (USGS Hazard tool)

10% in 50 yrs	0.054	Lat	38.197944
5% in 50 yrs	0.085	Long	-81.486306
2% in 50 yrs	0.153		

Corrections to SPT

C_N=	see each cell	*Overburden*
C_E=	0.8	*Energy Ratio*
C_B=	1.05	*Borehole Diameter*
C_R=	see each cell	*Rod Length*
C_S=	1	*Sampling Method*
Pa=	2116.2	*Atmospheric Pressure, psf*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Embankment)	85	614.8	5%-35%
CCR (Embankment)	85	604.3	≥35%
Dredge Spoil	100	597.1	≥35%
Alluvium	120	583.8	≥35%

Groundwater (Approximate)
(GW is assumed to be the bottom of the borehole if it is not encountered during drilling)

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 (5.97) at the Site, CRR	Factor of Safety, FS
617.3	1.5	0.46	CCR-Soil Mix (Embankment)	85	127.5	128	1.00	0.099	11	28	5%-35%	4.1	0.75	30	0.468	0.838	8.46
613.8	5.0	1.52	CCR-Soil Mix (Embankment)	85	425	425	0.99	0.098	17	24	5%-35%	2.2	0.75	26	0.318	0.570	5.80
611.3	7.5	2.29	CCR-Soil Mix (Embankment)	85	637.5	638	0.98	0.098	21	24	5%-35%	1.8	0.75	26	0.323	0.579	5.93
608.8	10.0	3.05	CCR (Embankment)	85	850	850	0.98	0.097	9	10	≥35%	1.6	0.80	16	0.175	0.314	3.23
603.8	15.0	4.57	CCR (Embankment)	85	1275	1275	0.97	0.096	6	6	≥35%	1.3	0.85	12	0.128	0.229	2.39
598.8	20.0	6.10	Dredge Spoil	100	1775	1775	0.95	0.095	4	3	≥35%	1.1	0.95	9	0.106	0.190	2.00
593.8	25.0	7.62	Alluvium	120	2375	2375	0.94	0.094	14	11	≥35%	0.9	0.95	18	0.188	0.337	3.60
588.8	30.0	9.15	Alluvium	120	3575	3513	0.93	0.094	2	1	5%-35%	0.8	0.95	3	0.057	0.103	1.09
583.8	35.0	10.67	Alluvium	120	4775	4401	0.89	0.096	7	4	≥35%	0.7	1.00	10	0.112	0.201	2.10

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., Ii, 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)

2. USGS Earthquake Hazard Toolbox (usgs.gov)

Prepared by: MN Date: 12/04/2025
Checked by: NG Date: 12/05/2025

LIQUEFACTION RESISTANCE CALCULATION

Borehole ID E-9			a_{max} (USGS Hazard tool)			Corrections to SPT		
Groundwater Elevation=	593.9	feet	10% in 50 yrs	0.054 Lat	38.197944	C_N=	see each cell	*Overburden*
Ground Surface Elevation=	617.4	feet	5% in 50 yrs	0.085 Long	-81.486306	C_E=	0.8	*Energy Ratio*
Depth to Water=	23.5	feet - below ground surface	2% in 50 yrs	0.153		C_B=	1.05	*Borehole Diam*
M_w=	5.97	*Moment magnitude of earthquake at the Site*				C_R=	see each cell	*Rod Length*
MSF=	1.79	*Magnitude Scaling Factor*				C_S=	1	*Sampling Meth*
a_{max}=	0.153	*peak horizontal acceleration at the ground surface based on USGS Unified Hazard Tool*				Pa=	2116.2	*Atmospheric Pr*

Material Type	Unit Weight (pcf)	Bottom Elevation (ft-msl)	Avg % Fines
CCR-Soil Mix (Embankment)	85	614.9	5%-35%
Earth Fill	125	610.9	≥35%
CCR (Embankment)	85	599.7	≥35%
Dredge Spoil	100	595.7	≥35%
Alluvium	120	587.4	≥35%

Groundwater (Approximate)
(GW is assumed to be the bot
if it is not encountered durin

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, $CRR_{7.5}$
615.9	1.5	0.46	CCR-Soil Mix (Embankment)	85	127.5	128	1.00	0.099	10	26	5%-35%	4.1	0.75	28	0.371
612.4	5.0	1.52	Earth Fill	125	565	565	0.99	0.098	44	54	≥35%	1.9	0.75	30	0.468
609.9	7.5	2.29	CCR (Embankment)	85	777.5	778	0.98	0.098	24	25	≥35%	1.6	0.75	30	0.468
607.4	10.0	3.05	CCR (Embankment)	85	990	990	0.98	0.097	19	19	≥35%	1.5	0.80	27	0.350
602.4	15.0	4.57	CCR (Embankment)	85	1415	1415	0.97	0.096	5	4	≥35%	1.2	0.85	10	0.115
597.4	20.0	6.10	Dredge Spoil	100	1915	1915	0.95	0.095	5	4	≥35%	1.1	0.95	10	0.113
592.4	25.0	7.62	Alluvium	120	2515	2421	0.94	0.097	2	1	≥35%	0.9	0.95	7	0.086
587.4	30.0	9.15	Alluvium	120	3115	2709	0.93	0.106	3	2	≥35%	0.9	0.95	8	0.092

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)

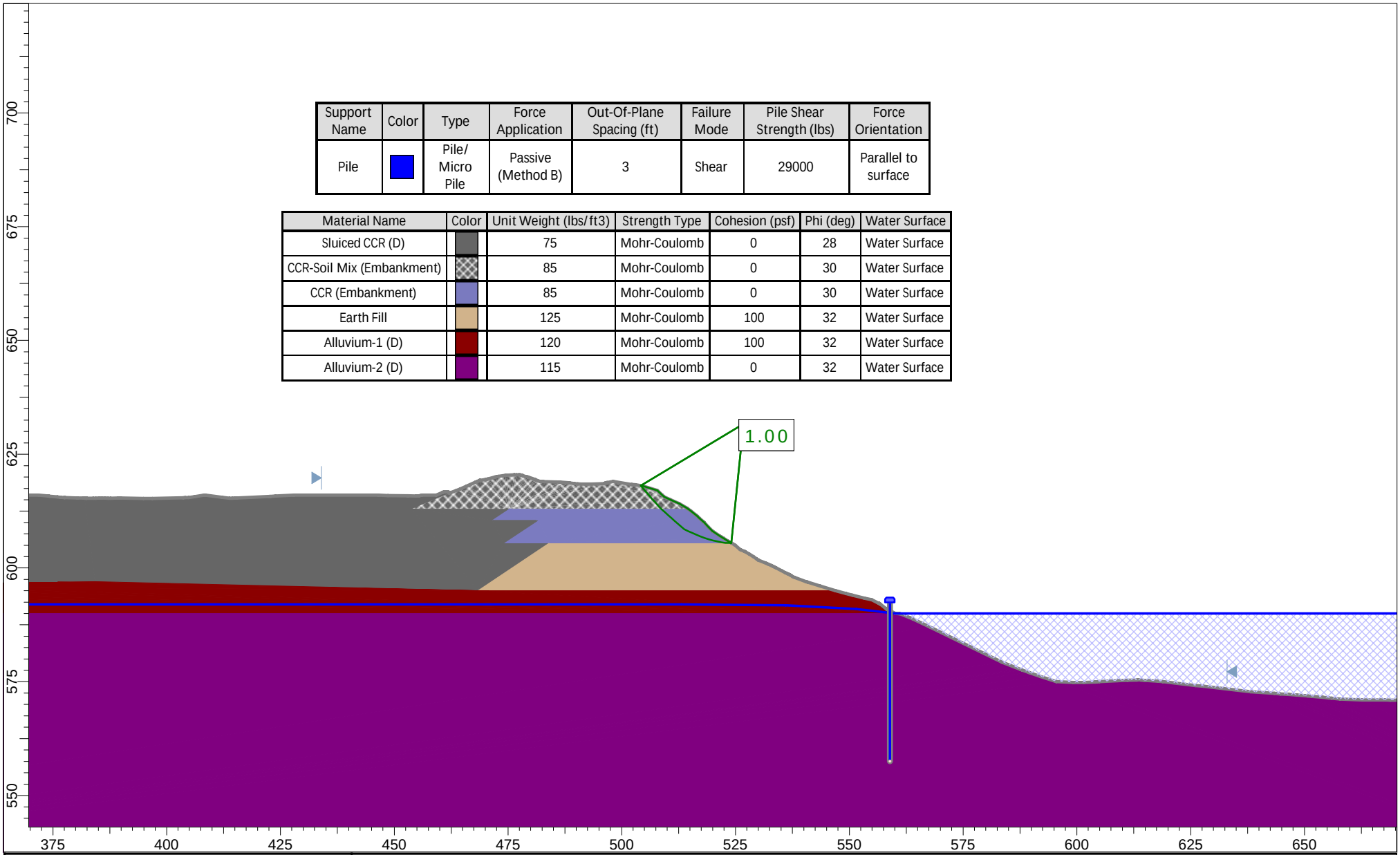
1. Youd, B. T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Jr, L. F. H., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., Iii, W. F. M., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Ii, K. H. S. (2001). Liquef
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)

APPENDIX E

Slope Stability Analyses Outputs

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Pile		Pile/ Micro Pile	Passive (Method B)	3	Shear	29000	Parallel to surface

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Sluiced CCR (D)		75	Mohr-Coulomb	0	28	Water Surface
CCR-Soil Mix (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
CCR (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
Earth Fill		125	Mohr-Coulomb	100	32	Water Surface
Alluvium-1 (D)		120	Mohr-Coulomb	100	32	Water Surface
Alluvium-2 (D)		115	Mohr-Coulomb	0	32	Water Surface

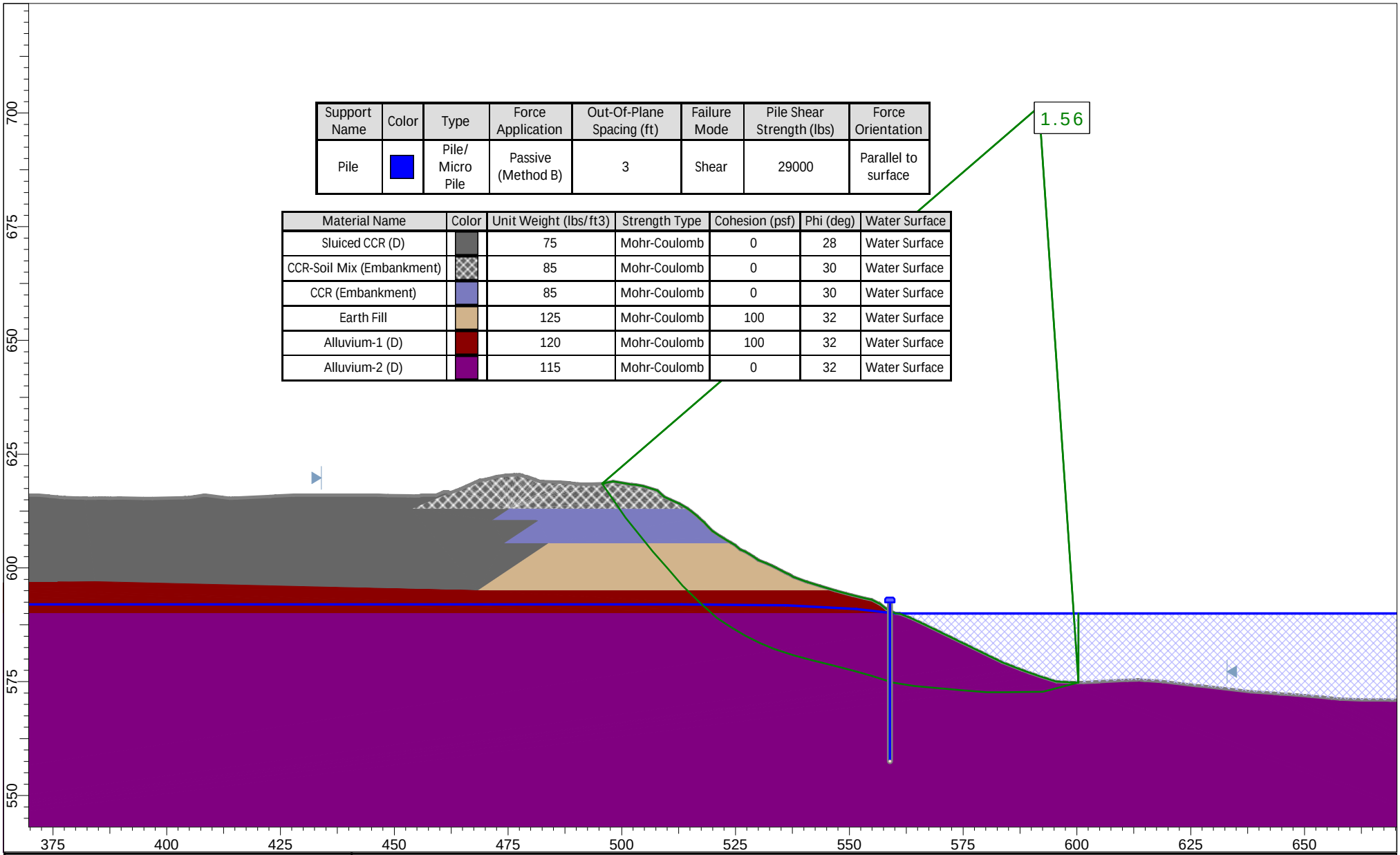


SLIDEINTERPRET 9.026

Project: Cabin Creek 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: 1:350	Client: AEP	Figure: A-1
Date: 7/1/2026					

Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Pile		Pile/ Micro Pile	Passive (Method B)	3	Shear	29000	Parallel to surface

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Sluiced CCR (D)		75	Mohr-Coulomb	0	28	Water Surface
CCR-Soil Mix (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
CCR (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
Earth Fill		125	Mohr-Coulomb	100	32	Water Surface
Alluvium-1 (D)		120	Mohr-Coulomb	100	32	Water Surface
Alluvium-2 (D)		115	Mohr-Coulomb	0	32	Water Surface

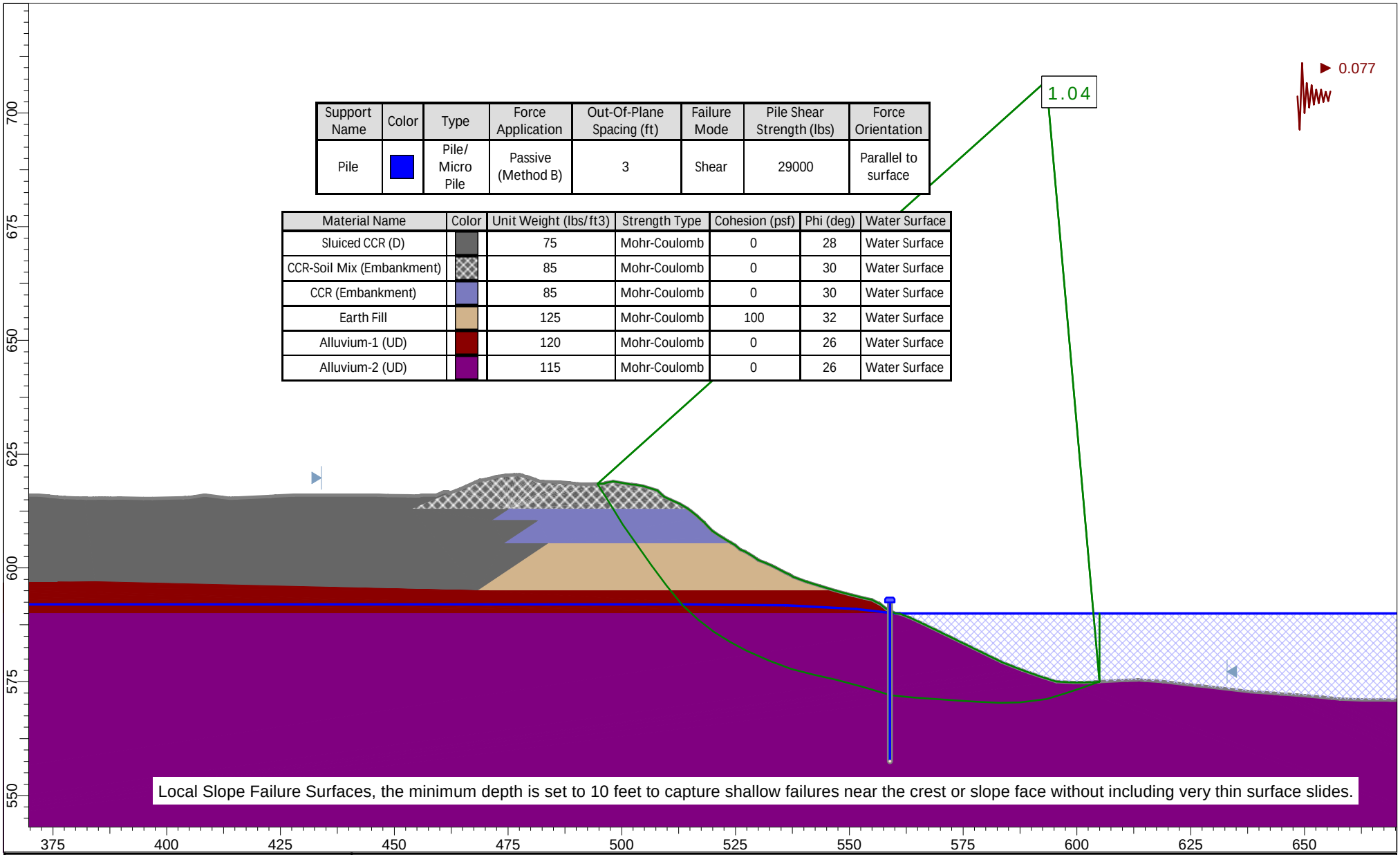


SLIDEINTERPRET 9.026

Project	Cabin Creek 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: 1:350
Client: AEP	Figure: A-2
Date: 7/1/2026	

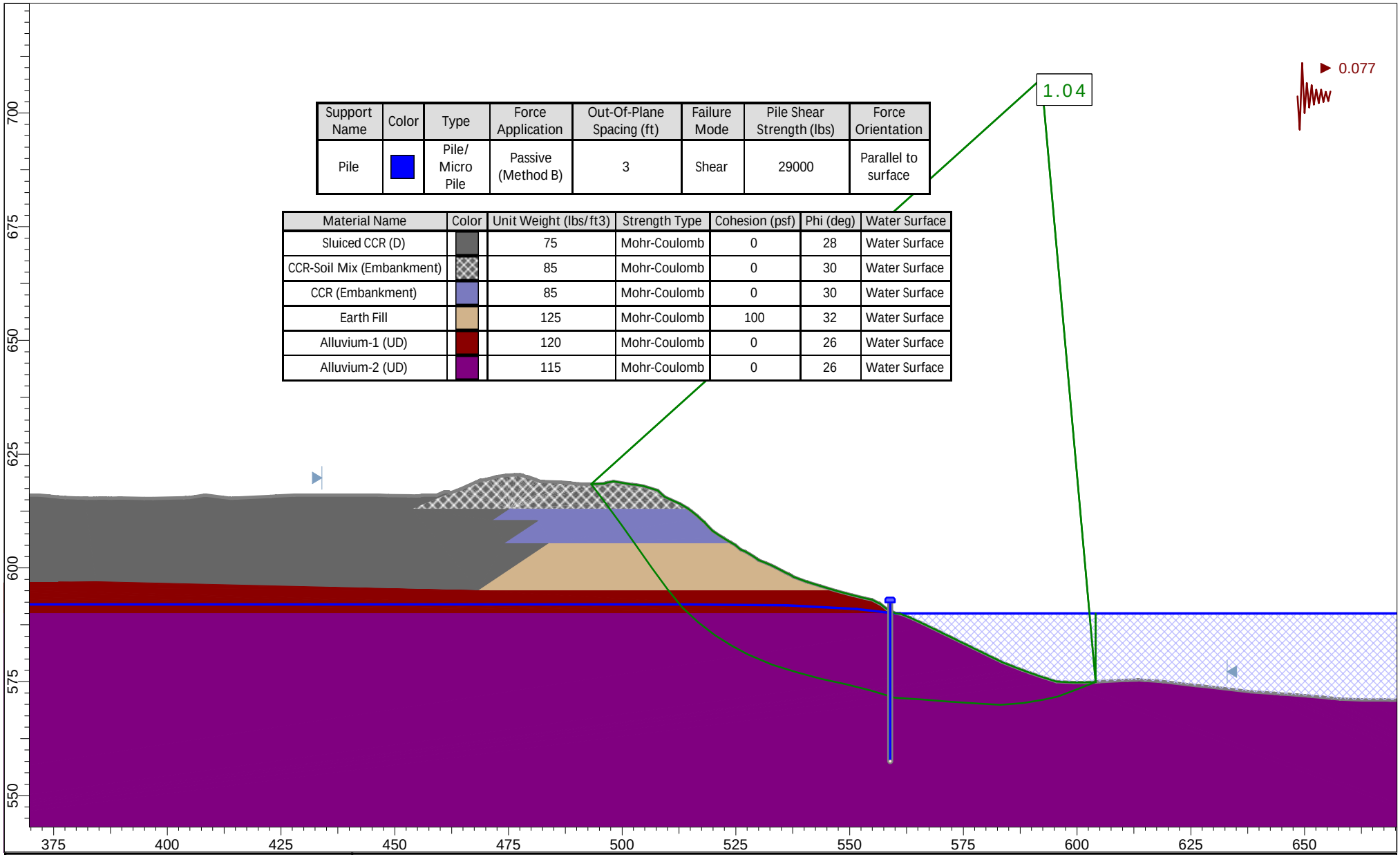
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Pile		Pile/ Micro Pile	Passive (Method B)	3	Shear	29000	Parallel to surface

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Sluiced CCR (D)		75	Mohr-Coulomb	0	28	Water Surface
CCR-Soil Mix (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
CCR (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
Earth Fill		125	Mohr-Coulomb	100	32	Water Surface
Alluvium-1 (UD)		120	Mohr-Coulomb	0	26	Water Surface
Alluvium-2 (UD)		115	Mohr-Coulomb	0	26	Water Surface



SLIDEINTERPRET 9.026

Project	Cabin Creek 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: 1:350
Client: AEP	Figure: A-3
Date: 7/1/2026	



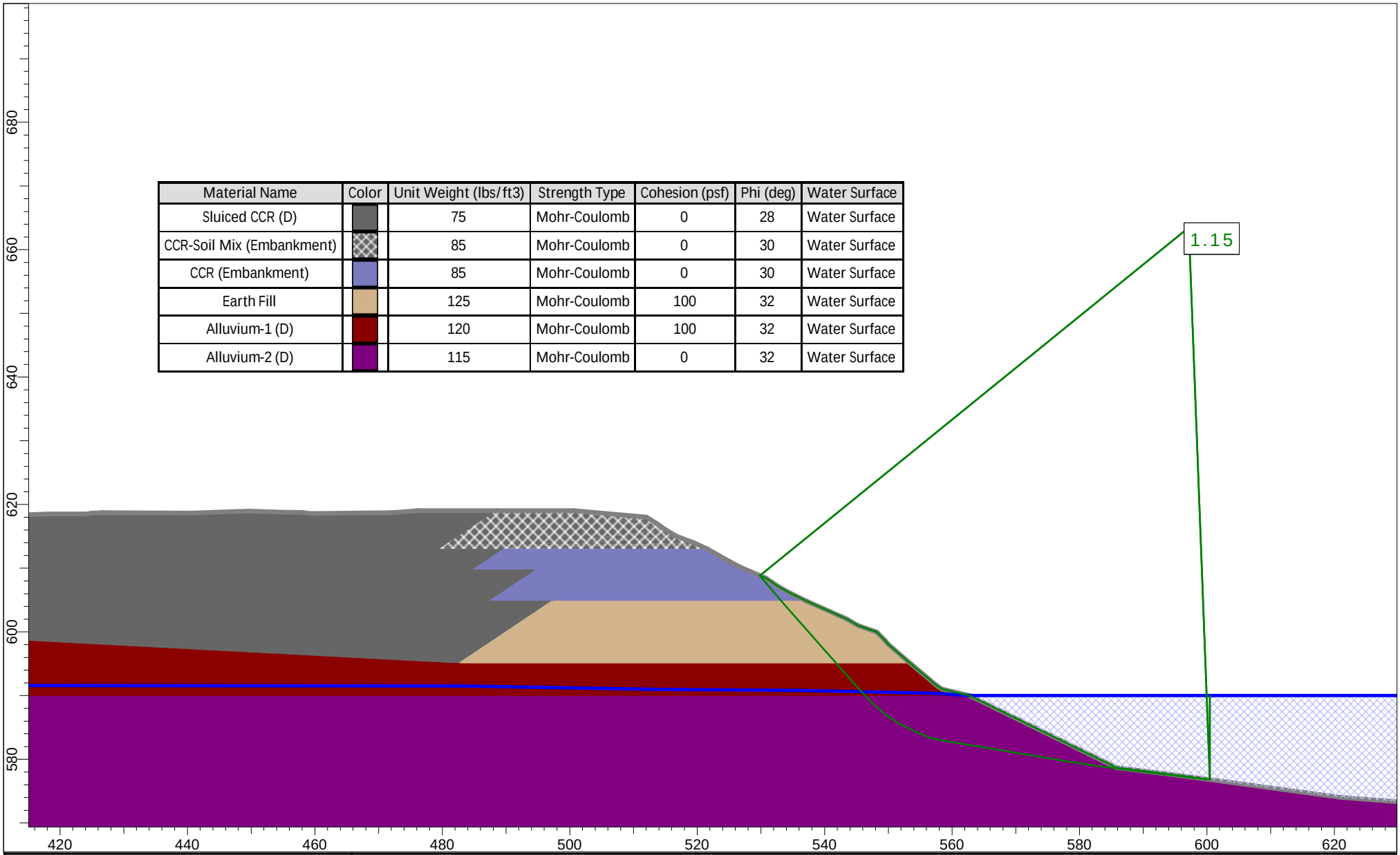
Support Name	Color	Type	Force Application	Out-Of-Plane Spacing (ft)	Failure Mode	Pile Shear Strength (lbs)	Force Orientation
Pile		Pile/ Micro Pile	Passive (Method B)	3	Shear	29000	Parallel to surface

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Sluiced CCR (D)		75	Mohr-Coulomb	0	28	Water Surface
CCR-Soil Mix (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
CCR (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
Earth Fill		125	Mohr-Coulomb	100	32	Water Surface
Alluvium-1 (UD)		120	Mohr-Coulomb	0	26	Water Surface
Alluvium-2 (UD)		115	Mohr-Coulomb	0	26	Water Surface

SLIDEINTERPRET 9.026

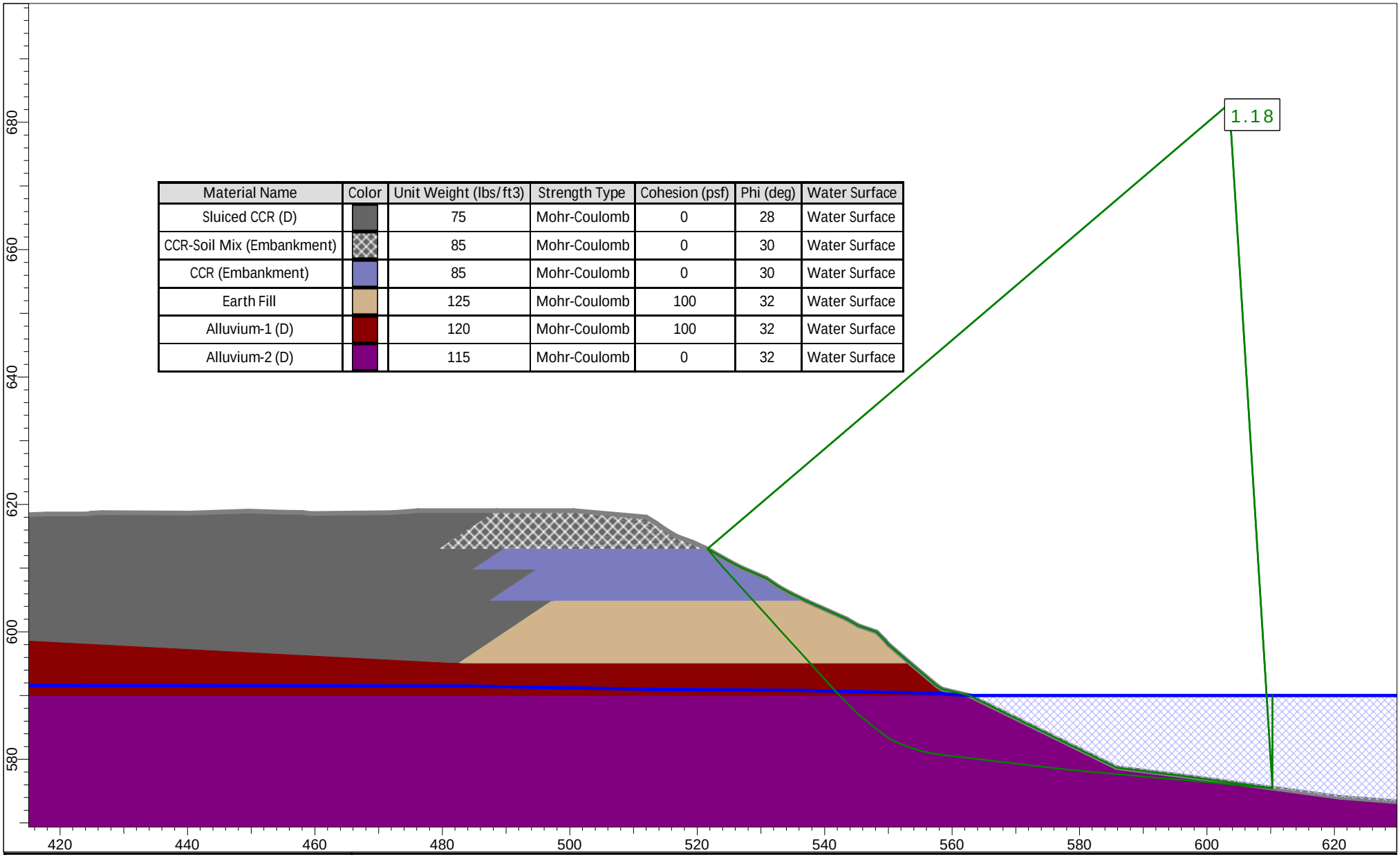
Project	Cabin Creek Ash Pond - Slope Stability Analyses
Description:	AEP Sec A-A (Existing Conditions, Pseudo-static) - Global Stability Analyses
Drawn by: NG	Checked by: SM
Reviewed by: SM	Scale: 1:350
Client: AEP	Figure: A-4
Date: 7/1/2026	

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Sluiced CCR (D)		75	Mohr-Coulomb	0	28	Water Surface
CCR-Soil Mix (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
CCR (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
Earth Fill		125	Mohr-Coulomb	100	32	Water Surface
Alluvium-1 (D)		120	Mohr-Coulomb	100	32	Water Surface
Alluvium-2 (D)		115	Mohr-Coulomb	0	32	Water Surface



SLIDEINTERPRET 9.026

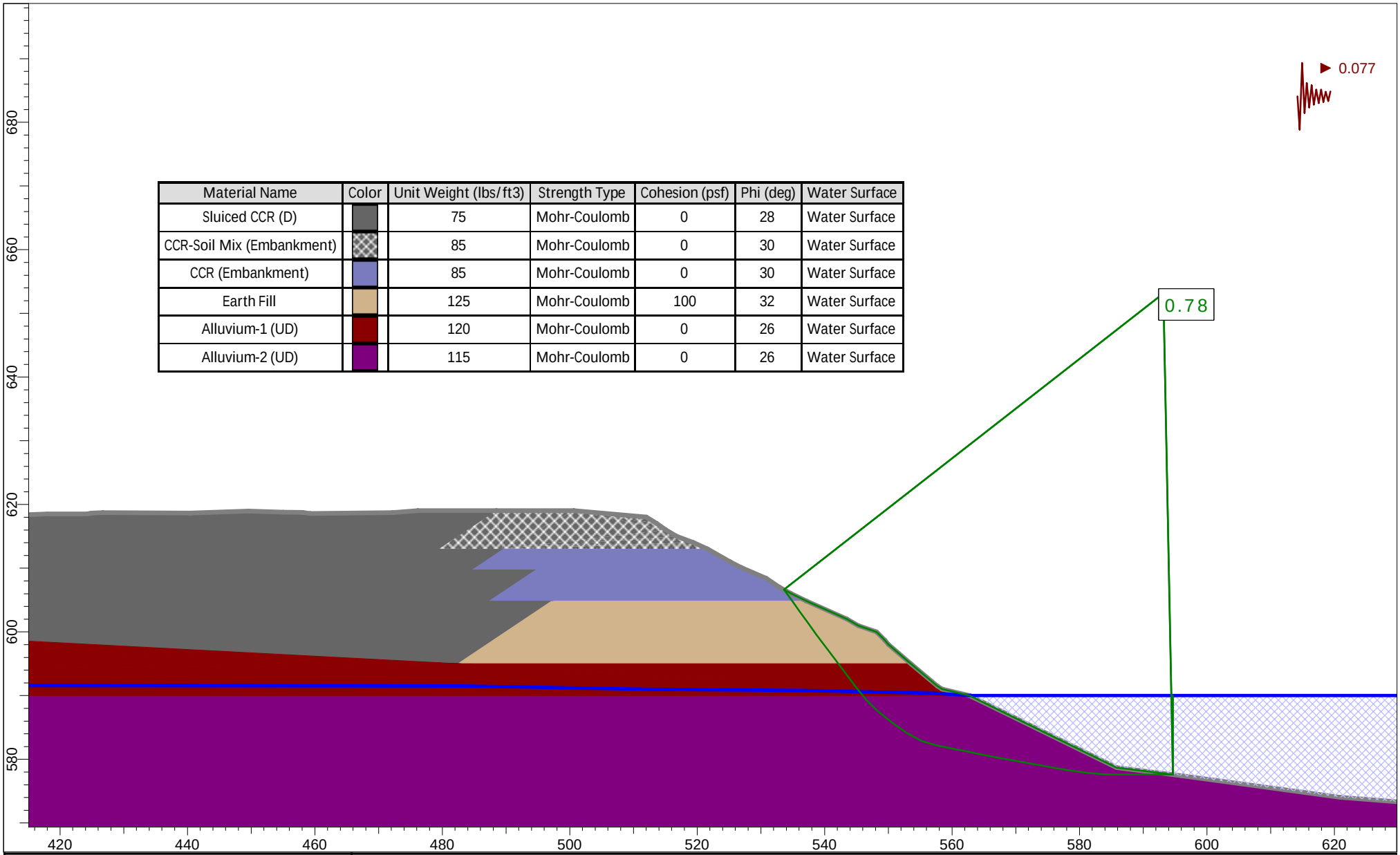
Project	Cabin Creek Ash Pond - Slope Stability Analyses
Description:	AEP Sec B-B (Existing Conditions, Long-term Static) - Local Stability Analyses
Drawn by: NG	Checked by: SM
Reviewed by: SM	Scale: 1:250
Client: AEP	Figure: B-1
Date: 7/1/2026	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Sluiced CCR (D)		75	Mohr-Coulomb	0	28	Water Surface
CCR-Soil Mix (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
CCR (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
Earth Fill		125	Mohr-Coulomb	100	32	Water Surface
Alluvium-1 (D)		120	Mohr-Coulomb	100	32	Water Surface
Alluvium-2 (D)		115	Mohr-Coulomb	0	32	Water Surface

SLIDEINTERPRET 9.026

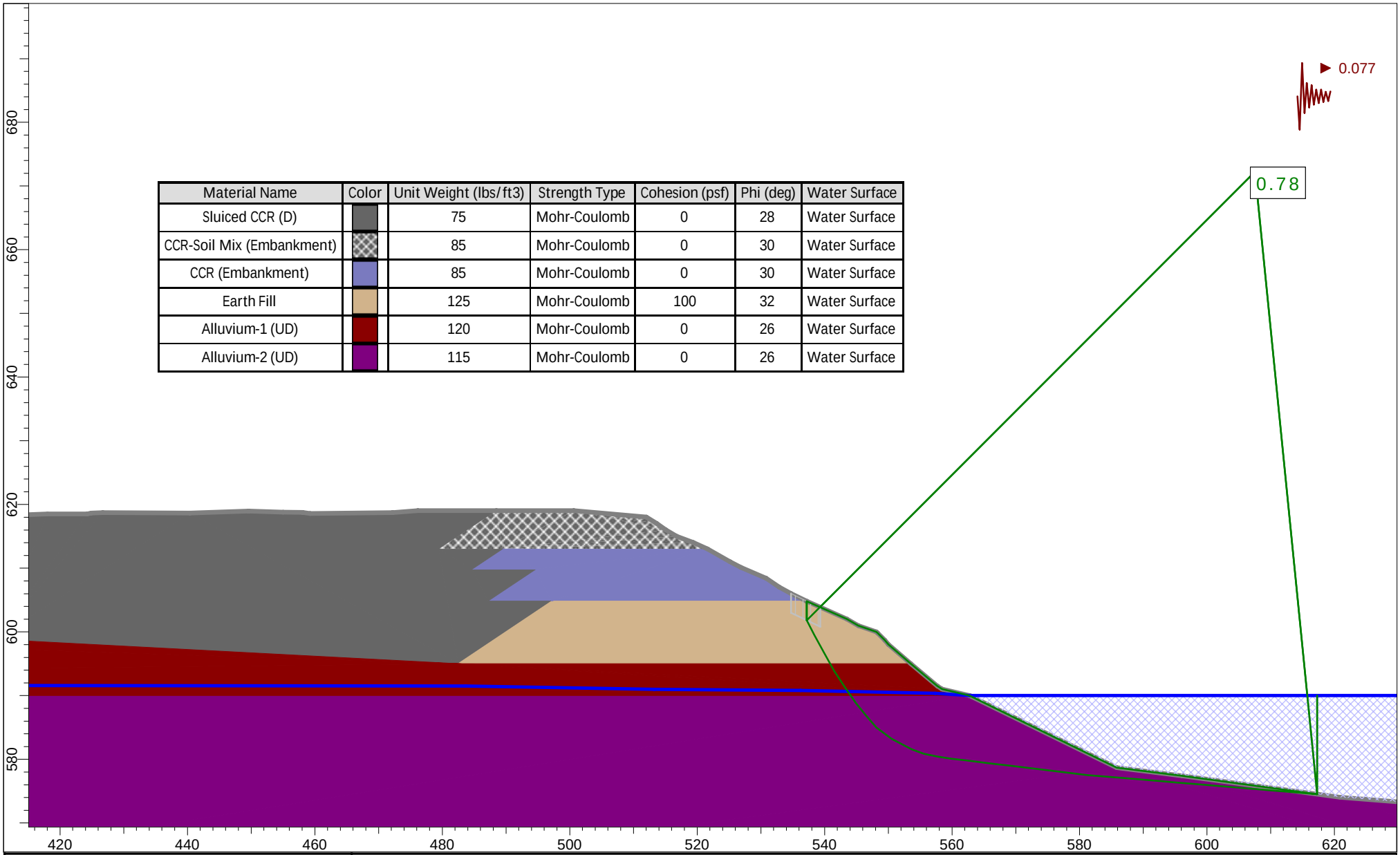
Project: Cabin Creek Ash Pond - Slope Stability Analyses					
Description: AEP Sec B-B (Existing Conditions, Long-term Static) - Global Stability Analyses					
Drawn by: NG	Checked by: SM	Reviewed by: SM	Scale: 1:250	Client: AEP	Figure: B-2
Date: 7/1/2026					



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Sluiced CCR (D)		75	Mohr-Coulomb	0	28	Water Surface
CCR-Soil Mix (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
CCR (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
Earth Fill		125	Mohr-Coulomb	100	32	Water Surface
Alluvium-1 (UD)		120	Mohr-Coulomb	0	26	Water Surface
Alluvium-2 (UD)		115	Mohr-Coulomb	0	26	Water Surface

SLIDEINTERPRET 9.026

Project	Cabin Creek Ash Pond - Slope Stability Analyses				
Description:	AEP Sec B-B (Existing Conditions, Pseudo-static) - Local Stability Analyses				
Drawn by: NG	Checked by: SM	Reviewed by: SM	Scale: 1:250	Client: AEP	Figure: B-3
Date: 7/1/2026					

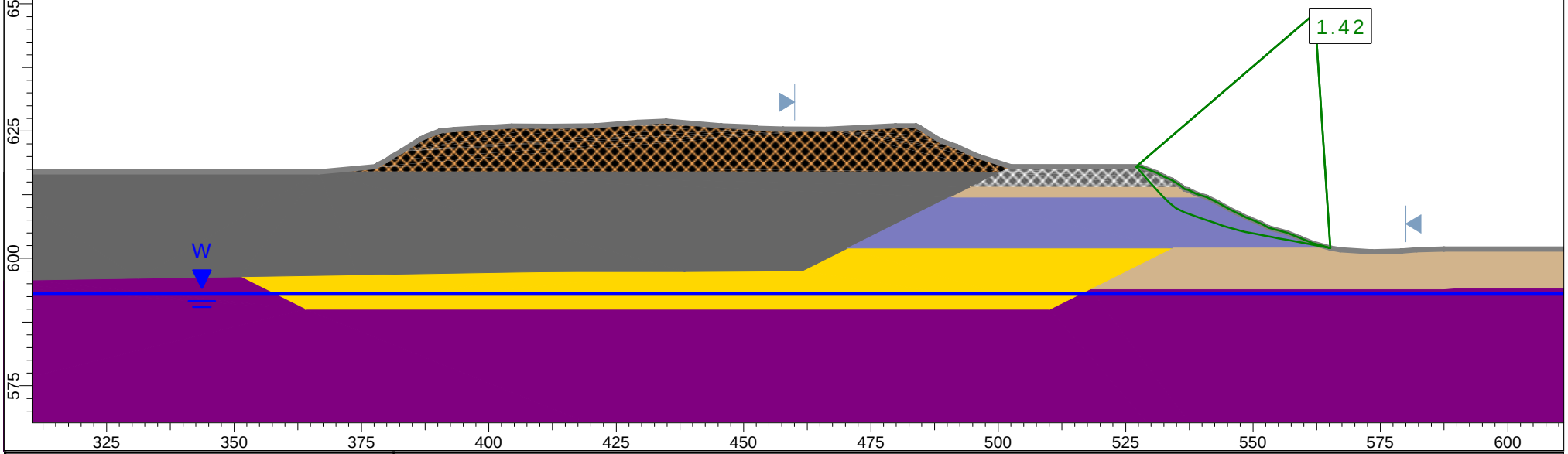


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Sluiced CCR (D)		75	Mohr-Coulomb	0	28	Water Surface
CCR-Soil Mix (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
CCR (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
Earth Fill		125	Mohr-Coulomb	100	32	Water Surface
Alluvium-1 (UD)		120	Mohr-Coulomb	0	26	Water Surface
Alluvium-2 (UD)		115	Mohr-Coulomb	0	26	Water Surface

SLIDEINTERPRET 9.026

Project	Cabin Creek Ash Pond - Slope Stability Analyses				
Description:	AEP Sec B-B (Existing Conditions, Pseudo-static) - Global Stability Analyses				
Drawn by: NG	Checked by: SM	Reviewed by: SM	Scale: 1:250	Client: AEP	Figure: B-4
Date: 7/1/2026					

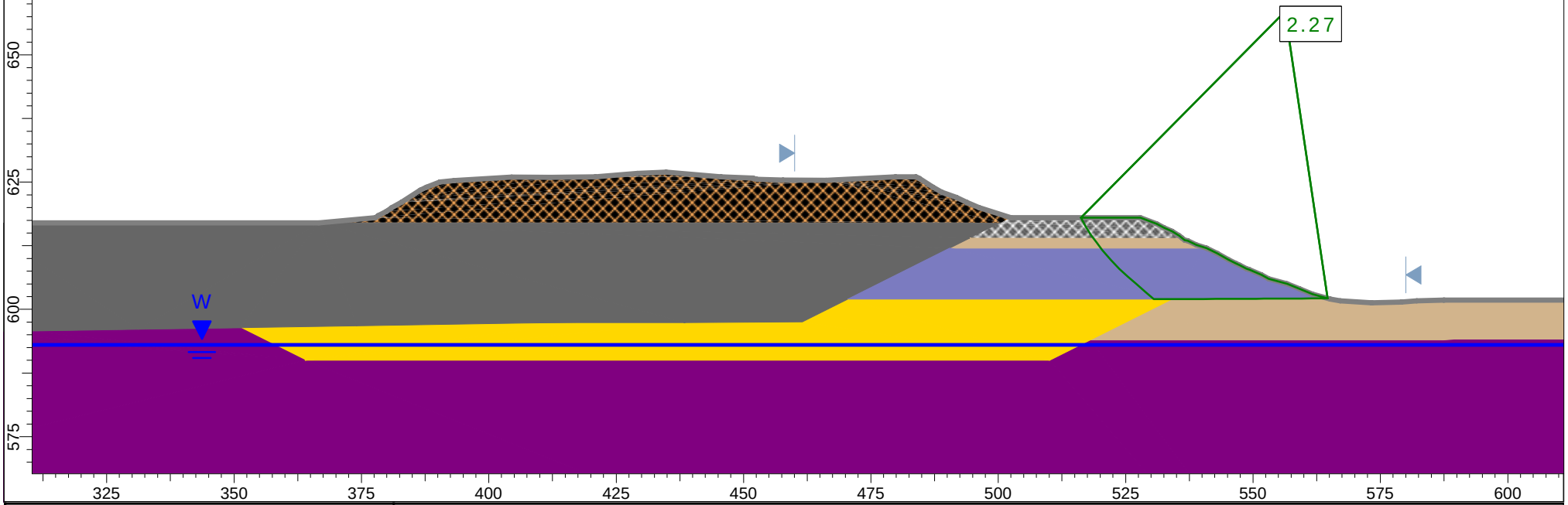
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
CCR-Soil Mix (Impounded)		75	Mohr-Coulomb	0	28	Water Surface
Sluiced CCR (D)		75	Mohr-Coulomb	0	28	Water Surface
CCR-Soil Mix (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
CCR (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
Earth Fill		125	Mohr-Coulomb	100	32	Water Surface
Dredge Spoil		100	Mohr-Coulomb	0	28	Water Surface
Alluvium-2 (D)		115	Mohr-Coulomb	0	32	Water Surface



SLIDEINTERPRET 9.026

Project	Cabin Creek Ash Pond - Slope Stability Analyses
Description:	AEP Sec C-C (Existing Conditions, Long-term Static) - Local Stability Analyses
Drawn by: NG	Checked by: SM
Reviewed by: SM	Scale: 1:350
Client: AEP	Figure: C-1
Date: 7/1/2026	

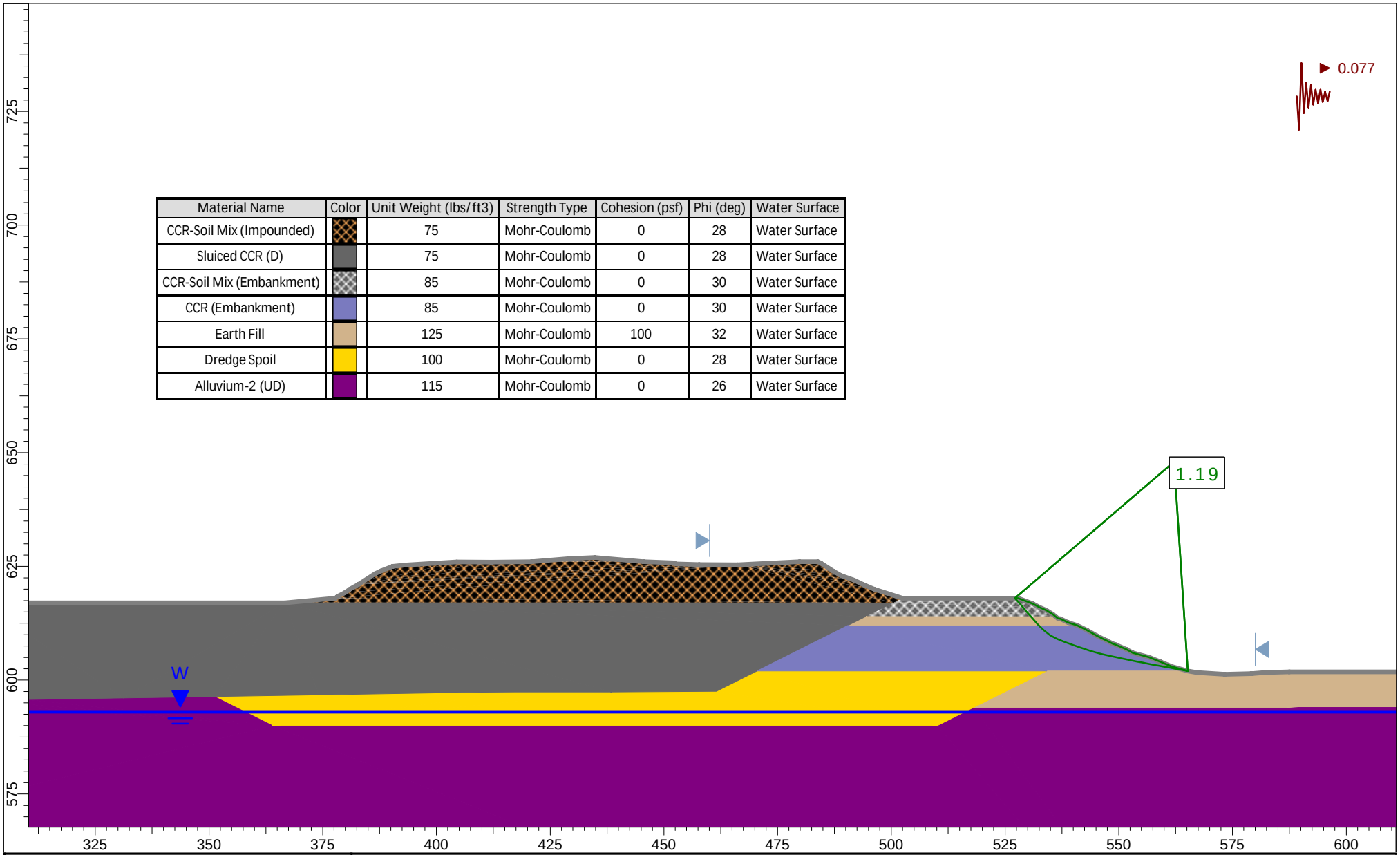
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
CCR-Soil Mix (Impounded)		75	Mohr-Coulomb	0	28	Water Surface
Sluiced CCR (D)		75	Mohr-Coulomb	0	28	Water Surface
CCR-Soil Mix (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
CCR (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
Earth Fill		125	Mohr-Coulomb	100	32	Water Surface
Dredge Spoil		100	Mohr-Coulomb	0	28	Water Surface
Alluvium-2 (D)		115	Mohr-Coulomb	0	32	Water Surface



SLIDEINTERPRET 9.026

Project	Cabin Creek Ash Pond - Slope Stability Analyses
Description:	AEP Sec C-C (Existing Conditions, Long-term Static) - Global Stability Analyses
Drawn by: NG	Checked by: SM
Reviewed by: SM	Scale: 1:350
Client: AEP	Figure: C-2
Date: 7/1/2026	

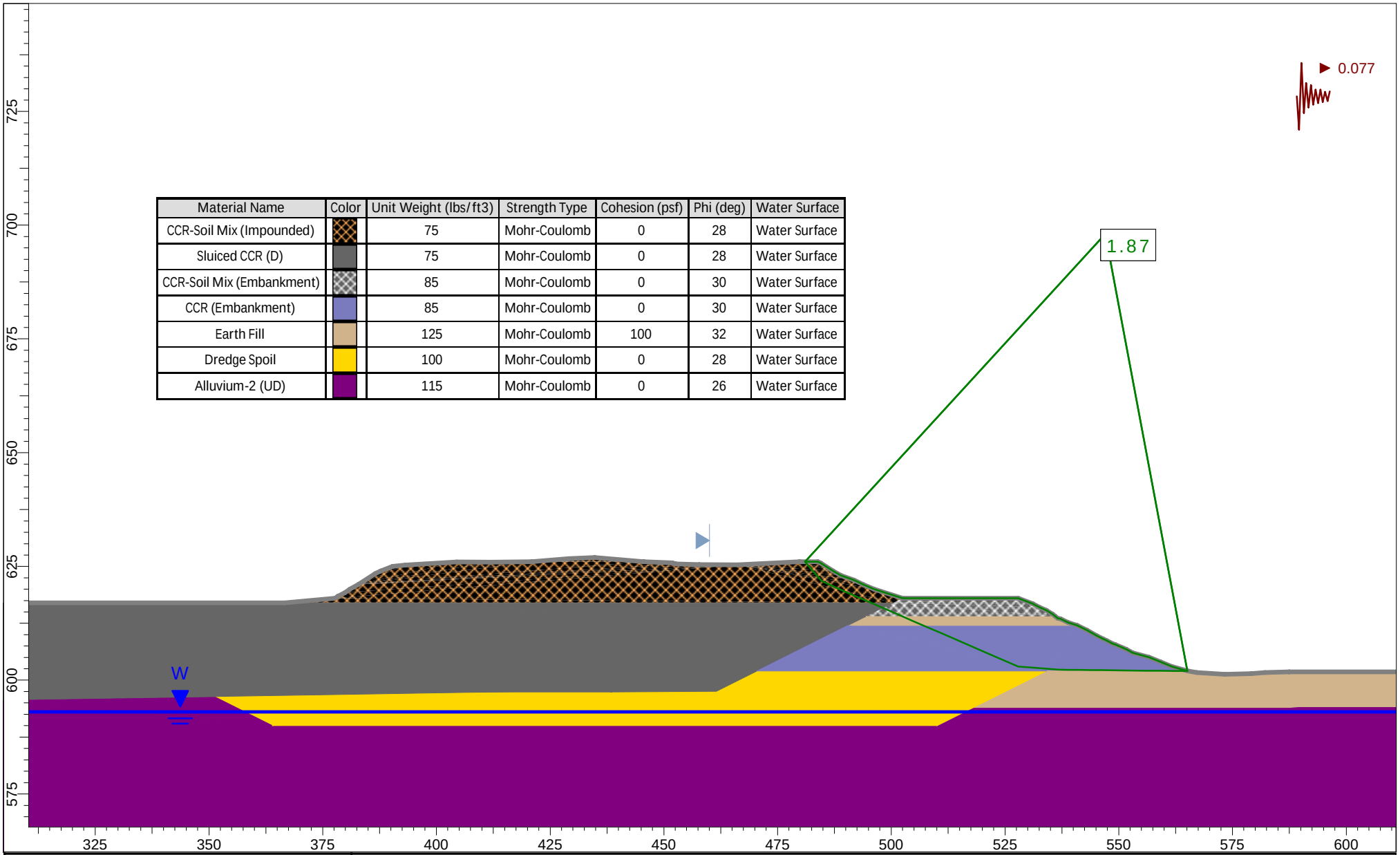
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
CCR-Soil Mix (Impounded)		75	Mohr-Coulomb	0	28	Water Surface
Sluiced CCR (D)		75	Mohr-Coulomb	0	28	Water Surface
CCR-Soil Mix (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
CCR (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
Earth Fill		125	Mohr-Coulomb	100	32	Water Surface
Dredge Spoil		100	Mohr-Coulomb	0	28	Water Surface
Alluvium-2 (UD)		115	Mohr-Coulomb	0	26	Water Surface



SLIDEINTERPRET 9.026

Project	Cabin Creek Ash Pond - Slope Stability Analyses
Description:	AEP Sec C-C (Existing Conditions, Pseudo-static) - Local Stability Analyses
Drawn by: NG	Checked by: SM
Reviewed by: SM	Scale: 1:350
Client: AEP	Figure: C-3
Date: 7/1/2026	

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
CCR-Soil Mix (Impounded)		75	Mohr-Coulomb	0	28	Water Surface
Sluiced CCR (D)		75	Mohr-Coulomb	0	28	Water Surface
CCR-Soil Mix (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
CCR (Embankment)		85	Mohr-Coulomb	0	30	Water Surface
Earth Fill		125	Mohr-Coulomb	100	32	Water Surface
Dredge Spoil		100	Mohr-Coulomb	0	28	Water Surface
Alluvium-2 (UD)		115	Mohr-Coulomb	0	26	Water Surface



SLIDEINTERPRET 9.026

Project	Cabin Creek Ash Pond - Slope Stability Analyses
Description:	AEP Sec C-C (Existing Conditions, Pseudo-static) - Global Stability Analyses
Drawn by: NG	Checked by: SM
Reviewed by: SM	Scale: 1:350
Client: AEP	Figure: C-4
Date: 7/1/2026	