

*Prepared for*



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# **2026 PERIODIC SAFETY FACTOR ASSESSMENT ASH POND A**

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## CERTIFICATION STATEMENT

This periodic safety factor assessment meets the requirements of §257.73(e) of the Code of Federal Regulations Title 40, Part 257, Subpart D, and was prepared in accordance with current practices and the standard of care exercised by scientists and engineers performing similar tasks in the field of civil engineering, and no other warranty is provided in connection therewith. The contents of this report are based solely on the observations of the conditions observed by Geosyntec personnel and information provided to Geosyntec by Santee Cooper. Consistent with applicable professional standards of care, our opinions and recommendations were based in part on data furnished by others. Although we were not able to independently verify such data, we found that it was consistent with other information that we developed in the course of our performance of the scope of services. The information contained in this report is intended for use solely by Santee Cooper.



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## 1. INTRODUCTION

### 1.1 Project Background

The Winyah Generating Station (WGS or Site) is an electric generating facility owned and operated by Santee Cooper. WGS is located between Pennyroyal and Turkey Creeks, tributaries to Sampit River, and is situated approximately four miles southwest of Georgetown, South Carolina (SC) (see Figures 1a and 1b for Site Location and Site Vicinity Maps, respectively).

On 17 April 2015, the United States Environmental Protection Agency (USEPA) published rules in 40 CFR Part 257 that regulate the design and management of existing and new Coal Combustion Residuals (CCR) units (CCR Rule). The CCR Rule became effective on 17 October 2015. Within the CCR Rule, §257.73(e) specifies the safety factor criteria for existing CCR surface impoundments.

Ash Pond A, encompassing approximately 26.3 acres (ac), is located southeast of the power block (Figure 2). Historically, Ash Pond A contained CCR in the form of fly ash, boiler slag, and bottom ash as well as stormwater. Efforts to remove CCR from Ash Pond A was initiated in 2015 and completed in 2021. The construction of a Class III Landfill at Ash Pond A was started in early 2022. Although Ash Pond A no longer contains CCR, the pond has not yet received official closure certification. It is considered as an existing surface impoundment under the CCR Rule.

In accordance with §257.102(g), a Notice of Intent for Ash Pond A was posted to the Operating Record on 9 April 2021 to initiate pond closure, and CCR and wastewater inflow to Ash Pond A ceased in April 2021.

This *2026 Periodic Safety Factor Assessment: Ash Pond A* (Safety Factor Assessment Report) was prepared by Geosyntec Consultants, Inc. (Geosyntec) on behalf of Santee Cooper to demonstrate that Ash Pond A satisfies criteria for the periodic safety factor assessments in accordance with §257.73(e) of the CCR Rule.

### 1.2 Project Site and Construction History

Ash Pond A currently encompasses approximately 26.3 ac and historically covered approximately 90 ac prior to the construction of the Class III Landfill. This unlined surface impoundment was commissioned in 1975 and was designated for the disposal of fly ash, bottom ash, and boiler slag. Ash Pond A is bounded by Landfill Area 2 to the north, the Cooling Pond to the east, Ash Pond B to the south, and the Discharge Canal to the west. Ash Pond A is separated from Ash Pond B by a divider dike, which traverses from west to east from the Discharge Canal to the Cooling Pond.

Ash Pond A was assigned “Low Hazard Potential” classification (Geosyntec, 2021a).

Ash Pond A was constructed by recompacting excavated soils from the impoundment interior to form the perimeter dikes and a divider dike. Ash Pond A perimeter dikes are approximately 12 ft to 15 ft in height along the north and west sides and approximately 20 ft to 24.5 ft in height along

the east side adjacent to the Cooling Pond (Thomas and Hutton, 2012). The upstream and downstream slopes of the perimeter dikes range from 2 Horizontal to 1 Vertical (2H:1V) to 3H:1V. The Ash Pond A dike crest is approximately 12- to 15-ft wide with an approximate elevation between 38.0 ft and 44.0 ft National Geodetic Vertical Datum of 1929 (NGVD29) (Thomas and Hutton, 2012).

### **1.3 Report Organization**

This Safety Factor Assessment Report presents the subsequent periodic safety factor assessments for Ash Pond A at WGS based on the results of the initial periodic safety factor assessments (2016 Safety Factor Assessment and 2021 Safety Factor Assessment) (Geosyntec, 2016, 2021), recent survey dated January 2026 (Worley, 2026), subsequent hydrologic and hydraulic (H&H) analyses and geotechnical engineering analyses, and reviews of available Site information. The remainder of this Safety Factor Assessment Report is organized as follows:

- Summary of changes in site conditions since the 2021 Safety Factor Assessments is presented in Section 2.
- H&H evaluation of Ash Pond A is presented in Section 3;
- Seismic hazard evaluations for WGS, the site response analysis of the Ash Pond A perimeter dikes and liquefaction potential evaluation are presented in Section 4;
- Slope stability analyses performed for the safety factor assessment are discussed in Section 5; and
- The summary and general conclusions are presented in Section 6.

## **2. CHANGES IN SITE CONDITIONS**

Santee Cooper personnel indicated that no changes were made to the Ash Pond A perimeter dikes and adjacent areas outside the dikes since preparation of the 2021 Safety Factor Assessment Report. Also, no additional geotechnical subsurface investigations were conducted since 2016; therefore, the subsurface stratigraphy developed in the 2016 Safety Factor Assessment remains valid. Class III Landfill Area 2 is being constructed within the northwestern portion of Ash Pond A. A review of the topographic survey dated January 2026 (Worley, 2026) and the topographic survey used in the 2021 Safety Factor Assessment indicated that all of the CCR have been excavated from Ash Pond A. There is no CCR impoundment within Ash Pond A.

Groundwater level measurements previously provided by Santee Cooper were reviewed and compared with the 2026 topographic survey. Based on this review, no site conditions indicative of a substantial change in groundwater conditions were identified. Therefore, for the purposes of this assessment, groundwater levels within the perimeter dike and beyond the downstream toe of the

perimeter dike were assumed to be consistent with those adopted in the 2016 and 2021 Safety Factor Assessments.

As discussed above, CCR and wastewater inflow to Ash Pond A ceased in April 2021. Since the 2021 report, the pipes between Ash Ponds A and B have been removed or plugged and the former spillway has been backfilled. As a result, the two ponds are no longer hydraulically connected.

### 3. HYDROLOGIC AND HYDRAULIC EVALUATION

#### 3.1 Hydrologic and Hydraulic Analysis

The following subsections discuss the regulatory framework, the methodology and assumptions, and the results of the H&H analysis for Ash Pond A and its appurtenances.

##### 3.1.1 Regulatory Framework

The CCR Rule (§257.73(d)(1)) requires that a periodic stability assessment:

*“...at a minimum, document whether the CCR unit has been designed, constructed, and maintained with:*

*...*

*(v) a single spillway or a combination of spillways configured as specified in paragraph (d)(1)(v)(A) of this section. The combined capacity of all spillways must be designed, constructed, operated, and maintained to adequately manage flow during and following the peak discharge event specified in paragraph (d)(1)(v)(B) of this section.”*

The CCR Rule (§257.73(d)(1)(v)(B)(3)) also states that the spillways must manage the peak discharge from the “100-year flood for a low hazard potential CCR surface impoundment.” Additionally, §257.73(d)(1)(v)(A) indicates that “All spillways must be either:

*(1) Of non-erodible construction and designed to carry sustained flows; or*

*(2) Earth- or grass-lined and designed to carry short-term, infrequent flows at non-erosive velocities where sustained flows are not expected.”*

Meanwhile, §257.73(e)(1) of the CCR Rule indicates:

*“(ii) The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40.”*

Because Ash Pond A was classified as a “Low Hazard Potential” surface impoundment, the 100-year storm event was selected as the inflow design flood (IDF). H&H analyses were performed to demonstrate that the inflow design flood control system for Ash Pond A maintains the minimum operating freeboard for the surface impoundment without overtopping of perimeter dikes, meeting

the criteria in §257.73(d)(1)(v). Considering the requirements of §257.73(d)(1) listed above, this Safety Factor Assessment Report established the “maximum surcharge pool” elevation in the slope stability analysis to demonstrate that the requirements of §257.73(e)(1)(ii) are met, based on the maximum water elevation within Ash Pond A computed from the H&H analysis.

### 3.1.2 Methodology and Assumptions

Stormwater runoff volumes and associated discharges to Ash Pond A were modeled using *HydroCAD Version 10.20* software (HydroCAD, 2024). *HydroCAD* utilizes frequency-based precipitation events, in conjunction with watershed properties, to calculate peak runoff by several accepted methods. The Soil Conservation Service (SCS) Technical Release 20 (TR-20) method was applied in *HydroCAD* to calculate stormwater runoff volumes (SCS, 1982).

Historically, Ash Pond A decanted ash sluice water, low volume wastewater, and Unit 2 Slurry Pond stormwater to Ash Pond B. Presently, Ash Pond A only receives stormwater inflows from the immediate pond area. Since the 2021 assessment, the three culverts connecting Ash Ponds A and B have been abandoned, and the former emergency spillway connecting Ash Ponds A and B has been backfilled. As a result, Ash Ponds A and B are no longer hydraulically connected. Geosyntec visited WGS on 8 April 2026 to inspect the condition of the CCR surface impoundment dikes and free water was not present within Ash Pond A during the site visit.

Details of the H&H analysis are provided in a document titled “*Inflow Design Flood Control System Plan: Ash Pond A*” (Geosyntec, 2026c) and as Attachment 2 of this Report.

### 3.1.3 Analysis Results

Under the conditions and assumptions described in Section 3.1.2, the maximum free water level during and following the IDF event (100-year rainfall with a 72-hour duration) was computed as 25.0 ft (Plant Vertical Datum). The H&H analysis results (i.e., *HydroCAD* results) are included as Attachment 2 of this Safety Factor Assessment Report.

## 4. SEISMIC HAZARD EVALUATION AND SITE RESPONSE ANALYSIS

### 4.1 Introduction

CCR regulations require site response analyses to be reevaluated every five years. The 2021 site response analysis by Geosyntec utilized the 2018 United States Geological Survey (USGS) seismic hazard curves to develop the target response spectrum and corresponding input motions at the base of the site response models. As part of the 2026 update, the target response spectrum was developed using the 2023 USGS hazard curves, and the impacts of the updated seismic hazard characterization on the site response analyses were evaluated.

## 4.2 Target Acceleration Response Spectra

A comparison of the 750-year return period hazard spectra derived from the 2018 and 2023 USGS hazard curves indicates only minor differences in the seismic hazard characterization as follows:

- Peak ground acceleration (PGA) increased from 0.153 g to 0.164 g;
- Spectral accelerations increased at periods between 0 and 0.02 seconds and between 0.15 and 10 seconds; and
- Spectral accelerations decreased at periods between 0.02 and 0.15 seconds.

To account for the increase in PGA, the input motion suite was uniformly scaled upward by approximately 7.2%.

Figure 3 compares the target response spectra developed using the 2018 and 2023 USGS hazard curves. Figures 4 and 5 present the input motion suite scaled to the 2018 and 2023 target PGAs, respectively.

## 4.3 Site Response Analysis Comparison

Site response analyses were performed using the computer program RS seismic, version 2.020 (Rocscience, 2026a). The subsurface profiles and calibrated soil model parameters developed for the 2021 analyses were maintained, and only the input motion scaling was updated to reflect the increase in target PGA derived from the 2023 USGS hazard curves.

The maximum shear strain and maximum shear stress envelopes were evaluated over the full depth of the soil profile. The envelopes represent the maximum response from the suite of five input motions at each depth, allowing direct comparison between the 2021 and 2026 analyses. Comparative plots for Ash Pond A are presented in Figure 6.

Overall, the 2026 analyses show minor increases in both maximum shear strain and maximum shear stress throughout the profiles, consistent with the 7.2 percent increase in input motion amplitude. The observed differences are minor and generally limited to localized portions of the soil columns, indicating that the updated seismic hazard characterization does not materially affect the site response behavior at the evaluated surface impoundment.

## 4.4 Liquefaction Susceptibility Comparison

The liquefaction potential analyses were updated using the results of the 2026 site response analyses. Consistent with the 2021 evaluation, liquefaction triggering was assessed using the simplified procedures of Seed and Idriss (1971) and Boulanger and Idriss (2014), where the factor of safety against liquefaction triggering ( $FS_{liq}$ ) is calculated as the ratio of cyclic resistance ratio

(CRR) to cyclic stress ratio (CSR). The CSR was obtained from the maximum shear stress profile developed during the site response analysis and is directly proportional to the maximum cyclic shear stress induced at a given depth.

Figures 7 through 9 compare the liquefaction factors of safety calculated using the 2021 and 2026 site response analysis results. The updated analyses indicate minor reductions in the calculated factors of safety against liquefaction triggering, consistent with the modest increase in maximum shear stress throughout the soil profiles. However, all evaluated borings and CPT soundings continue to exhibit factors of safety greater than 1.0, indicating that liquefaction triggering is not anticipated under the design earthquake.

#### **4.5 Seismic Slope Stability Evaluation**

The 2021 safety factor assessments evaluated seismic slope stability using the pseudo-static procedure presented by Hynes-Griffin and Franklin (1984). With this approach, the maximum horizontal equivalent acceleration (MHEA) at the depth of the critical slip surface was obtained from the site response analyses and used to determine the seismic coefficient ratio ( $k_h$ /MHEA) corresponding to an allowable permanent deformation of 12 inches. The 2021 assessments adopted a  $k_h$ /MHEA ratio of 0.50 based on the upper-bound relationship presented by Hynes-Griffin and Franklin (1984). The seismic coefficient,  $k_h$ , was then applied within the slope stability analyses to evaluate the seismic factor of safety.

To maintain consistency with the previous assessments, the same methodology was applied using the updated 2026 site response analysis results. Updated MHEA values were determined at the depth of the critical slip surfaces and multiplied by the selected  $k_h$ /MHEA ratio of 0.50 to calculate revised seismic coefficients. Figure 10 presents comparison of the 2021 and 2026 MHEA profiles. Table 1 summarizes the comparison of MHEA and resulting  $k_h$  values used in the 2021 and 2026 evaluations.

The comparison indicates only a minor change in the calculated seismic coefficient for Ash Pond A. Overall, the revised  $k_h$  value remains similar to that used in the 2021 assessment, indicating that the updated seismic hazard characterization results in only modest changes to the pseudo-static loading conditions applied in the slope stability analyses.

#### **4.6 Bray and Macedo (2019) Seismic Slope Stability Evaluation**

In addition to updating the pseudo-static seismic coefficients using the revised site response analyses, the seismic stability of Ash Pond A was reevaluated using the procedures developed by Bray and Macedo, which represent a more modern approach for estimating earthquake-induced slope deformations. Unlike the pseudo-static approach adopted in the 2021 assessments, the Bray and Macedo methodology directly estimates permanent seismic displacement and therefore provides a more meaningful measure of seismic performance.

The horizontal seismic coefficient ( $k_h$ ) was iteratively adjusted within the slope stability model for the critical cross section until a factor of safety equal to 1.0 was achieved. The resulting yield coefficient was then used as an input to the Bray and Macedo displacement calculations together with the site-specific ground motion parameters developed from the updated seismic hazard and site response analyses.

The calculated seismic displacement for Ash Pond A was less than 12 inches, as shown in Table 2. The estimated displacement is below the allowable deformation criteria used for the seismic stability evaluations and indicates satisfactory seismic performance under the design earthquake. Accordingly, the Bray and Macedo analyses support the conclusions of the pseudo-static slope stability evaluations and demonstrate that Ash Pond A is expected to maintain stability under the design seismic loading conditions.

#### **4.7 Conclusion**

The 2026 update incorporated the 2023 USGS seismic hazard curves into the site response analyses for the WGS Ash Pond A and evaluated the resulting impacts on site response behavior, liquefaction susceptibility, and seismic slope stability. Relative to the 2018 hazard characterization used for the 2021 analyses, the updated hazard curves resulted in a modest increase in peak ground acceleration from 0.153 g to 0.164 g and corresponding increases in the amplitudes of the input motion suite.

The updated site response analyses indicate only minor increases in maximum shear strain and maximum shear stress throughout the evaluated soil profiles. The overall shape and distribution of the response profiles remain consistent with the 2021 analyses, demonstrating that the updated seismic hazard characterization does not materially alter the seismic response of Ash Pond A.

Updated liquefaction evaluations, based on the revised shear stress profiles, resulted in minor reductions in the calculated factors of safety against liquefaction triggering. However, all evaluated borings and CPT soundings continue to exhibit factors of safety greater than 1.0, indicating that liquefaction triggering is not anticipated under the design earthquake. Consequently, the conclusions of the 2021 liquefaction assessments remain unchanged.

Seismic slope stability was reevaluated using both the original Hynes-Griffin and Franklin (1984) pseudo-static procedure and the Bray and Macedo (2019) methodology. Updated maximum horizontal equivalent accelerations (MHEA) and associated seismic coefficient were generally similar to those used in the 2021 assessments, indicating only modest changes in seismic loading conditions. Consistent with these findings, the Bray and Macedo analyses predicted permanent seismic displacements of less than 12 inches for Ash Pond A. The estimated deformation is below the allowable deformation criteria and indicates satisfactory seismic performance under the design earthquake.

Based on the results of the updated analyses, the changes associated with adoption of the 2023 USGS seismic hazard curves do not materially affect the seismic performance of Ash Pond A. The conclusions of the 2021 site response, liquefaction, and seismic stability assessments remain valid, and Ash Pond A continues to satisfy the applicable seismic performance and stability requirements of the CCR Rule.

## 5. SAFETY FACTOR ASSESSMENT

This section presents the periodic safety factor assessments for the Ash Pond A perimeter dikes. This evaluation is presented in detail in Attachment 3 of this Safety Factor Assessment Report and summarized herein.

### 5.1 Regulatory Framework

Slope stability analyses were conducted to assess whether the Ash Pond A perimeter dikes satisfy the safety factor (also referred to as “factor of safety”) criteria of §257.73(e)(1) of the CCR Rule. Specifically, §257.73(e)(1) requires that:

- “(i) The calculated static factor of safety under the long-term, maximum storage pool loading condition must equal or exceed 1.50.*
- “(ii) The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40.*
- “(iii) The calculated seismic factor of safety must equal or exceed 1.00.*
- “(iv) For embankments constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20.”*

Because the dike fills and foundation soils beneath the dike fill along the critical section of Ash Pond A are not found to be susceptible to liquefaction, as described above, the liquefaction factor of safety (i.e., §257.73(e)(1)(iv)) is not required and is not evaluated as part of this Safety Factor Assessment. The remainder of Section 5 describes the geometric model, methodology, and analysis results for each case.

### 5.2 Analysis Models

A review of the topographic survey data from January 2026 showed no CCR impounded by the perimeter dikes of Ash Pond A (Worley, 2026). During the 2021 Safety Factor assessment, cross-section E was the only cross-section with CCR impoundment by the perimeter dikes. Therefore, the same critical cross section E was selected for updated analysis in 2026 Safety Factor Assessment.

Ash Pond A was dry during Geosyntec's April 2026 site visit; therefore, the Maximum Normal Storage Pool condition was represented by an operating water level at the pond bottom. Review

of historical groundwater measurements provided by Santee Cooper and the 2026 topographic survey indicated no evidence of substantial changes in groundwater conditions, thus groundwater levels within the perimeter dike and beyond the downstream toe of the perimeter dike were assumed to be consistent with those used in the 2016 and 2021 Safety Factor Assessments. For the Maximum Surcharge Pool condition, the 100-year, 72-hour rainfall event was used as the Inflow Design Flood (IDF) in accordance with CCR regulations for a Low Hazard Potential impoundment. The resulting maximum water surface elevation of 25.0 ft (Plant Vertical Datum), identical to that estimated in the 2021 Safety Factor Assessment, was used in the 2026 Safety Factor Assessment.

### **5.3 Methodology**

#### **5.3.1 Static Slope Stability**

Global slope stability analyses were performed using Spencer's method (Spencer, 1973), as implemented in the computer program SLIDE<sup>®</sup>, version 9.043 (Rocscience, 2026b). Spencer's method, which satisfies vertical and horizontal force equilibrium as well as moment equilibrium, is considered to be more rigorous than other methods, such as the simplified Janbu method (Janbu, 1973) and the simplified Bishop method (Bishop, 1955).

Both the rotational mode (e.g., non-circular slip surfaces) and the non-rotational mode (i.e., block slip surfaces) were considered during the factor of safety assessment analyses, and the slip mode resulting in the lowest calculated FS was reported. SLIDE<sup>®</sup> generates potential slip surfaces, calculates the FS for each of these surfaces, and identifies the most critical slip surface with the lowest calculated FS.

#### **5.3.2 Seismic Slope Stability**

Pseudo-static slope stability analyses were performed utilizing Spencer's method to evaluate the seismic performance of the perimeter dike structures using a procedure consistent with a guidance document prepared for the USEPA (USEPA, 1995) and recommendations made by Hynes-Griffin and Franklin (1984). The seismic factor of safety was evaluated by applying a seismic horizontal force coefficient ( $k_h$ ) to compute an additional horizontal force ( $F = k_h \times W$ ) for each slice, based on slice weight ( $W$ ), during the design seismic event. The  $k_h$  for each evaluated cross section was developed from the Maximum Horizontal Equivalent Acceleration (MHEA) computed during the site response analysis (Section 4) at the depth of the anticipated critical slip surface for each cross section. The  $k_h$  value is dependent on the allowable displacement ( $u$ ) for a dike structure. For the purpose of this Safety Factor Assessment Report, the allowable displacement of Ash Pond A perimeter dike structures was selected as 12 inches. Based on this allowable displacement and the upper bound relation, the Hynes-Griffin and Franklin (1984) procedure was used to adjust the MHEA at the target depth to compute the  $k_h$  applied in SLIDE<sup>®</sup>, resulting in  $k_h = 0.025g$ .

#### **5.4 Static Safety Factor – Maximum Normal Storage Pool**

§257.73(e)(1)(i) requires that the static factor of safety meets or exceeds 1.50 for the maximum normal storage pool conditions within the surface impoundment. The static safety factor was evaluated for the critical cross section of Ash Pond A as shown on Figure 2 of Attachment 3.

#### **5.5 Static Safety Factor – Maximum Surcharge Pool**

§257.73(e)(1)(ii) requires that the static factor of safety meets or exceeds 1.40 for the maximum surcharge pool conditions within the surface impoundment. The static safety factor was evaluated for the critical cross section of Ash Pond A as shown on Figure 3 of Attachment 3.

#### **5.6 Seismic Safety Factor – Maximum Normal Storage Pool**

§257.73(e)(1)(iii) requires that the seismic factor of safety meets or exceeds 1.00 for the maximum normal storage pool conditions within the surface impoundment. The seismic safety factor was evaluated for the critical cross section with the computed seismic horizontal force coefficient (i.e., 0.025g) applied to each slice within SLIDE<sup>®</sup>. During the evaluation of the seismic safety factor, soil shear strengths for cohesive soils were conservatively reduced by 20% to account for the influence of cyclic degradation (Hynes-Griffin and Franklin, 1984).

#### **5.7 Summary of Results**

As presented in Table 3, the calculated factors of safety for the static case with the maximum normal storage pool, the static case with the maximum surcharge pool, and seismic case with the maximum normal storage pool are 2.47, 2.45, and 1.50, respectively. These analysis results indicate that the perimeter dikes of Ash Pond A at WGS satisfy the periodic safety factor assessment criteria given in §257.73(e)(1) of the CCR Rule. Further details of the safety factor assessment for Ash Pond A can be found in Attachment 3.

### **6. SUMMARY AND GENERAL CONCLUSIONS**

The following provides a summary and general conclusion of the safety factor assessments presented in this Safety Factor Assessment Report:

- The seismic hazard evaluation resulted in the selection of the design PGA as 0.164 g at the Site. This PGA corresponds to a seismic event with a two percent probability of exceedance in 15 years, established conservatively with consideration of the remaining operating life of Ash Pond A in a consistent manner with the return period specified in the CCR Rule. Also, this PGA represents a peak ground motion corresponding to “geologically realistic” conditions. The site response analyses were performed to compute the maximum cyclic shear stresses and MHEAs, which were applied to evaluate the liquefaction potential analyses and seismic safety factors of the Ash Pond A perimeter dikes, respectively.

- Liquefaction potential analysis was performed based on the Simplified Procedure recommended by Seed and Idriss (1971) and an update by Boulanger and Idriss (2014). The  $FS_{liq}$  was computed as the ratio of CRR to CSR and indicated that dike fill and foundation soils are not found to be susceptible to liquefaction under the design earthquake event. Therefore, the liquefaction factor of safety is not required and is not evaluated as part of this Safety Factor Assessment.

Based on the safety factor assessment of the critical cross section of the Ash Pond A perimeter dikes, Ash Pond A satisfies the required safety factors presented in §257.73(e)(1).

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# TABLES

**Table 1. Comparison of MHEA and  $k_h$  Values at the Critical Slip Surface Depth using Chart**

| <b>Location</b> | <b>2021 Analysis</b> |                         | <b>2026 Analysis</b> |                         |
|-----------------|----------------------|-------------------------|----------------------|-------------------------|
|                 | <b>MHEA (g)</b>      | <b><math>k_h</math></b> | <b>MHEA (g)</b>      | <b><math>k_h</math></b> |
| Ash Pond A      | 0.053                | 0.027                   | 0.050                | 0.025                   |

MHEA: maximum horizontal equivalent acceleration

**Table 2. Summary of Seismic Displacements Calculated using the Bray and Macedo Methodology**

| <b>Location</b> | <b>Section</b> | <b>Yield Coefficient (<math>k_y</math>)</b> | <b>Displacement (in)</b> |
|-----------------|----------------|---------------------------------------------|--------------------------|
| Ash Pond A      | Section E      | 0.12                                        | 3.5                      |

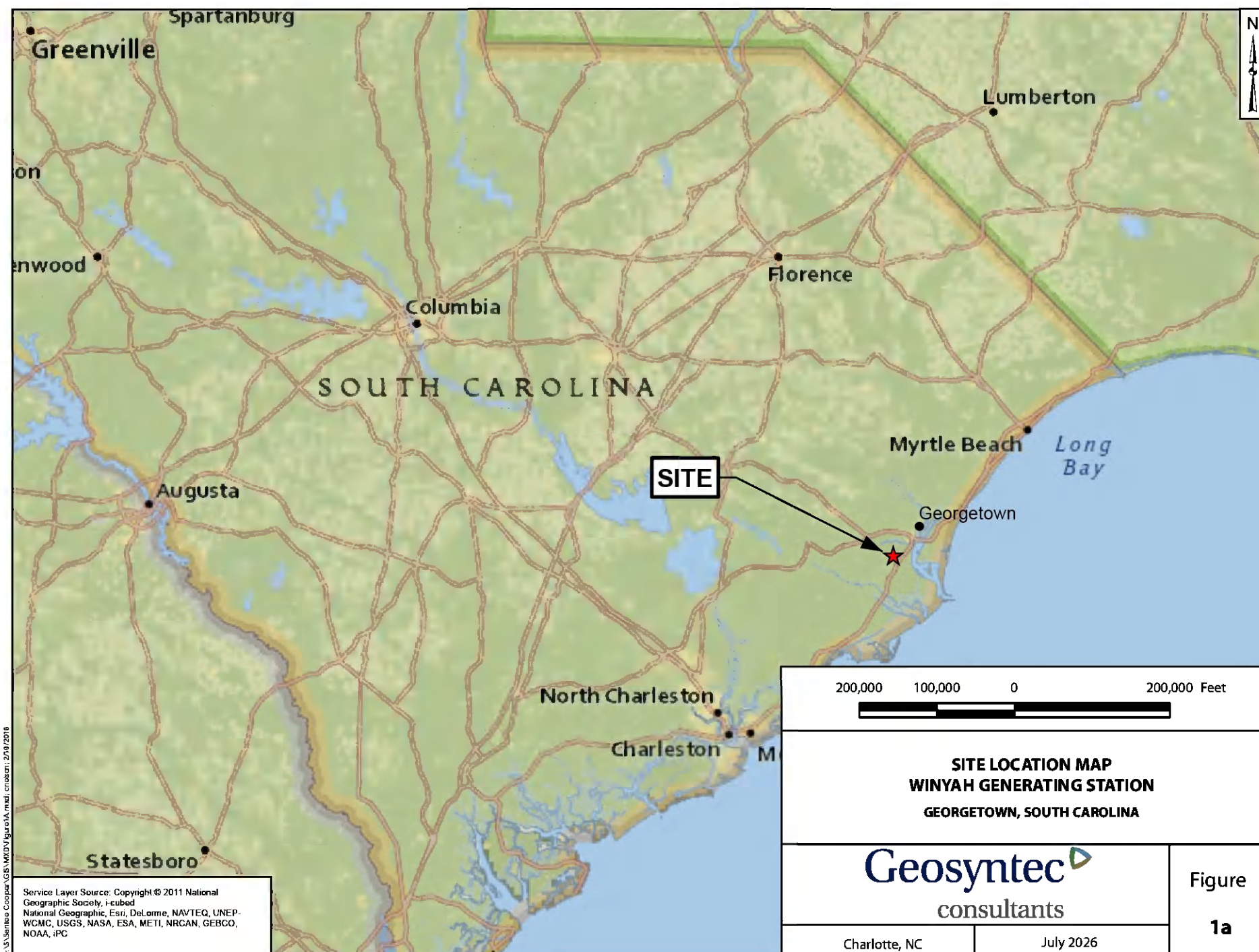
**Table 3. Summary of Safety Factor Analysis Results**

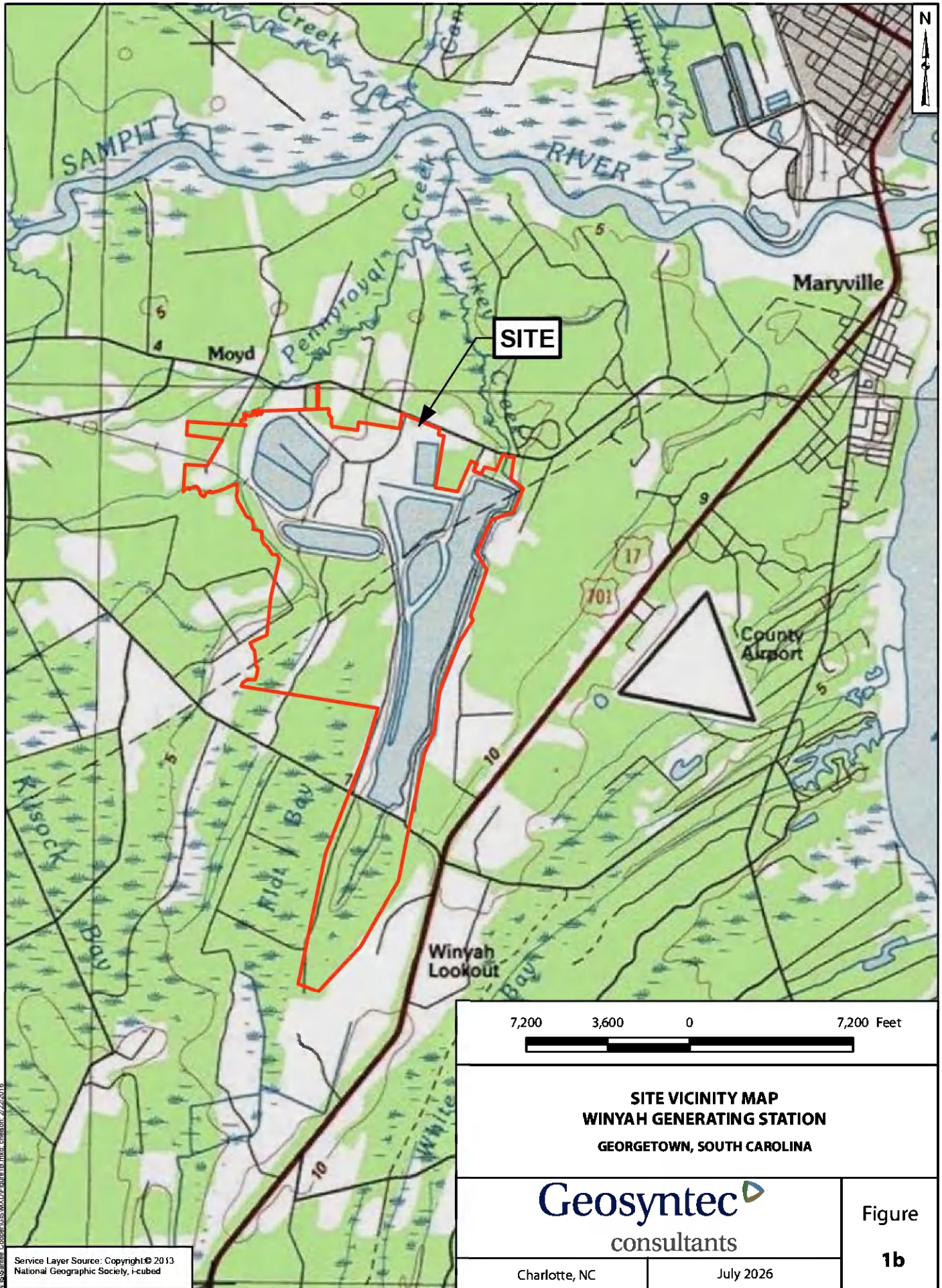
| <b>Safety Factor Case</b>             | <b>Target FS</b> | <b>Cross Section E<sup>[2]</sup></b> |
|---------------------------------------|------------------|--------------------------------------|
| Static - Maximum Normal Storage Pool  | 1.50             | <i>2.47</i>                          |
| Static - Maximum Surge Pool           | 1.40             | <i>2.45</i>                          |
| Seismic - Maximum Normal Storage Pool | 1.00             | <i>1.50</i>                          |
| Liquefaction <sup>[1]</sup>           | 1.20             | Not Applicable                       |

Notes:

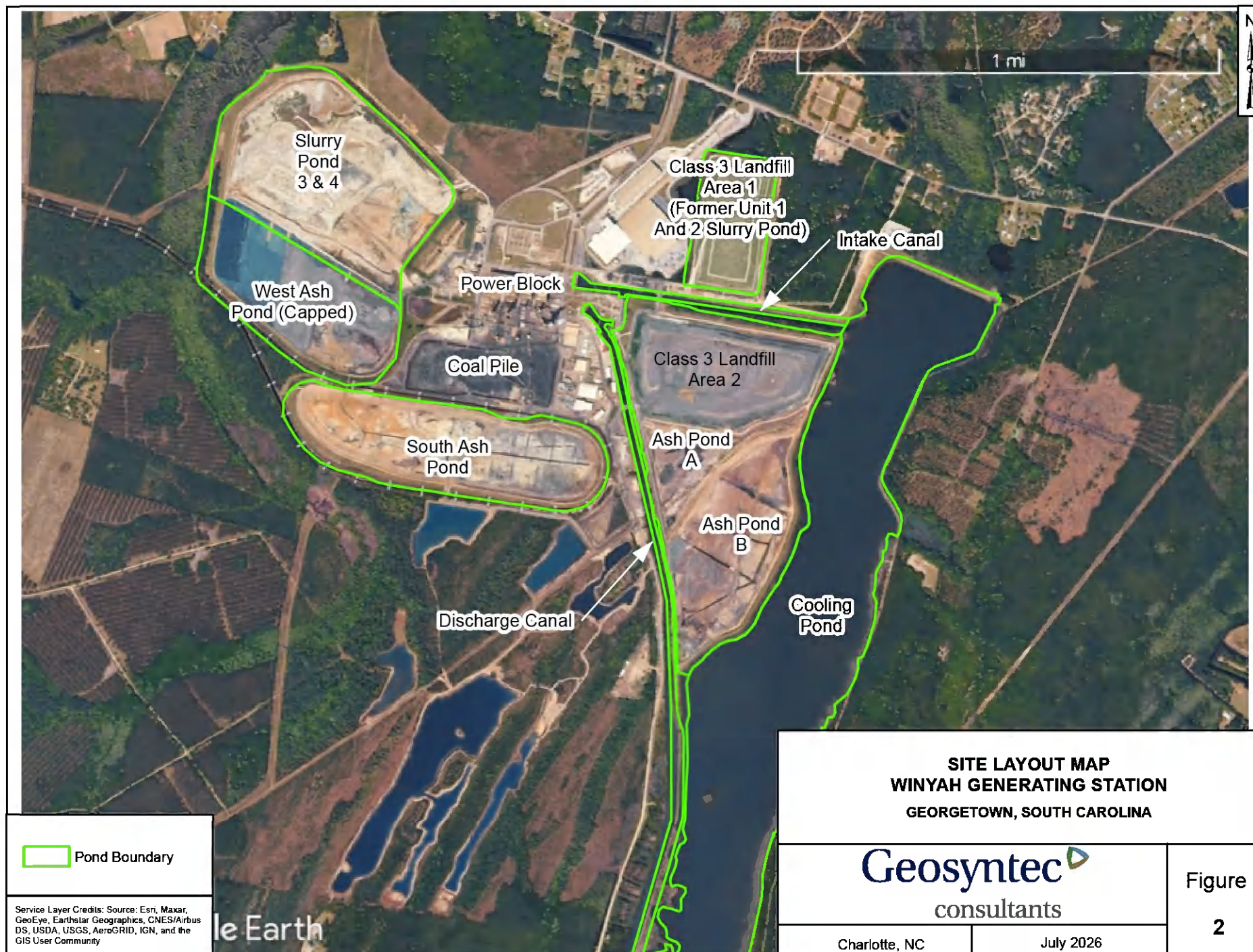
1. The liquefaction safety factor was not evaluated since dike fill soils and foundation soils beneath the dike fill were found to be non-liquefiable.
2. Only critical failure surfaces passing through the perimeter dikes were considered.

## **FIGURES**





Service Layer Source: Copyright © 2013  
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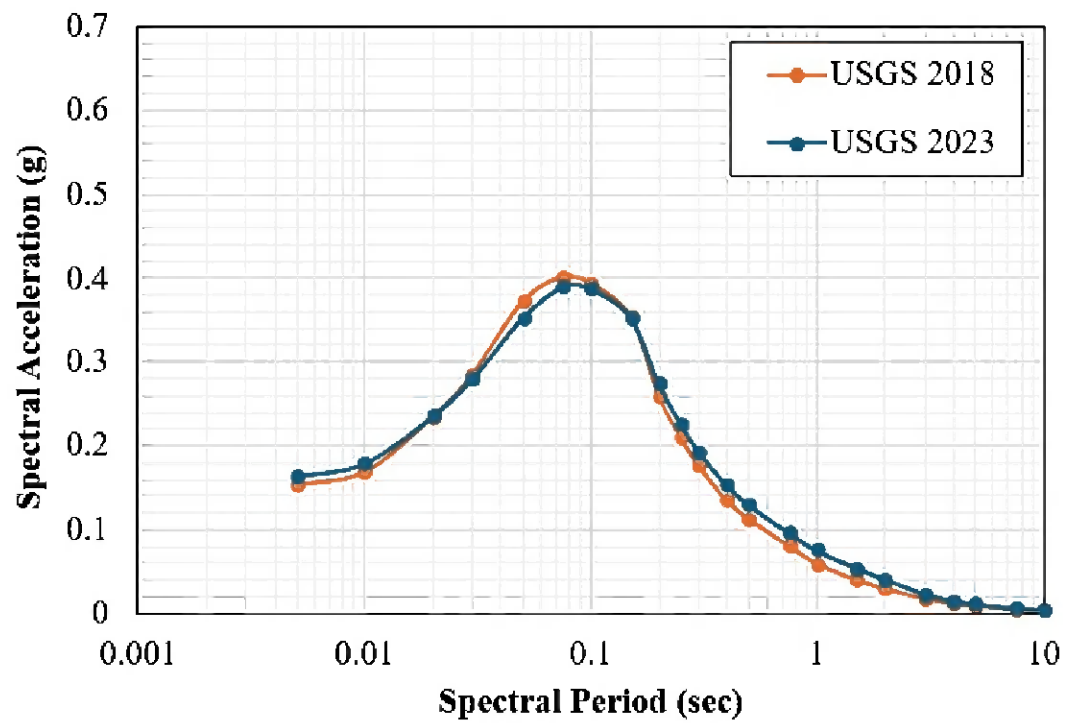


**Figure 3. Target Response Spectrum Comparison for WGS**

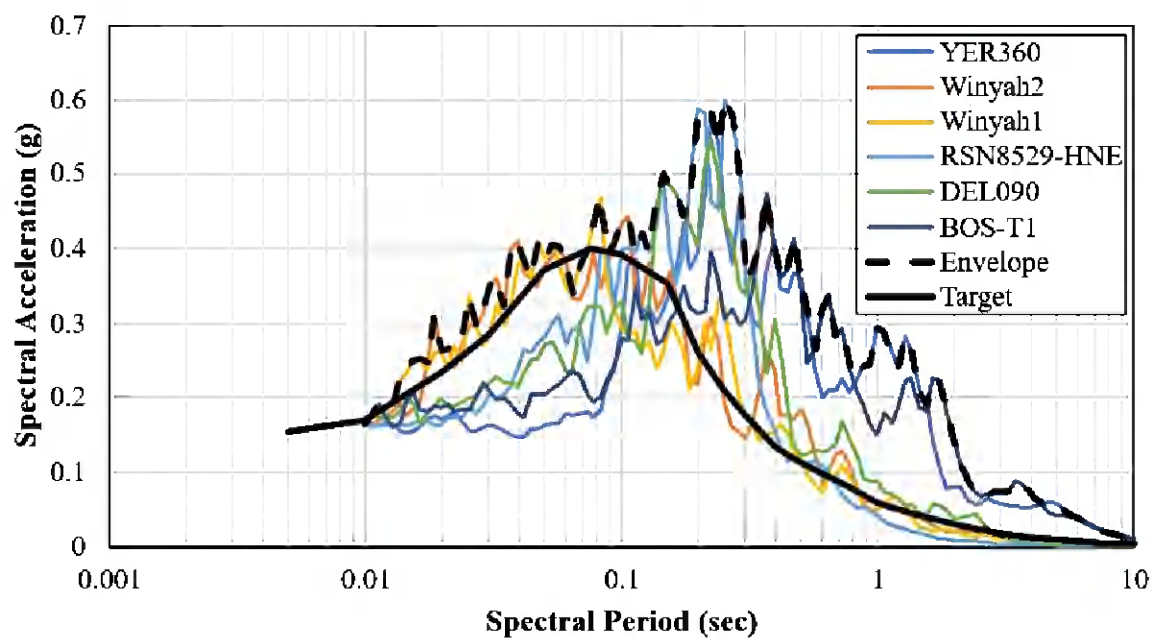


**Figure 4. Input Motions Scaled to 2018 Target PGA**

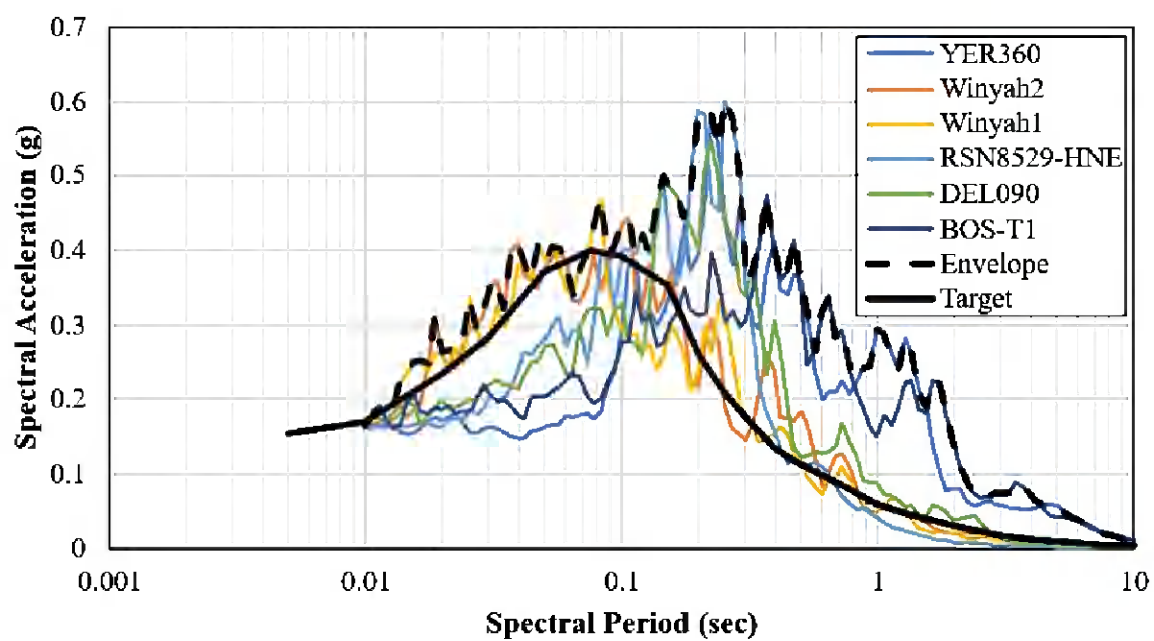


**Figure 5. Input Motions Scaled to 2023 Target PGA**

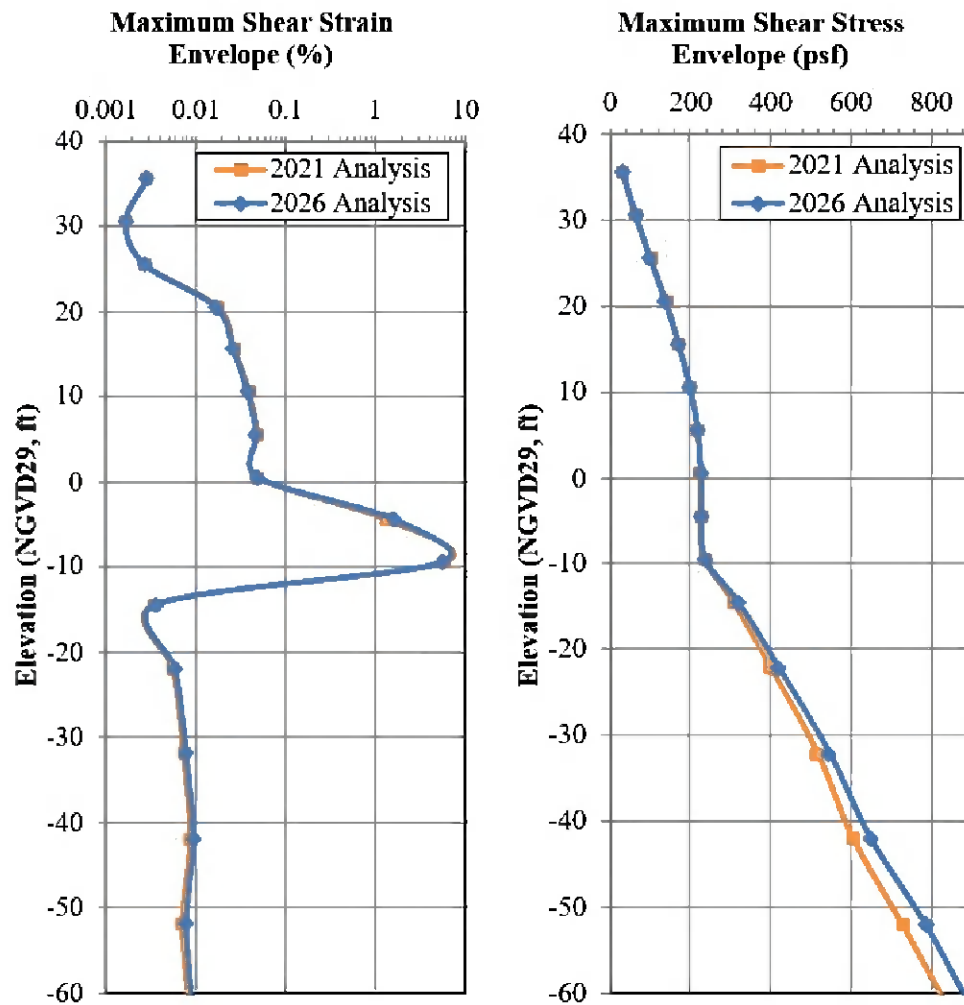


**Figure 6. Comparison of Maximum Shear Strain and Shear Stress Envelopes at Ash Pond**

**A**

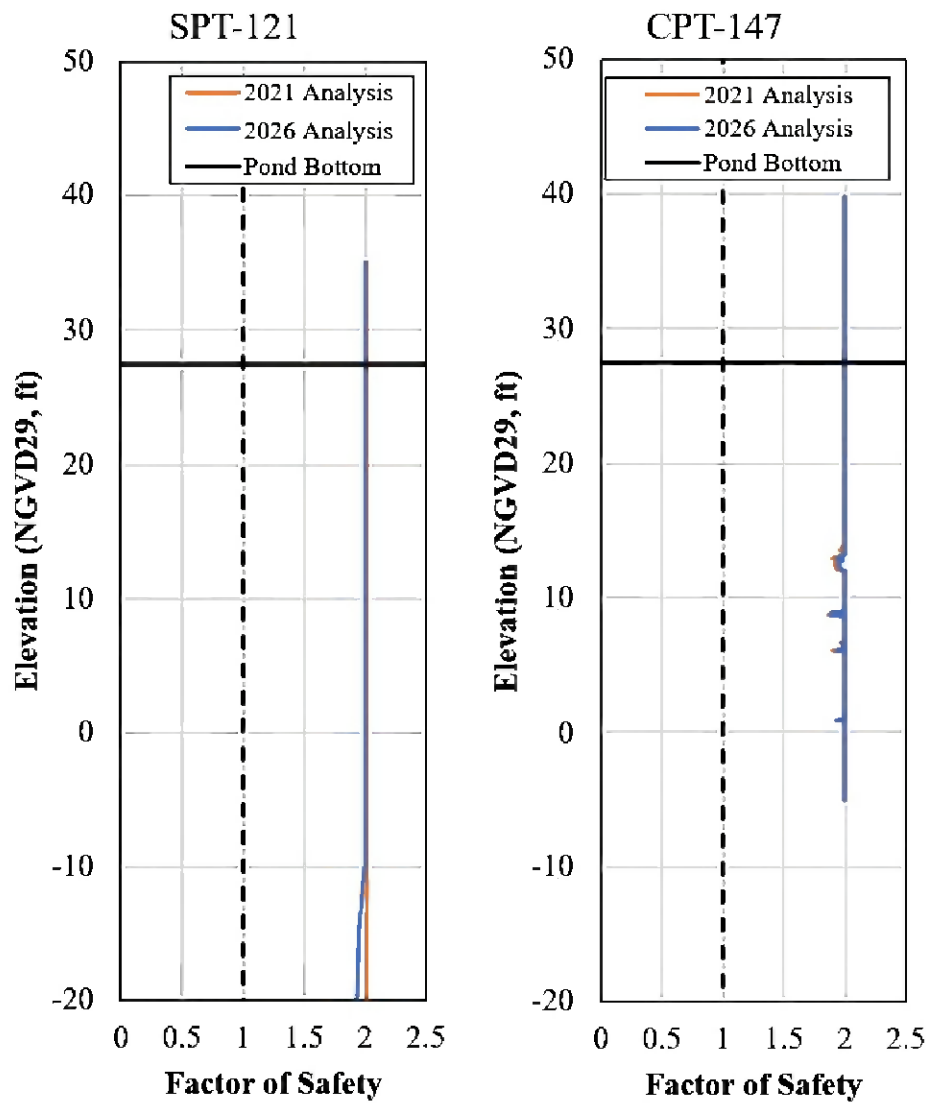


**Figure 7. Comparison of Liquefaction Results for SPT-121 and CPT-147 at Ash Pond A**

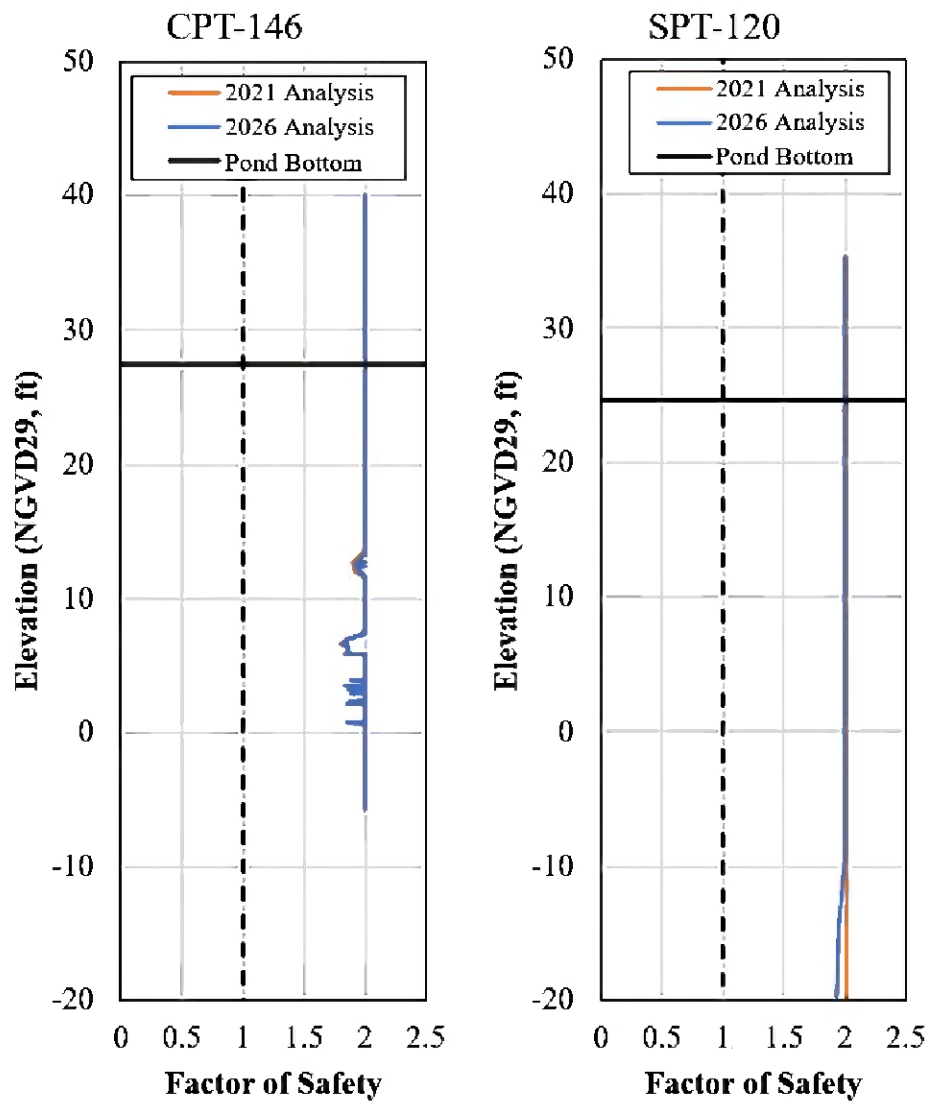


**Figure 8. Comparison of Liquefaction Results for CPT-146 and SPT-120 at Ash Pond A**

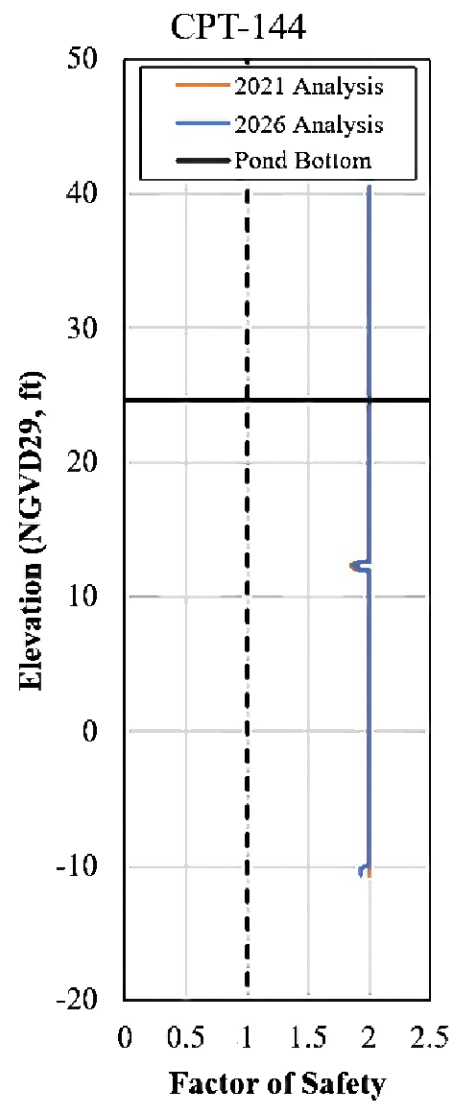


**Figure 9. Comparison of Liquefaction Results for CPT-144 at Ash Pond A**

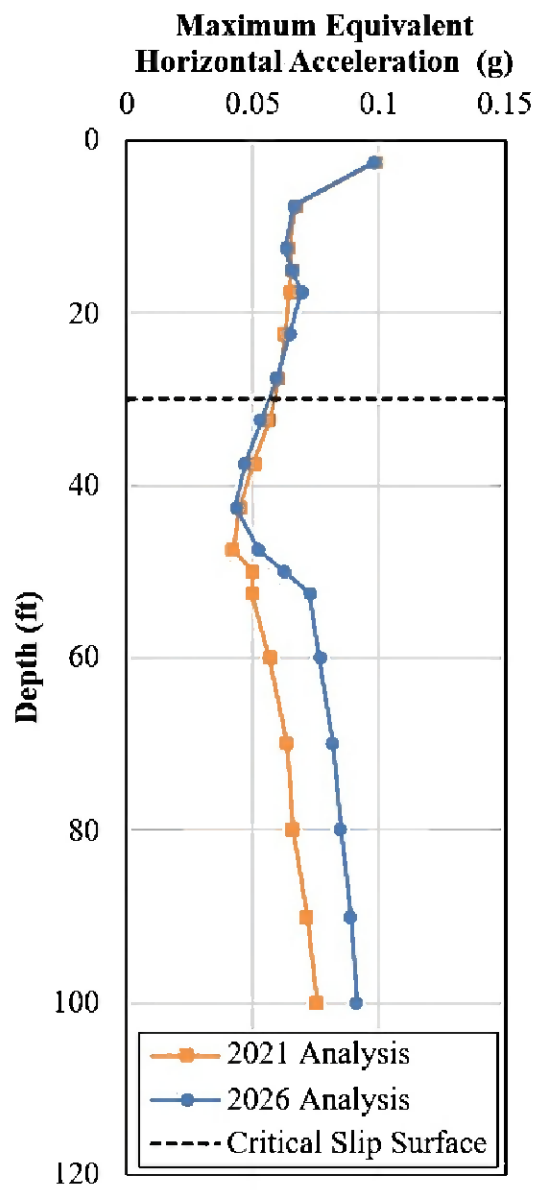
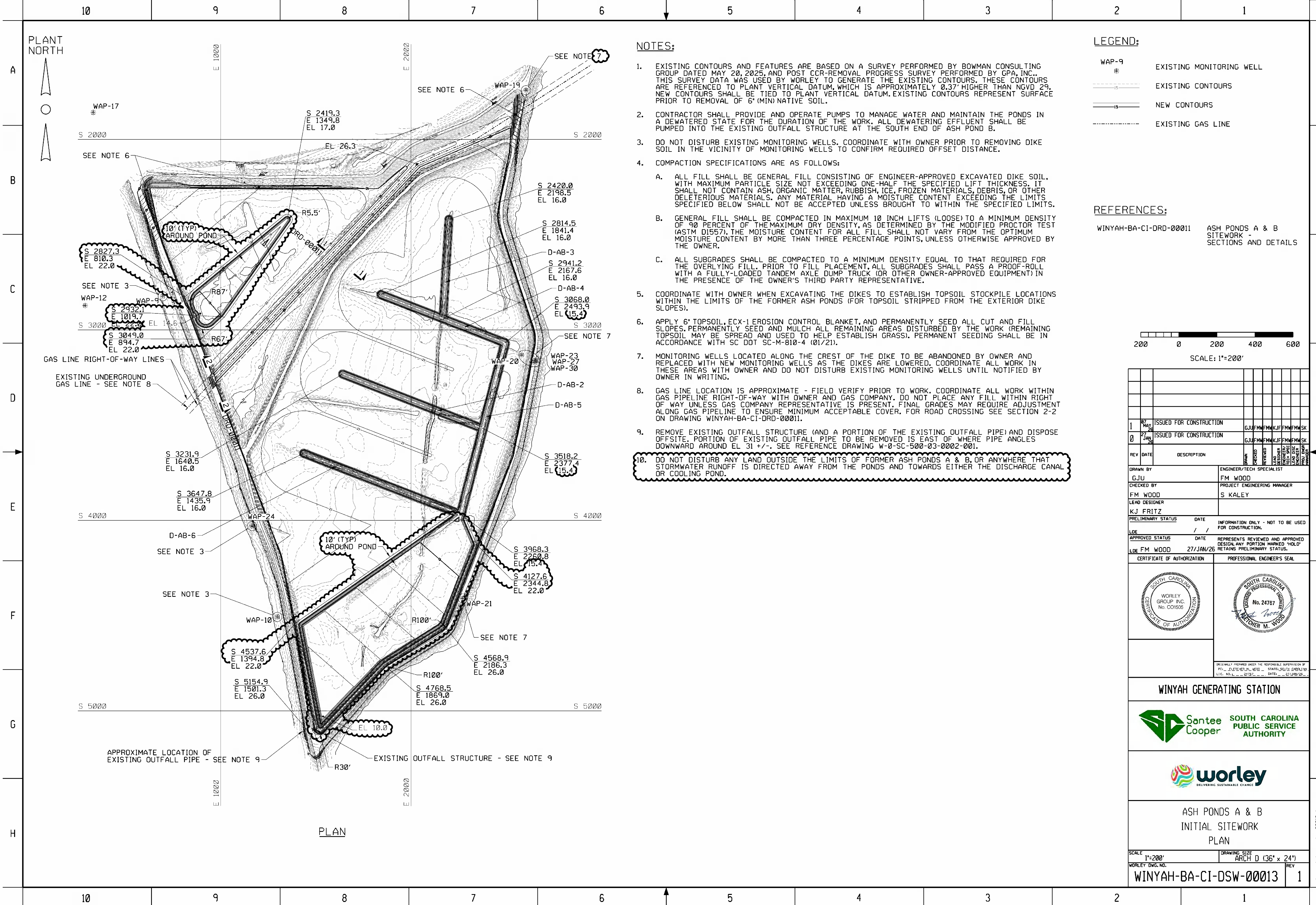


**Figure 10. Comparison of MHEA at Ash Pond A**

## **ATTACHMENTS**

**ATTACHMENT 1**  
**TOPOGRAPHIC SURVEY (JANUARY 2026)**



**ATTACHMENT 2**  
**HYDROLOGIC AND HYDRAULIC**  
**ANALYSIS RESULTS**

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Written by: A. Brand Date: 09/08/26 Reviewed by: S. Herr Date: 09/08/26  
 Client: Santee Cooper Project: WGS AP A H&H Project No.: 26009458 Phase No.: 04

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## HYDROLOGIC AND HYDRAULIC ANALYSIS WINYAH GENERATING STATION, ASH POND A, GEORGETOWN, SOUTH CAROLINA

### 1 PURPOSE AND BACKGROUND

Winyah Generating Station (WGS or the Site) is a 1,260-megawatt coal-fired electric generating facility owned and operated by the South Carolina Public Service Authority (Santee Cooper). The Site is situated between Pennyroyal and Turkey Creeks and is located in Georgetown, South Carolina. Historically, coal combustion residuals (CCR) generated at WGS have been managed in existing CCR surface impoundments.

The purpose of this computation package is to evaluate the hydraulic capacity of Ash Pond A to support spillway capacity assessments, static factor of safety analyses, and hazard rankings required by the United States Environmental Protection Agency's (USEPA's) Coal Combustion Residuals (CCR) Rule. Ash Pond A is regulated by the CCR Rule as an existing CCR surface impoundment. Under the CCR Rule, a low hazard ranking classification is associated with the 100-year (yr) flood. Since Ash Pond A is a low hazard surface impoundment, the 100-yr flood is analyzed herein.

Ash Pond A, encompassing approximately 26.3 acres (ac), is located southeast of the power block. Ash Pond A is bounded by Landfill Area 2 to the north, the Cooling Pond to the east, Ash Pond B to the south, and the Discharge Canal to the west. Ash Pond A is separated from Ash Pond B by a divider dike, which traverses from west to east from the Discharge Canal to the Cooling Pond. Ash Pond A is bounded by perimeter dikes which are approximately 20 feet (ft) to 24.5 ft in height to the east and 12 ft to 15 ft in height to the north (Worley, 2026). **Figure 1** depicts the boundary and hydraulic features associated with Ash Pond A.

Historically, Ash Pond A decanted ash sluice water, low volume wastewater, and Unit 2 Slurry Pond stormwater to Ash Pond B. Presently, Ash Pond A only receives stormwater inflows from the immediate pond area. Since the 2021 assessment, the three culverts connecting Ash Ponds A and B have been abandoned, and the former emergency spillway connecting Ash Ponds A and B has been backfilled. As a result, Ash Ponds A and B are no longer hydraulically connected.

Ash Pond A does not currently hold ponded water, as dewatering activities are underway to facilitate the closure of the surface impoundment. However, for this analysis, dewatering activities are assumed to be non-operational due to temporary loss of power during a large storm event.

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Written by: A. Brand Date: 09/08/26 Reviewed by: S. Herr Date: 09/08/26  
 Client: Santee Cooper Project: WGS AP A H&H Project No.: 26009458 Phase No.: 04

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## 2 METHODOLOGY AND INPUT PARAMETERS

Stormwater runoff volumes and associated discharges to Ash Pond A were modeled using *HydroCAD Version 10.20* software (HydroCAD, 2024). *HydroCAD* utilizes frequency-based precipitation events, in conjunction with watershed properties, to calculate peak runoff by several accepted methods. The Soil Conservation Service (SCS) Technical Release 20 (TR-20) method was applied in *HydroCAD* to calculate stormwater runoff volumes (SCS, 1982).

The following parameters and assumptions were selected for calculating stormwater runoff volumes to Ash Pond A.

### Rainfall

The 72-hour (hr) duration precipitation event was used in this analysis. The rainfall depth corresponding to the 72-hr duration precipitation event for the 100-yr flood for the Site is 12.8 inches (in.) (NOAA, 2021). The design storm hydrograph was developed using SCS Type III rainfall distribution and was directly input to the *HydroCAD* model.

### Drainage Areas and Curve Numbers

The contributing watershed area for Ash Pond A is 26.3 ac. The area was delineated using the dike crests to correspond to the pond's direct drainage area. The pond was assigned a curve number (CN) based on guidance provided in Technical Release 55 (TR-55) (SCS, 1986) representing the type of ground cover in that area. Ash Pond A was assumed to be 100% ash (CN = 86). The contributing watershed area and CN is summarized in **Table 1** and was directly input into the *HydroCAD* model.

### Times of Concentration

The time of concentration represents the time required for runoff to flow from the most hydraulically remote point of the drainage area to the point under investigation. The flow path from the most remote point within Ash Pond A is characterized by sheet flow and shallow concentrated flow (shown in **Figure 2**). *HydroCAD* applied the Overton and Meadows formulation to calculate travel time for sheet flow for distances less than 300 ft (NRCS, 2010):

$$T_t = \frac{0.007(nL)^{0.8}}{P_{2-24}^{0.5} S^{0.4}}$$

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where:  $T_t$  = travel time for overland sheet flow (hr);  
 $n$  = Manning's roughness coefficient for sheet flow (--);  
 $L$  = flow length (ft);  
 $P_{2-24}$  = 2-yr, 24-hr rainfall (in.); and  
 $S$  = slope of hydraulic grade line (or land slope) (ft/ft).

A Manning's roughness coefficient of 0.020 was used to represent sheet flow in Ash Pond A. The sheet flow length was limited to 100 ft, because sheet flow beyond 100 ft typically transitions to shallow concentrated flow. The rainfall depth for the 2-yr, 24-hr frequency storm event is 4.42 in. (NOAA, 2021). The parameters used to model sheet flow within Ash Pond A are shown in **Table 2**.

Shallow concentrated flow travel time was computed using the Upland Method (NRCS, 2010).

$$T_t = \frac{L}{V}$$

where:  $T_t$  = travel time (s);  
 $L$  = flow length (ft); and  
 $V$  = average velocity (ft/s).

The average velocity was computed using the following equation (NRCS, 2010).

$$V = K_v S^{0.5}$$

where:  $V$  = average velocity (ft/s);  
 $K_v$  = velocity factor (ft/s); and  
 $S$  = slope of hydraulic grade line (or land slope) (ft/ft).

A velocity factor of 16.1 ft/s, representing flow across an unpaved surface, was used to calculate shallow concentrated flow travel time within Ash Pond A. The parameters used to describe shallow concentrated flow within Ash Pond A are presented in **Table 2**.

The resulting times of concentration for sheet flow and shallow concentrated flow are presented in **Table 2**.

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## Inflows

Stormwater inflow generated from Ash Pond A is modeled as Subcatchment 1S. Stormwater inflow generated from Subcatchment 1S flows into Pond 1P (Ash Pond A). The *HydroCAD* model routing diagram is provided in **Attachment A**.

## Storage Capacities

The available stormwater storage volume in Ash Pond A between elevations 22 ft and 36 ft (Plant Vertical Datum) was calculated by developing an area-volume curve based on topographic data (Worley, 2026). The lowest available contour within Ash Pond A is 22 ft (Plant Vertical Datum). The minimum crest elevation of the Ash Pond A perimeter dikes is 36 ft (Plant Vertical Datum). The surface area of each contour was measured and tabulated at each elevation. Next, the available surface water volume in each depth increment was calculated by averaging the surface area of the upper and lower contours and multiplying by the change in elevation between the contours. The cumulative storage volume of Ash Pond A between these elevations is 246.0 acre-feet (ac-ft). The area-volume data are presented in **Table 3**.

Ash Pond A does not currently hold ponded water, as dewatering activities are underway to facilitate the closure of the surface impoundment.

## Outlet Structures

For this analysis, dewatering activities are assumed to be non-operational due to temporary loss of power during a large storm event.

## **3 RESULTS OF ANALYSES**

The resulting peak water surface elevation and water volume for the 100-yr flood are shown in **Table 4**. Ash Pond A will effectively contain the 100-yr flood and maintain a freeboard of 16.6 ft. This hydrologic and hydraulic analysis demonstrates that Ash Pond A contains the 100-yr flood assuming Ash Pond A does not hold ponded water.

## **4 REFERENCES**

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NOAA. (2021). *Precipitation-Frequency Atlas of the United States*. Atlas 14, Volume 2, Version 3.0.

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Written by: A. Brand Date: 09/08/26 Reviewed by: S. Herr Date: 09/08/26  
Client: Santee Cooper Project: WGS AP A H&H Project No.: 26009458 Phase No.: 04

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NRCS. (2010). *National Engineering Handbook: Part 630 Hydrology*.

SCS. (1982). *Technical Release Number 20 (TR-20)*. National Technical Information Service.

SCS. (1986). *Technical Release Number 55 (TR-55)*. National Technical Information Service.

Worley Group, Inc. (2026). *Ash Ponds A & B Initial Sitework Plan*

## TABLES

**Table 1. Land Use Conditions**

| <b>Drainage Area<br/>ID</b> | <b>Land Use<br/>Description</b> | <b>CN</b> | <b>Area<br/>(acres)</b> | <b>Total<br/>Area<br/>(acres)</b> | <b>Weighted<br/>CN</b> |
|-----------------------------|---------------------------------|-----------|-------------------------|-----------------------------------|------------------------|
| Ash Pond A                  | CCR                             | 86        | 26.3                    | 26.3                              | 86                     |

Table 2. Time of Concentration

| Drainage Area ID | Time of Concentration - Sheet Flow Contribution |                      |                            |                                      |                              |                                                               | Time of Concentration - Shallow Concentrated Flow Contribution |                           |                            |                              |                                                                            | Total Time of Concentration<br>{Tc}<br>(min) |
|------------------|-------------------------------------------------|----------------------|----------------------------|--------------------------------------|------------------------------|---------------------------------------------------------------|----------------------------------------------------------------|---------------------------|----------------------------|------------------------------|----------------------------------------------------------------------------|----------------------------------------------|
|                  | Surface Description                             | Manning's No.<br>{n} | Flow Length<br>{L}<br>(ft) | 2-Yr, 24-Hr Rainfall<br>{I}<br>(in.) | Land Slope<br>{S}<br>(ft/ft) | Time of Concentration from Sheet Flow<br>{Tc, sheet}<br>(min) | Surface Description                                            | Velocity Factor<br>(ft/s) | Flow Length<br>{L}<br>(ft) | Land Slope<br>{S}<br>(ft/ft) | Time of Concentration from Shallow Concentrated Flow<br>{Tc, scf}<br>(min) |                                              |
| Ash Pond A       | Smooth                                          | 0.020                | 100                        | 4.42                                 | 0.0588                       | 1.1                                                           | Unpaved                                                        | 16.1                      | 429                        | 0.0174                       | 3.4                                                                        | 4.5                                          |

**Table 3. Ash Pond A Stage Storage**

| <b>Elevation (ft)<br/>(Plant Vertical<br/>Datum)</b> | <b>Area<br/>(ac)</b> | <b>Incremental Volume<br/>(ac-ft)</b> | <b>Cumulative Volume<br/>(ac-ft)</b> |
|------------------------------------------------------|----------------------|---------------------------------------|--------------------------------------|
| 22                                                   | 2.9                  | 0.0                                   | 0                                    |
| 23                                                   | 6.2                  | 4.6                                   | 4.6                                  |
| 24                                                   | 9.8                  | 8.0                                   | 12.6                                 |
| 25                                                   | 13.3                 | 11.6                                  | 24.1                                 |
| 26                                                   | 15.9                 | 14.6                                  | 38.8                                 |
| 27                                                   | 17.9                 | 16.9                                  | 55.7                                 |
| 28                                                   | 19.3                 | 18.6                                  | 74.3                                 |
| 29                                                   | 20.0                 | 19.7                                  | 94.0                                 |
| 30                                                   | 20.6                 | 20.3                                  | 114.3                                |
| 31                                                   | 21.1                 | 20.9                                  | 135.1                                |
| 32                                                   | 21.6                 | 21.3                                  | 156.5                                |
| 33                                                   | 22.0                 | 21.8                                  | 178.3                                |
| 34                                                   | 22.4                 | 22.2                                  | 200.5                                |
| 35                                                   | 22.8                 | 22.6                                  | 223.1                                |
| 36                                                   | 23.1                 | 23.0                                  | 246.0                                |

**Table 4. H&H Calculation Results**

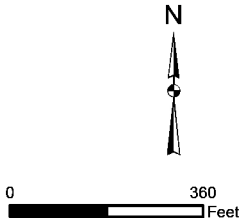
| <b>Pond ID</b> | <b>Storm Event</b> | <b>Normal<br/>Operating<br/>Elevation (ft)<br/>(Plant<br/>Vertical<br/>Datum)</b> | <b>Peak Water<br/>Surface<br/>Elevation<br/>(ft) (Plant<br/>Vertical<br/>Datum)</b> | <b>Water<br/>Volume<br/>(ac-ft)</b> | <b>Freeboard (ft)</b> |
|----------------|--------------------|-----------------------------------------------------------------------------------|-------------------------------------------------------------------------------------|-------------------------------------|-----------------------|
| Ash Pond A     | 100-Yr Flood       | Dry                                                                               | 25.0                                                                                | 24.2                                | 11.0                  |

## FIGURES

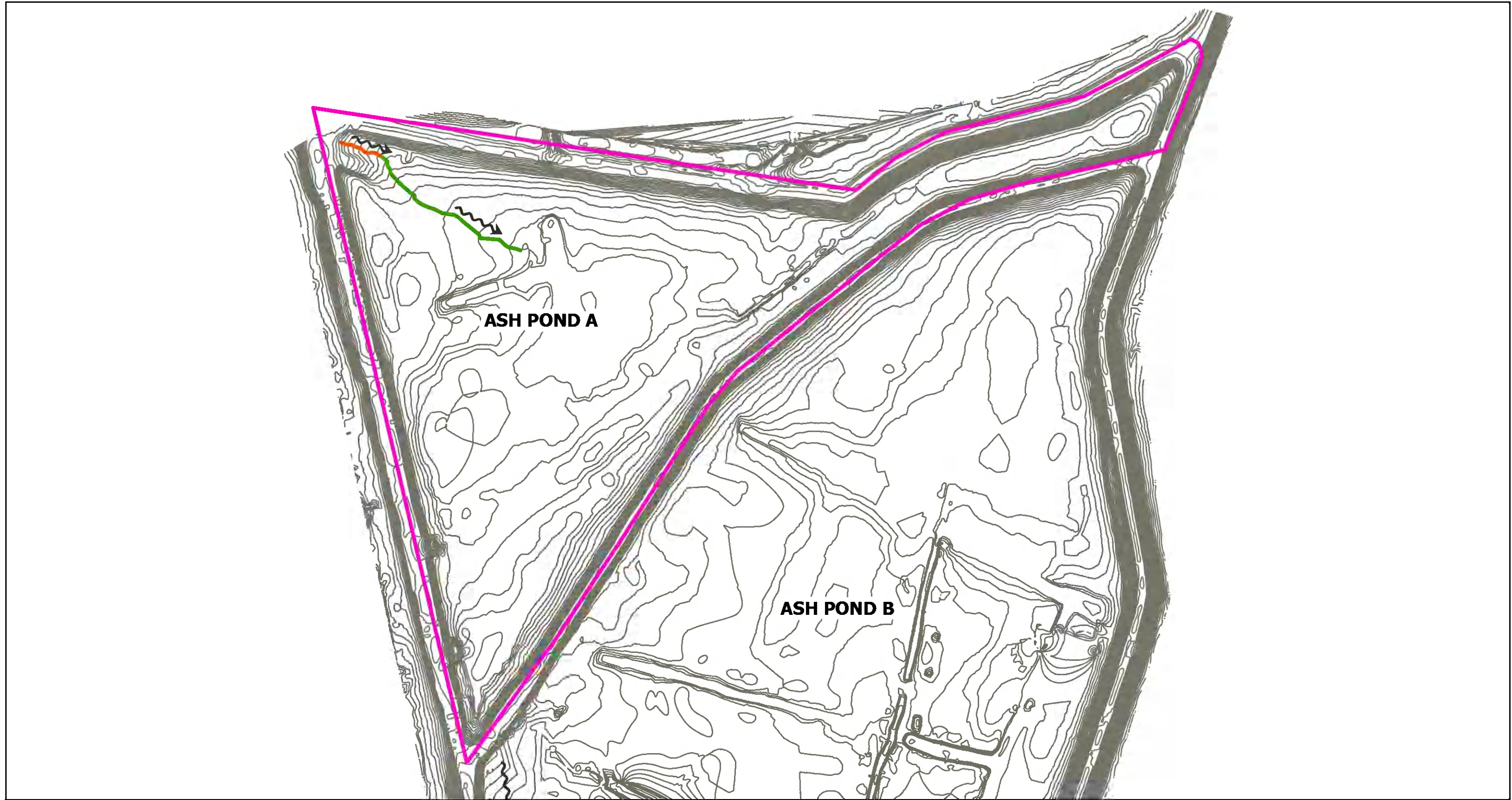


**Legend**  
Pond Boundary

Note:  
1. Aerial imagery taken from Google Earth, dated April 2025.



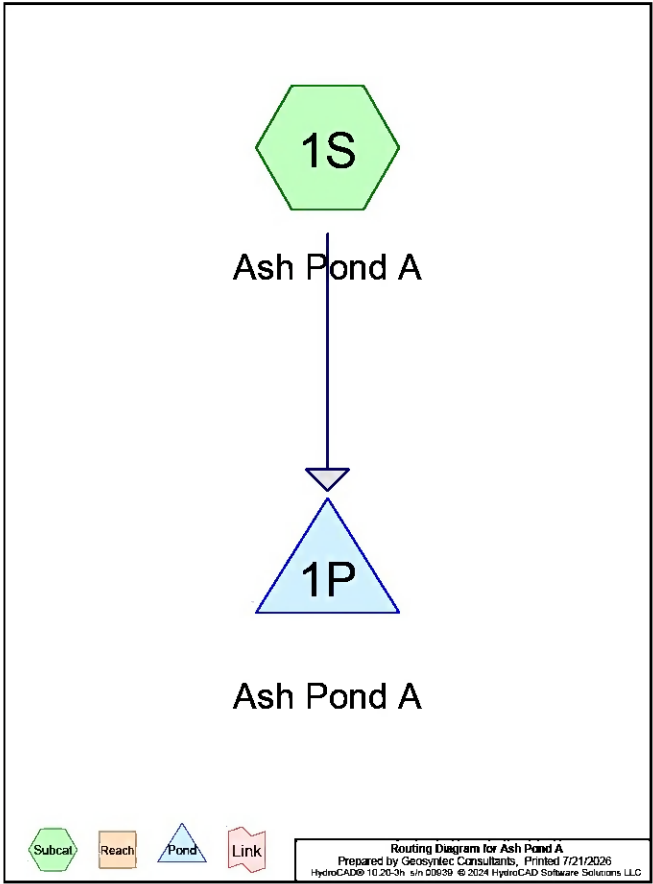
|                                         |                |
|-----------------------------------------|----------------|
| WINYAH GENERATING STATION<br>ASH POND A |                |
| Geosyntec<br>consultants                | Figure<br>1    |
| 26009458                                | September 2026 |



|                                                                                                     |                                                                                     |                                  |                           |
|-----------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------|----------------------------------|---------------------------|
| <b>Legend</b><br>● Pond Boundary<br>● Shallow Concentrated Flow<br>● Sheet Flow<br>➔ Flow Direction | <b>Note:</b><br>1. Topographic survey information by Worley Group, dated 1-27-2026. | <b>WINYAH GENERATING STATION</b> |                           |
|                                                                                                     |                                                                                     | <b>ASH POND A</b>                |                           |
|                                                                                                     |                                                                                     | <b>Geosyntec</b><br>consultants  | <b>Figure</b><br><b>2</b> |
|                                                                                                     |                                                                                     | 26009458                         | September 2026            |

## ATTACHMENTS

## ATTACHMENT A



| Area Listing (all nodes) |    |                                    |
|--------------------------|----|------------------------------------|
| Area (acres)             | CN | Description (subcatchment-numbers) |
| 26.300                   | 86 | (1S)                               |
| 26.300                   | 86 | TOTAL AREA                         |

| Rainfall Events Listing (selected events) |               |                |       |       |                  |     |                |     |
|-------------------------------------------|---------------|----------------|-------|-------|------------------|-----|----------------|-----|
| Event#                                    | Event Name    | Storm Type     | Curve | Mode  | Duration (hours) | B/S | Depth (inches) | AMC |
| 1                                         | 100-YR, 72-HR | Type III 24-hr |       | Scale | 72.00            | 1   | 12.80          | 2   |

| Soil Listing (all nodes) |            |                      |
|--------------------------|------------|----------------------|
| Area (acres)             | Soil Group | Subcatchment Numbers |
| 0.000                    | HSG A      |                      |
| 0.000                    | HSG B      |                      |
| 0.000                    | HSG C      |                      |
| 0.000                    | HSG D      |                      |
| 26.300                   | Other      | 1S                   |
| 26.300                   |            | TOTAL AREA           |

| Ground Covers (all nodes) |                  |                  |                  |                  |                  |                 |                         |
|---------------------------|------------------|------------------|------------------|------------------|------------------|-----------------|-------------------------|
| HSG-A<br>(acres)          | HSG-B<br>(acres) | HSG-C<br>(acres) | HSG-D<br>(acres) | Other<br>(acres) | Total<br>(acres) | Ground<br>Cover | Subcatchment<br>Numbers |
| 0.000                     | 0.000            | 0.000            | 0.000            | 26.300           | 26.300           |                 | 1S                      |
| 0.000                     | 0.000            | 0.000            | 0.000            | 26.300           | 26.300           | TOTAL AREA      |                         |

Time span=0.00-900.00 hrs, dt=0.01 hrs, 90001 points

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN

Reach routing by Sim-Route method - Pond routing by Sim-Route method

Subcatchment1S: Ash Pond A

Runoff Area=26.300 ac 0.00% Impervious Runoff Depth=11.03"

Flow Length=529' Tc=4.5 min CN=86 Runoff=124.07 cfs 24.184 af

Pond 1P: Ash Pond A

Peak Elev=25.00' Storage=24.183 af Inflow=124.07 cfs 24.184 af

Outflow=0.00 cfs 0.000 af

Total Runoff Area = 26.300 ac Runoff Volume = 24.184 af Average Runoff Depth = 11.03"

100.00% Pervious = 26.300 ac 0.00% Impervious = 0.000 ac

Summary for Subcatchment 1S: Ash Pond A

Runoff = 124.07 cfs @ 36.06 hrs, Volume= 24.184 af, Depth=11.03"

Routed to Pond 1P: Ash Pond A

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-900.00 hrs, dt= 0.01 hrs

Type III 24-hr scaled to 72.00 hrs 100-YR, 72-HR Rainfall=12.80"

| Area (ac) | CN | Description           |
|-----------|----|-----------------------|
| 26.300    | 86 |                       |
| 26.300    |    | 100.00% Pervious Area |

| Tc (min) | Length (feet) | Slope (ft/ft) | Velocity (ft/sec) | Capacity (cfs) | Description                                          |
|----------|---------------|---------------|-------------------|----------------|------------------------------------------------------|
| 1.1      | 100           | 0.0588        | 1.54              |                | Sheet Flow, Sheet Flow                               |
|          |               |               |                   |                | n= 0.020 P2= 4.42"                                   |
| 3.4      | 429           | 0.0174        | 2.12              |                | Shallow Concentrated Flow, Shallow Concentrated Flow |
|          |               |               |                   |                | Unpaved Ky= 16.1 fps                                 |
| 4.5      | 529           | Total         |                   |                |                                                      |

Summary for Pond 1P: Ash Pond A

Inflow Area = 26.300 ac, 0.00% Impervious, Inflow Depth = 11.03" for 100-YR, 72-HR event

Inflow = 124.07 cfs @ 36.06 hrs, Volume= 24.184 af

Outflow = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af, Atten= 100%, Lag= 0.0 min

Routing by Sim-Route method, Time Span= 0.00-900.00 hrs, dt= 0.01 hrs

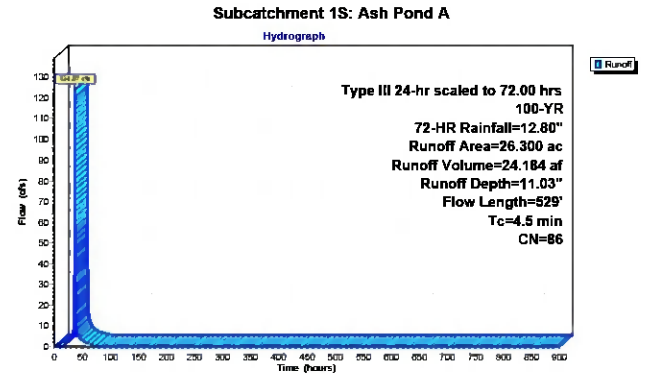
Peak Elev= 25.00' @ 72.26 hrs Surf.Area= 13.349 ac Storage= 24.183 af

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)

Center-of-Mass del. time= (not calculated: no outflow)

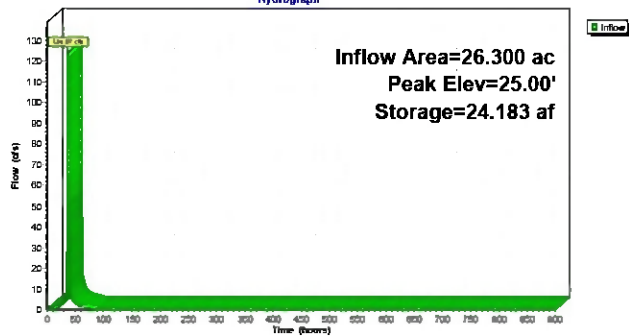
| Volume | Invert | Avail.Storage | Storage Description                                 |
|--------|--------|---------------|-----------------------------------------------------|
| #1     | 22.00' | 246.025 af    | Custom Stage Data (Prismatic) listed below (Recalc) |

| Elevation (feet) | Surf.Area (acres) | Inc.Store (acre-feet) | Cum.Store (acre-feet) |
|------------------|-------------------|-----------------------|-----------------------|
| 22.00            | 2.911             | 0.000                 | 0.000                 |
| 23.00            | 6.195             | 4.553                 | 4.553                 |
| 24.00            | 9.809             | 8.002                 | 12.555                |
| 25.00            | 13.338            | 11.574                | 24.128                |
| 26.00            | 15.916            | 14.627                | 38.756                |
| 27.00            | 17.919            | 16.918                | 55.673                |
| 28.00            | 19.317            | 18.618                | 74.291                |
| 29.00            | 20.037            | 19.677                | 93.968                |
| 30.00            | 20.614            | 20.325                | 114.294               |
| 31.00            | 21.089            | 20.851                | 135.145               |
| 32.00            | 21.577            | 21.333                | 156.478               |
| 33.00            | 21.983            | 21.780                | 178.258               |
| 34.00            | 22.412            | 22.197                | 200.455               |
| 35.00            | 22.818            | 22.615                | 223.071               |
| 36.00            | 23.091            | 22.954                | 246.025               |



**Pond 1P: Ash Pond A**

Hydrograph



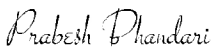
**ATTACHMENT 3**  
**SAFETY FACTOR ASSESSMENT**

## CALCULATION PACKAGE COVER SHEET

Client: Santee Cooper Project: Winyah Generating Station Project No.: 26009458  
Phase No.: 003

Title of Package: SAFETY FACTOR ASSESSMENT: ASH POND A

Computations by: Signature   9-18-2026  
Printed Name Sahul Kharel / Sachini Madanayake Date  
Title Staff Professional / Engineer

Assumptions and Procedures Checked by: Signature  9-18-2026  
Printed Name Prabesh Bhandari Date  
(peer reviewer) Title Project Engineer

Computations Checked by: Signature   9-18-2026  
Printed Name Sahul Kharel / Sachini Madanayake Date  
Title Staff Professional / Engineer

Computations Backchecked by: Signature   9-18-2026  
(originator) Printed Name Sahul Kharel / Sachini Madanayake Date  
Title Staff Professional / Engineer

Approved by: Signature  9-18-2026  
(pm or designate) Printed Name E. Onur Tastan Date  
Title Principal

Approval notes: \_\_\_\_\_

Revisions (number and initial all revisions)

| No.   | Sheet | Date  | By    | Checked by | Approval |
|-------|-------|-------|-------|------------|----------|
| _____ | _____ | _____ | _____ | _____      | _____    |
| _____ | _____ | _____ | _____ | _____      | _____    |
| _____ | _____ | _____ | _____ | _____      | _____    |

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Written by: S. Kharel / S. Madanayake Date: 9-18-2026 Checked by: P. Bhandari / E.O. Tastan Date: 9-18-2026  
 Client: Santee Cooper Project: Winyah Generating Station Project No.: 26009458 Phase No.: 003

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## SAFETY FACTOR ASSESSMENT: ASH POND A

### INTRODUCTION

This calculation package was prepared as Attachment 3 to the *2026 Periodic Safety Factor Assessment: Ash Pond A* (2026 Safety Factor Assessment Report) and presents the slope stability analyses for the critical portion of Ash Pond A perimeter dikes at Winyah Generating Station (WGS), Georgetown County, South Carolina. On 17 April 2015, the United States Environmental Protection Agency (USEPA) published 40 Code of Federal Regulations [CFR] Parts 257 and 261 (CCR Rule). Under the CCR Rule, Ash Pond A is classified as an “existing surface impoundment” and must meet specific requirements with respect to periodic safety factor assessments. This calculation package presents the slope stability analysis performed as part of the periodic safety factor assessment required by §257.73(e)(1) of the CCR Rule for existing CCR surface impoundments. The remainder of this calculation package presents: (i) safety factor criteria; (ii) methodology; (iii) cross section geometry; (iv) engineering parameters; (v) results; and (vi) conclusions.

### SAFETY FACTOR CRITERIA

Slope stability analyses were conducted to assess whether the Ash Pond A perimeter dikes satisfy the factor of safety (FS) criteria described within §257.73(e)(1) of the CCR Rule. Specifically, §257.73(e)(1) requires that:

- “(i) *The calculated static factor of safety under the long-term, maximum storage pool loading condition must equal or exceed 1.50.*
- “(ii) *The calculated static factor of safety under the maximum surcharge pool loading condition must equal or exceed 1.40.*
- “(iii) *The calculated seismic factor of safety must equal or exceed 1.00.*
- “(iv) *For embankments constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20.”*

It is noted that the liquefaction potential analysis results presented in Section 4 of the 2026 Safety Factor Assessment Report did not indicate that the Ash Pond A dike fill or foundation soils immediately beneath the perimeter dikes are expected to undergo triggering of liquefaction under the design earthquake. Therefore, the liquefaction FS for the Ash Pond A perimeter dikes utilizing post-liquefaction residual shear strengths was not evaluated as part of this safety factor assessment.

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## METHODOLOGY

### Static Slope Stability

Global slope stability analyses were performed using Spencer's method (Spencer, 1973), as implemented in the computer program SLIDE<sup>®</sup>, version 9.043 (Rocscience, 2026). Spencer's method, which satisfies vertical and horizontal force equilibrium as well as moment equilibrium, is considered to be more rigorous than other methods, such as the simplified Janbu method (Janbu, 1973) and the simplified Bishop method (Bishop, 1955).

Both the rotational mode and the non-rotational mode were considered for the stability analyses presented in this calculation package. SLIDE<sup>®</sup> generates potential slip surfaces, calculates the FS for each of these surfaces, and identifies the critical slip surface with the lowest calculated FS. Information required for these analyses include the slope geometry, subsurface soil stratigraphy, phreatic surface elevation, external loading conditions, and engineering properties of subsurface materials.

### Seismic Slope Stability

Pseudo-static slope stability analyses were performed to evaluate the seismic performance of the perimeter dike structures using a procedure consistent with Hynes-Griffin and Franklin (1984). The procedure is described as follows:

1. Estimate the maximum horizontal equivalent acceleration (MHEA) for the potential critical slip surfaces of the perimeter dike system based on results from the site response analyses presented in Section 4 of the 2026 Safety Factor Assessment Report.
2. Compute the seismic horizontal force coefficient ( $k_h$ ) using the ratio of the critical acceleration ( $N$ ) to the peak value of earthquake acceleration ( $A$ ) based on an allowable deformation ( $u$ ) for which the perimeter dikes are considered stable (from Figure 7 of Hynes-Griffin and Franklin [1984]). The critical acceleration,  $N$ , was selected as the  $k_h$  for the purposes of this analysis and the MHEA at the depth of the critical slip surface was selected as the peak earthquake acceleration,  $A$ .
3. Perform slope stability analysis applying the seismic horizontal force coefficient to compute a horizontal force ( $F = k_h \times W$ ), on each slice based on slice weight ( $W$ ) and evaluate the resulting FS. If the calculated FS meets or exceeds the target FS (i.e.,  $FS \geq 1.0$ ), the slope is expected to experience less deformation than the selected allowable displacement and meet the requirements of the CCR Rule.

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|             |                                  |          |                                  |              |                                  |            |                  |
|-------------|----------------------------------|----------|----------------------------------|--------------|----------------------------------|------------|------------------|
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It is noted that during pseudo-static slope stability analyses, undrained shear strengths were conservatively reduced by 20 percent to account for potential strength degradation during cyclic loading (Hynes-Griffin and Franklin, 1984).

Additional details on the seismic slope stability are provided in Section 4 of the 2026 Safety Factor Assessment Report.

## CROSS SECTION GEOMETRY

The following section describes the development for the (i) external geometry; (ii) subsurface stratigraphy; and (iii) water levels and phreatic surface for the cross sections evaluated as part of this safety factor assessment.

### External Geometry

The height of the Ash Pond A perimeter dikes is approximately 15 ft adjacent to the Intake and Discharge Canals and approximately 25 ft adjacent to the Cooling Pond. The upstream and downstream side slopes range from 2 horizontal to 1 vertical (2H:1V) to 3H:1V, while the dike crest is typically 12 to 15 ft wide (Thomas and Hutton, 2012). Certain sections of the dike have wider crest widths resulting from recent regrading activities.

Five cross sections were developed and evaluated as part of the 2016 Safety Factor Assessment Report (Geosyntec, 2016). These cross sections were selected based on the critical slope geometry, engineering parameters of subsurface materials, and phreatic conditions. The external geometry of each cross section was based on a topographic survey prepared by Thomas and Hutton (2012) and a limited bathymetric survey within the Cooling Pond at the downstream toe of the perimeter dikes. Parker Land Surveying, LLC visited WGS in November 2015 to collect survey transects within the Cooling Pond at the base of the Ash Pond A perimeter dikes where the Cooling Pond appeared to be the deepest based on aerial photography and prior site visit observations.

A review of the topographic survey data from January 2026 showed no CCR impounded by the perimeter dikes of Ash Pond A (Worley, 2026). During the 2021 Safety Factor assessment, cross-section E was the only cross-section with CCR impoundment by the perimeter dikes. Therefore, the same critical cross section E was selected for updated analysis in 2026 Safety Factor Assessment (Figure 1). Although no CCR remains within Ash Pond A, the impoundment has not received official closure certification. Therefore, a safety factor assessment is required because the pond is still considered an existing CCR unit under the CCR Rule. In addition, the assessment evaluates conditions with maximum surcharge water levels (discussed in the following section), which may represent a critical stability case and govern the overall factor of safety.

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## Subsurface Stratigraphy

The subsurface stratigraphy for each cross section was developed based on soil borings and cone penetration tests (CPTs) conducted as part of Geosyntec’s 2013 subsurface investigation. Santee Cooper personnel indicated that no additional geotechnical investigations were conducted in the area of Ash Pond A since 2016; therefore, the subsurface stratigraphy developed in the 2016 Safety Factor Assessment remains valid. Generally, the subsurface in the depth of interest for slope stability analyses consists of the following strata (from top to bottom): Dike Fill, Foundation Soils, Chicora Member, and Williamsburg Formation Clay. Further discussion on the development of subsurface conditions can be found in the 2016 Safety Factor Assessment Report (Geosyntec, 2016).

## Water Levels

The CCR Rule requires the evaluation of safety factors considering static and seismic slope stability analyses under long-term “Maximum Normal Storage Pool” conditions and static slope stability analyses under short-term “Maximum Surge Pool” conditions. Water levels in the surface impoundment, perimeter dike, and downstream toe were determined as described below.

**Maximum Normal Storage Pool Condition:** Free water was not observed during the Geosyntec’s site visit in April 2026. Ash Pond A remains dry except during and shortly after storm events. Therefore, an operating water level at the bottom of the pond was assumed as the “Maximum Normal Storage Pool” within Ash Pond A. Groundwater level measurements previously provided by Santee Cooper were reviewed and compared with the 2026 topographic survey. Based on this review, no site conditions indicative of a substantial change in groundwater conditions were identified. Therefore, for the purposes of this assessment, groundwater levels within the perimeter dike and beyond the downstream toe of the perimeter dike were assumed to be consistent with those adopted in the 2016 and 2021 Safety Factor Assessments.

**Maximum Surge Pool Condition:** Because Ash Pond A was classified as a “Low Hazard Potential” surface impoundment (Geosyntec, 2021a), the 100-year rainfall event with a rainfall duration of 72 hours was selected as the Inflow Design Flood (IDF), as required by §257.73(d)(1)(v)(B). The maximum surface water elevation within Ash Pond A during and after the IDF is shown in Attachment 2 to the 2026 Safety Factor Assessment Report (H&H Analysis Results) as 25.0 ft (Plant Vertical Datum), identical to the 2021 Safety factor Assessment Report. The water levels within the perimeter dike and beyond the downstream toe of the perimeter dike for the Maximum Surge Pool condition were also assumed to be identical to those for the Normal Storage Pool condition in the 2016 and 2021 Safety Factor Assessments.

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## Final Cross Section Geometry

The final geometric models implemented within SLIDE<sup>®</sup> for Cross Section E are provided in Figure 2 and Figure 3 for the maximum normal and surcharge storage pool conditions, respectively.

## ENGINEERING PARAMETERS

The following sections describe the engineering parameters selected for the analyses presented in this calculation package.

### Material Parameters

Material parameters for dike fill, foundation soils, and underlying strata have been evaluated in the 2016 Safety Factor Assessment Report (Geosyntec, 2016) using in-situ and laboratory data collected in the vicinity of Ash Pond A. Geosyntec assumed the site subsurface conditions have generally remained unchanged. Table 1 provides a summary of the material properties selected for the cross section evaluated as part of the 2026 Safety Factor Assessment. For Cross Section E, specific dike fill and sandy foundation soil drained shear strength parameters were developed from in-situ measurements (i.e., Standard Penetration Test (SPT) N-values and CPT soundings). The interpretation and selection of properties are shown on Figure 4 for Cross Section E.

It was assumed that seismic waves from the design earthquake event may load clayey foundation soils rapidly enough to induce an undrained loading condition. In accordance with a recommendation made within Hynes-Griffin and Franklin (1984), the selected undrained shear strength value for the clayey foundation soils was conservatively reduced by 20 percent for the seismic slope stability analyses to account for potential cyclic degradation during an earthquake.

### Seismic Loading and Allowable Displacement

An evaluation of the seismic hazard for WGS and the site response analysis for Ash Pond A perimeter dikes are presented in Section 4 of the 2026 Safety Factor Assessment Report. Maximum shear stress profiles were computed for the six ground motions for WGS. The maximum shear stress profiles were used to compute the profiles of MHEA in general accordance with Bray et al (1995). The largest MHEA from the six ground motions at the critical slip surface depth was selected to compute the horizontal seismic coefficient for the seismic slope stability analyses. An MHEA of 0.050g was selected for Cross Section E.

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As described in the Methodology section, the horizontal seismic coefficient ( $k_h$ ) must be computed assuming an allowable deformation ( $u$ ). An allowable deformation of 12 inches (in.) (30.48 centimeters [cm]) was selected for the Ash Pond A perimeter dike structures. This is a conservative allowable deformation typically used for seismic analyses of large waste disposal structures (e.g., landfills) (Kavazanjian, 1999). Using the Hynes-Griffin and Franklin (1984) chart and assuming the “Upper Bound” displacement, the ratio of  $N/A$  (or  $k_h/MHEA$ ) was conservatively selected as 0.50, as shown in Figure 5. Thus, a  $k_h$  value of 0.025 was computed for Cross Section E.

## RESULTS

The safety factor evaluation for Cross Section E was performed according to the methodology and parameters outlined within this calculation package, and the results are summarized within Table 2. Computed FS were found to exceed the minimum safety factors required by §257.73(e)(1) of the CCR Rule. Figures 6, 7 and 8 depict the calculated safety factors for Cross Section E. While both rotational and non-rotational failure modes were considered in the analyses, rotational slip surfaces were consistently more critical failure modes of concern and are the critical slip surfaces as presented in Figures 6, 7 and 8.

## CONCLUSIONS

Based on the assumptions, analyses, and results presented within this calculation package, Ash Pond A at WGS satisfies the safety factor requirements described within the CCR Rule for existing CCR surface impoundments.

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- Bishop, A. (1955), “*The Use of the Slip Circle in the Stability Analysis of Slopes*,” *Geotechnique*, Volume 5, No. 1, Jan 1955, pp. 7-17.
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- Geosyntec Consultants, Inc. (2021a), “2021 Periodic Safety Factor Assessment: Ash Pond A, Winyah Generating Station, Georgetown, South Carolina”, submitted to Santee Cooper.

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|-------------|----------------------------------|----------|----------------------------------|--------------|----------------------------------|------------|------------------|
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Janbu, N. (1973), “Slope Stability Computations in Embankment-Dam Engineering”, R.C. Hirschfeld and S.J. Poulos, Eds. New York: Wiley, pp. 47-86.

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Spencer, E. (1973), “The Thrust Line Criterion in Embankment Stability Analysis,” Géotechnique, Vol. 23, No. 1, pp. 85-100, March 1973.

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# **TABLES**

**Table 1. Selected Material Parameters for Analysis**

| <b>Material</b>                | <b>Total Unit Weight<br/>(pcf)<sup>[2]</sup></b> | <b>Drained Parameters</b> |            | <b>Undrained<br/>Parameters<sup>[1]</sup></b> |                   |
|--------------------------------|--------------------------------------------------|---------------------------|------------|-----------------------------------------------|-------------------|
|                                |                                                  | $\phi'$ (°)               | $c'$ (psf) | $S_u/\sigma'_{vo}$                            | $S_{u,min}$ (psf) |
| Dike Fill                      | 125                                              | 40 <sup>[3]</sup>         | 0          | -                                             | -                 |
| Clayey Foundation<br>Soils     | 100                                              | 18                        | 250        | 0.32 <sup>[4]</sup>                           | 100               |
| Sandy Foundation<br>Soils      | 115                                              | 33 <sup>[3]</sup>         | 0          | -                                             | -                 |
| Loose Foundation<br>Soils      | 110                                              | 20 <sup>[3]</sup>         | 0          | -                                             | -                 |
| Chicora                        | 130                                              | 50 <sup>[2]</sup>         | 0          | -                                             | -                 |
| Williamsburg<br>Formation Clay | 105                                              | 50 <sup>[2]</sup>         | -          | -                                             | -                 |
| Fly Ash                        | 100                                              | 0 <sup>[2]</sup>          | 34         | -                                             | -                 |

Notes:

1. Undrained strength parameters for clayey foundation soils were applied for the seismic slope stability case only. Dike fill soils were observed to consist primarily of poorly graded silty sands in the vicinity of Ash Pond A.
2. The selection of shear strength parameters for Chicora, Williamsburg Formation Clay, and Fly Ash, as well as total unit weights for all materials, is explained in the 2016 Safety Factor Assessment Report (Geosyntec, 2016).
3. These drained shear strengths ( $\phi'$ ) vary by location. Interpretation of in-situ results applied in the selection is provided in Figure 3.
4. The selected undrained strength ratio ( $S_u/\sigma'_{vo}$ ) varies between locations. Interpretation of in-situ results applied in the selection is provided in Figure 3. A more detailed explanation of the undrained strength ratio for clayey foundation soils is provided in the 2016 Safety Factor Assessment Report (Geosyntec, 2016).

**Table 2. Summary of Safety Factor Analysis Results**

| <b>Safety Factor Case</b>             | <b>Target FS</b> | <b>Cross Section E<sup>[2]</sup></b> |
|---------------------------------------|------------------|--------------------------------------|
| Static - Maximum Normal Storage Pool  | 1.50             | <i>2.47</i>                          |
| Static - Maximum Surcharge Pool       | 1.40             | <i>2.45</i>                          |
| Seismic - Maximum Normal Storage Pool | 1.00             | <i>1.50</i>                          |
| Liquefaction <sup>[1]</sup>           | 1.20             | Not Applicable                       |

Notes:

1. The liquefaction safety factor was not evaluated because dike fill soils were found to be non-liquefiable.
2. Only critical failure surfaces passing through the perimeter dikes were considered.

## **FIGURES**

NORTH

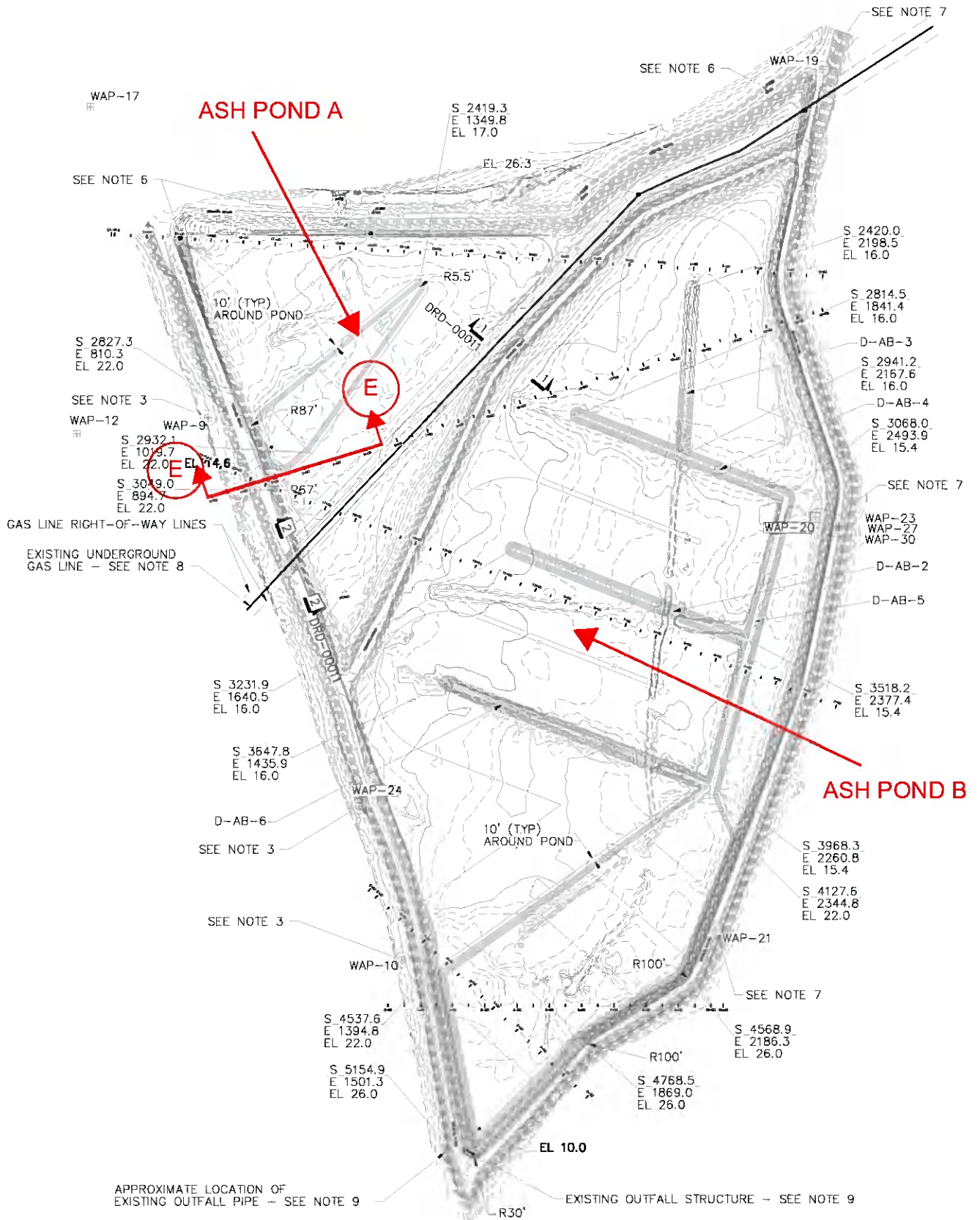


FIGURE 1. CROSS SECTION LOCATION PLAN

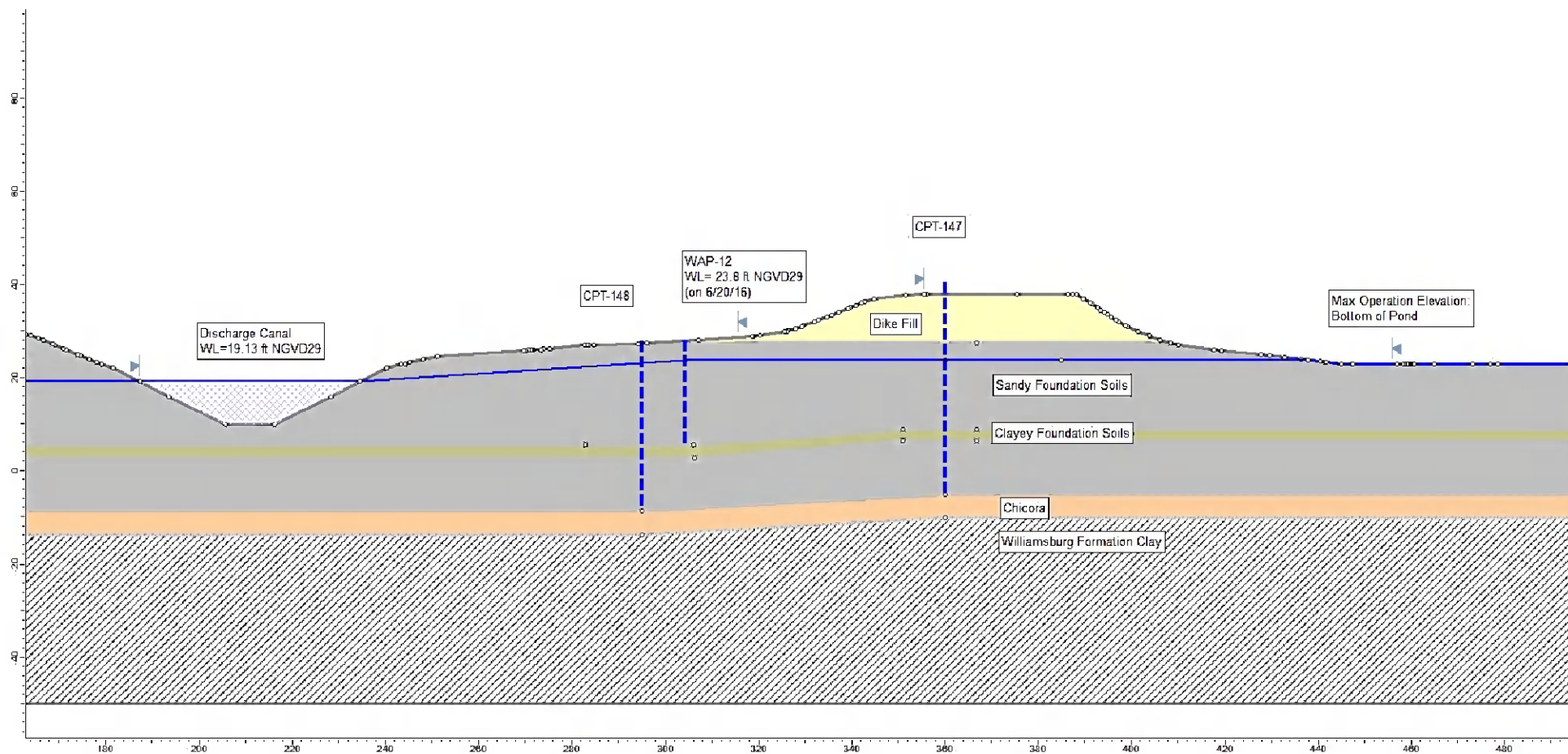


**Figure 2. Cross Section E Geometry during Maximum Normal Pool Conditions**

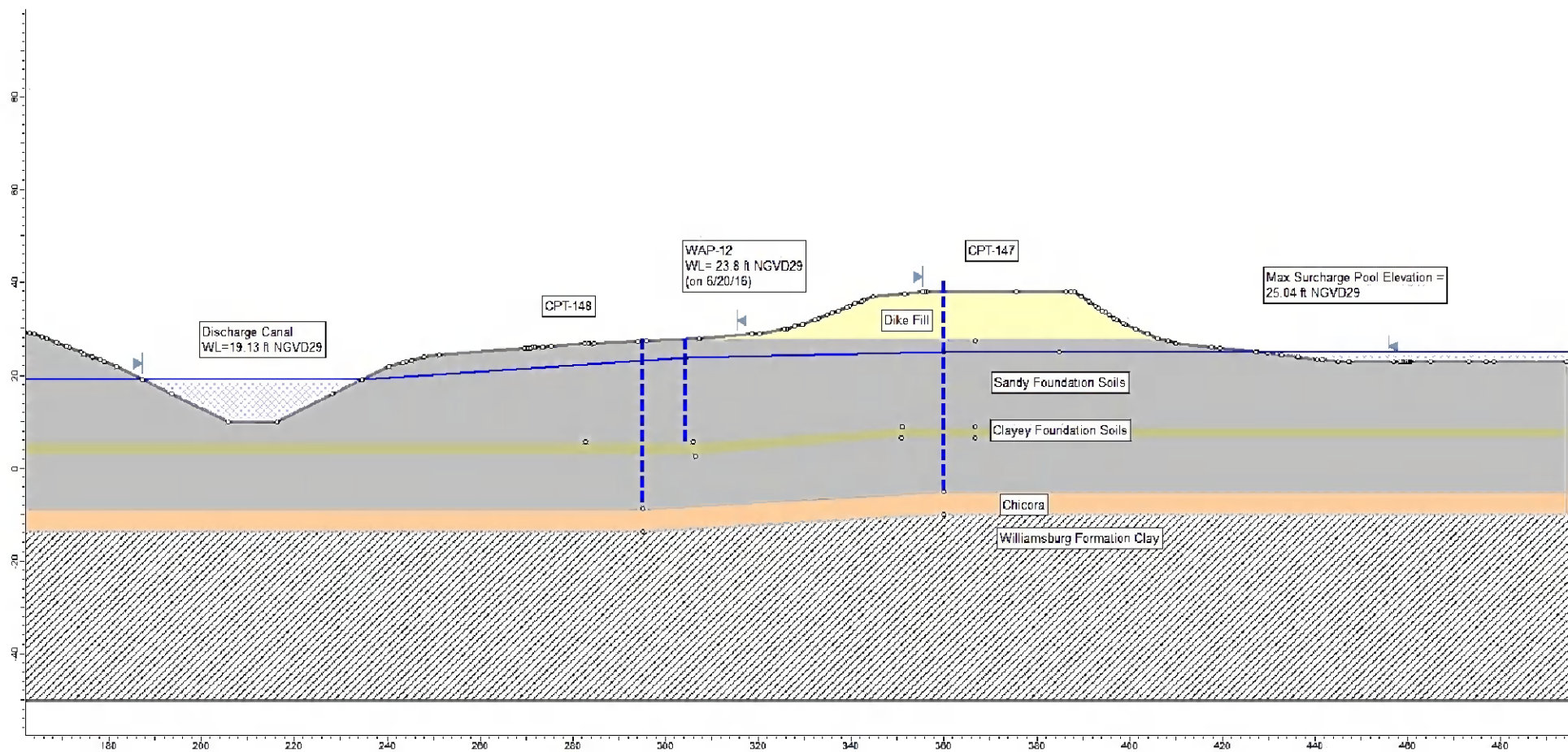


**Figure 3. Cross Section E Geometry during Maximum Surcharge Pool Conditions**

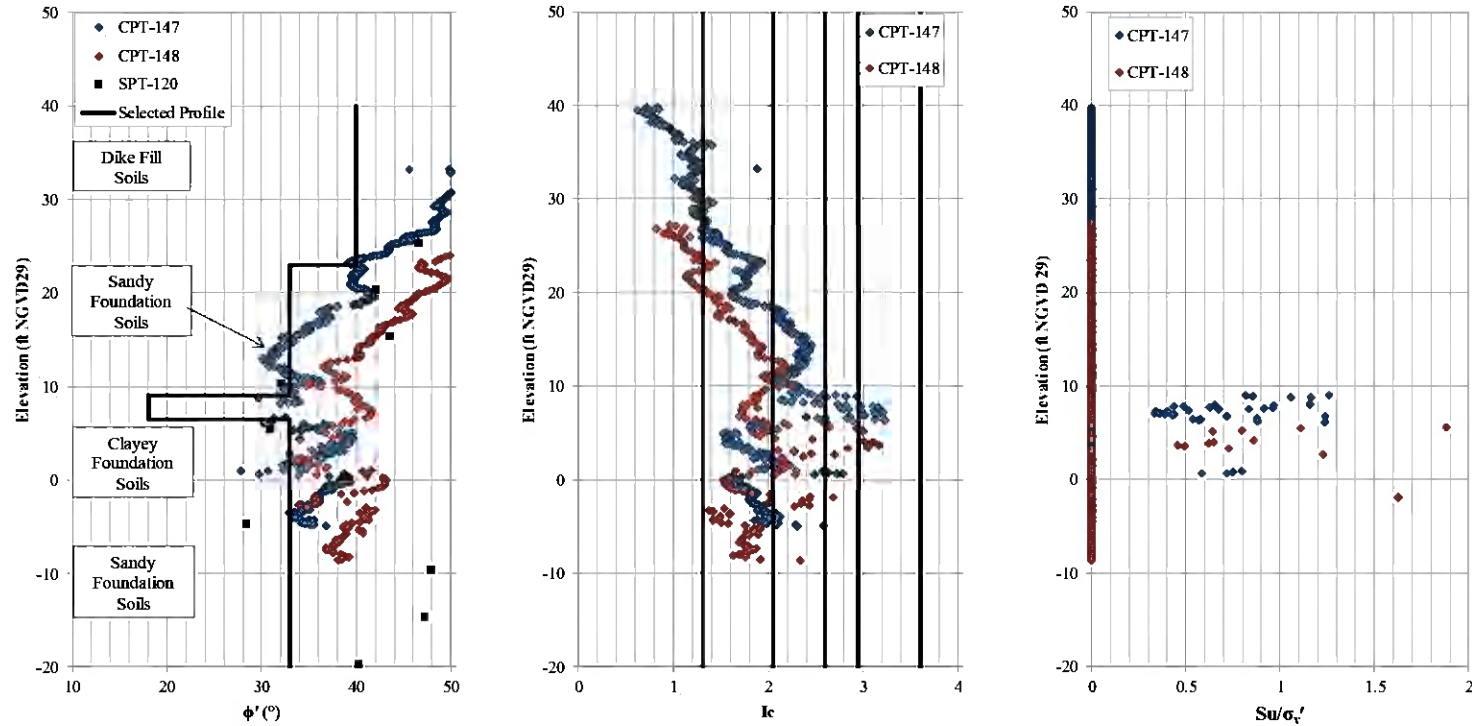


**Figure 4. Subsurface Stratigraphy and Shear Strength Model for Cross Section E**

Note:

1. Clayey foundation soils were observed in CPT-147 and were modeled from El. 9.0 to 6.5 ft NGVD29 and from 5.6 to 2.7 ft NGVD29 within CPT-148 with a  $\phi' = 18^\circ$  and a  $c' = 250$  psf during static slope stability and with 80 percent of the  $S_u/\sigma'_v = 0.40$  (i.e.,  $S_u/\sigma'_v = 0.32$ ) and a  $S_{u,min} = 100$  psf during pseudo-static stability analysis (i.e., seismic safety factor). Note that  $S_u/\sigma'_v$  for intervals where  $I_c < 2.60$  were plotted as zeros on the rightmost figure above.

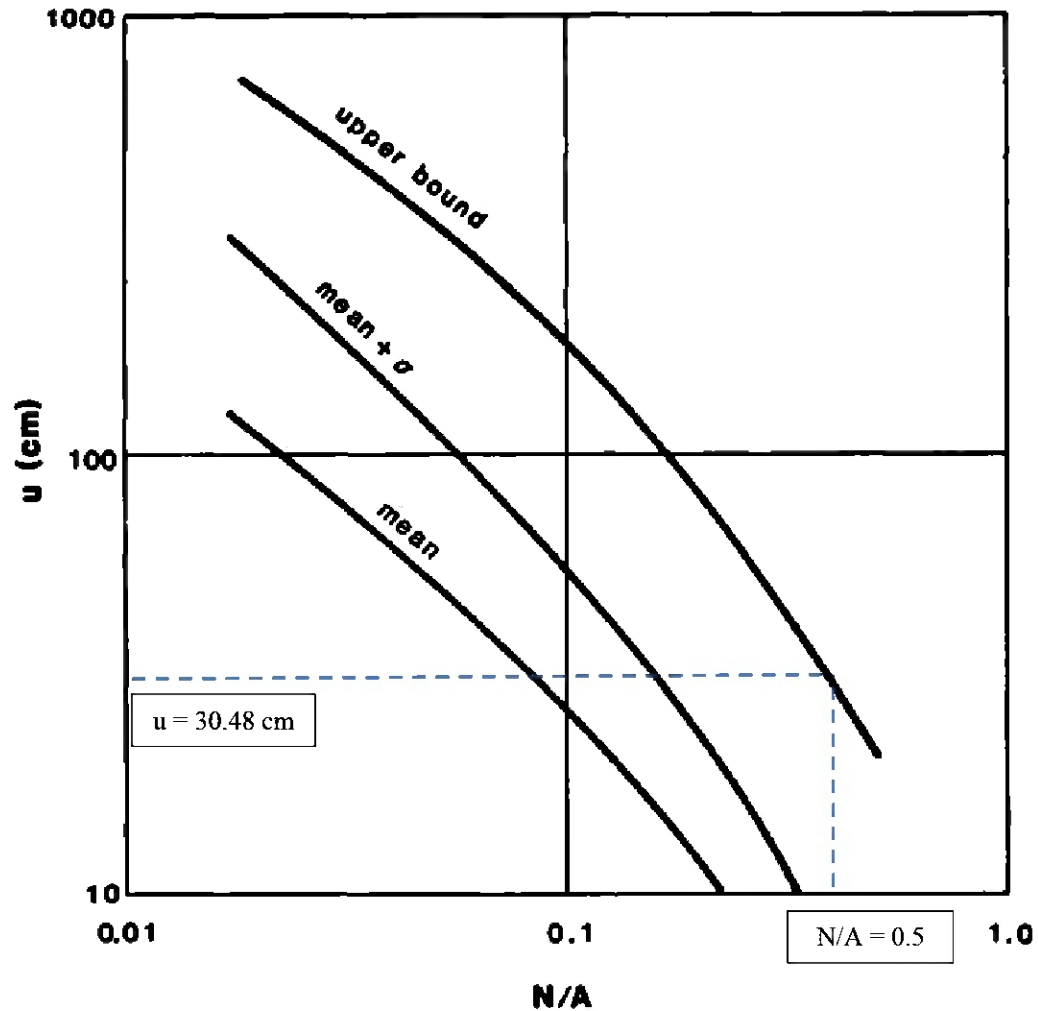


**Figure 5. Allowable Deformation ( $u$ ) vs.  $N/A$   
(from Figure 7 of Hynes-Griffin and Franklin, 1984)**

Notes:

1. An allowable deformation ( $u$ ) of 12 in. (30.48 cm) and the "Upper Bound" curve were selected for the 2026 Safety Factor Assessment.
2. A ratio of  $N/A$  of 0.50 was selected assuming 12 in. of allowable deformation.

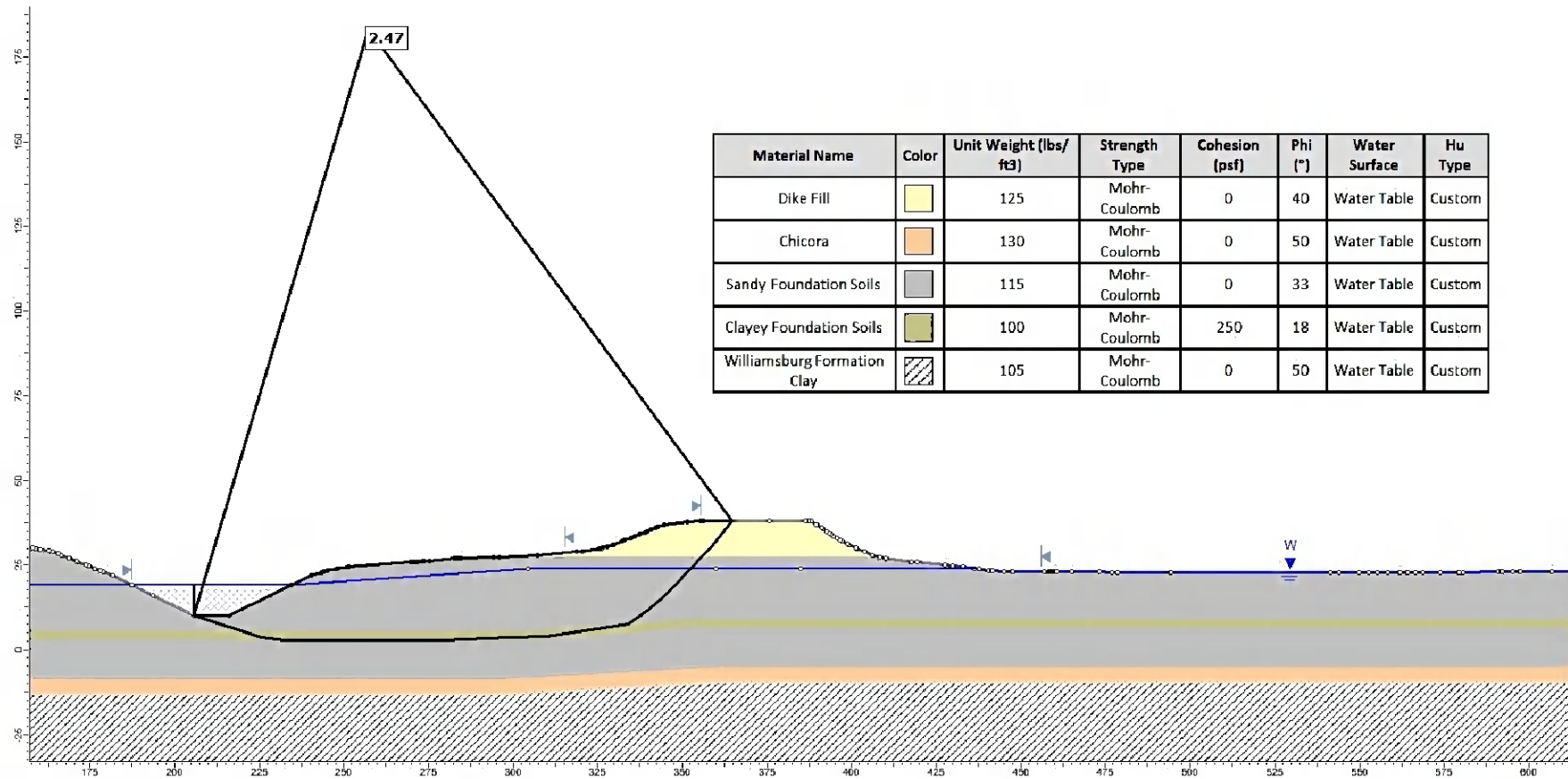


**Figure 6. Calculated Factor of Safety for Cross Section E: Static Factor of Safety – Maximum Normal Storage Pool Condition**

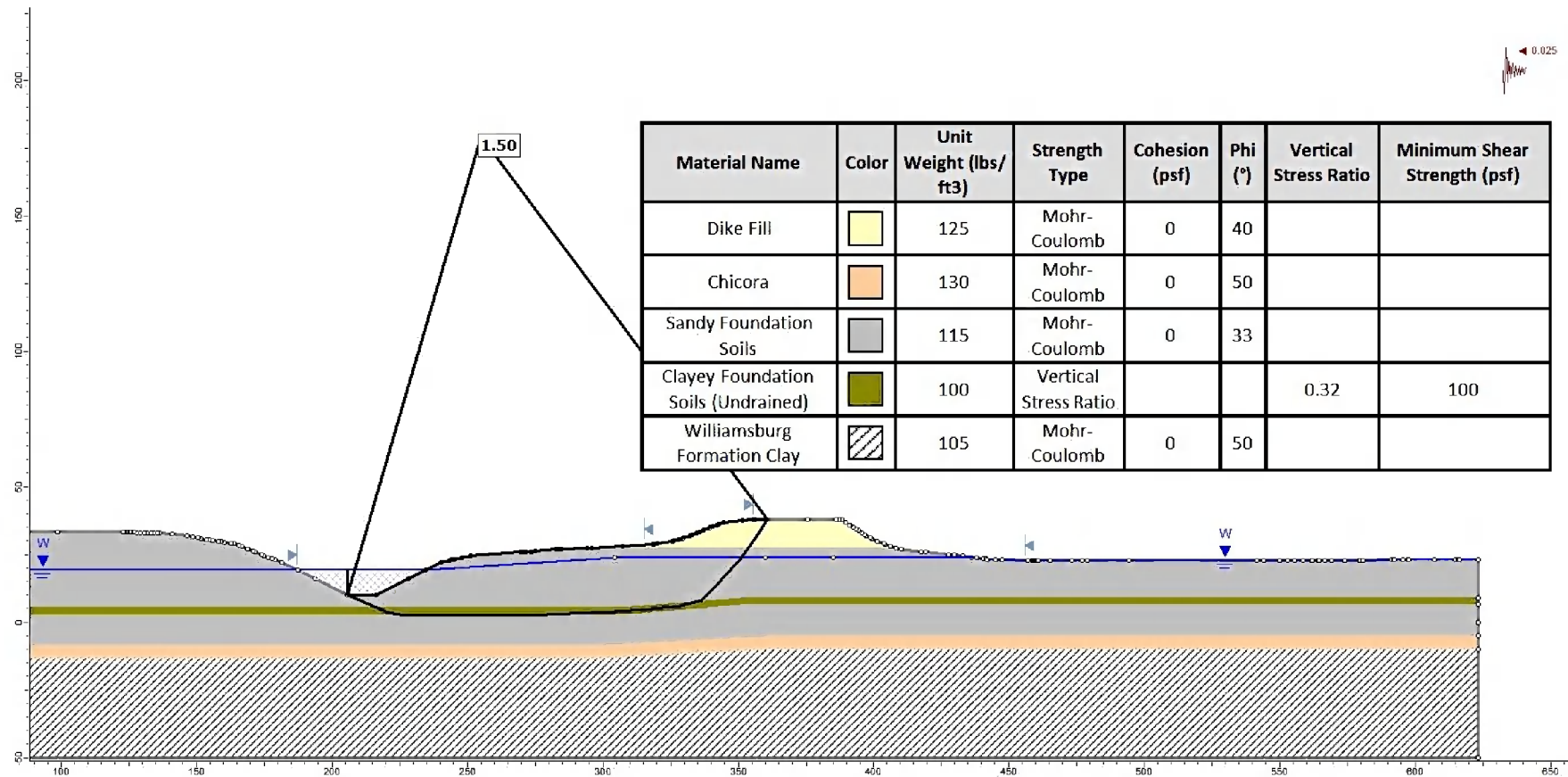


**Figure 7. Calculated Factor of Safety for Cross Section E: Seismic Factor of Safety – Maximum Normal Storage Pool**

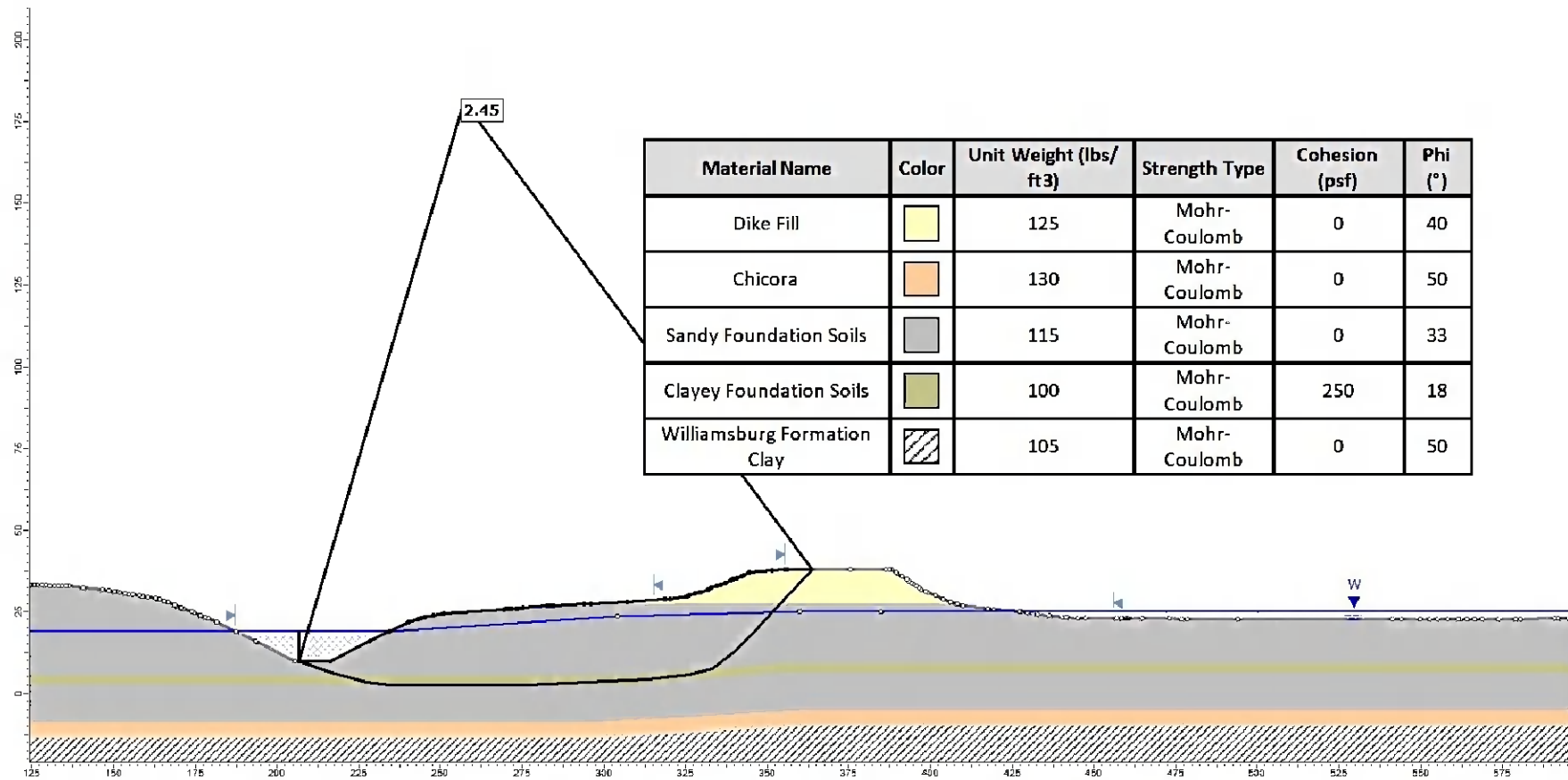


**Figure 8. Calculated Factor of Safety for Cross Section E: Static Factor of Safety – Maximum Surcharge Storage Pool Condition**