

Prepared for



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**2026 PERIODIC SAFETY FACTOR
ASSESSMENT
SLURRY POND 3 AND 4
WINYAH GENERATING STATION
Georgetown, South Carolina**

Prepared by



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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) outlines the structural stability criteria for existing CCR surface impoundments.

The Slurry Pond 3&4 (Slurry Pond) is situated west of the power block (Figure 2). The Slurry Pond contains CCR in the form of flue gas desulfurization (FGD) residuals as well as stormwater. It is considered as an existing surface impoundment under the CCR Rule. In accordance with §257.102(g), a Notice of Intent for the Slurry Pond was posted to the Operating Record on 9 April 2021 to initiate pond closure, and CCR and wastewater inflow to the Slurry Pond ceased in April 2021. Santee Cooper indicated the surface impoundment is planned to be closed by CCR removal within two years.

This *2026 Periodic Safety Factor Assessment: Slurry Pond* (Safety Factor Assessment Report) was prepared by Geosyntec Consultants, Inc. (Geosyntec) on behalf of Santee Cooper to demonstrate that the Slurry Pond 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

The Slurry Pond, encompassing approximately 107.2 acres (ac), is situated near the northwest corner of the Site, directly adjacent to Pennyroyal Creek. This unlined surface impoundment was commissioned in 1980 and was designated to receive FGD that do not meet specifications for beneficial use as wallboard-grade gypsum, process water from the power generating activities, and stormwater runoffs from the Limestone Slurry/Ball Mill area and Coal Pile (generally from the west half of the Coal Pile). The Slurry Pond is bounded to the south by the West Ash Pond (capped) and to the east by plant cooling towers and the plant area. The Slurry Pond perimeter dikes are bordered by Pennyroyal Creek and residential property on the west and north sides. The Slurry Pond was assigned “High Hazard Potential” classification (Geosyntec, 2016).

The Slurry Pond was constructed by compacting excavated soils from the surface impoundment interior to form the perimeter dikes and the divider dike, which separates the Slurry Pond from the adjacent West Ash Pond to the southwest. During the initial construction, a finger dike was constructed into the center of the Slurry Pond primarily to allow solids to settle prior to

recirculation of the wastewater, but also provided for access, maintenance, and observation of the pond interior. The Slurry Pond is bounded by perimeter dikes which are approximately 30 feet (ft) in height to the north, 26 ft in height to the west, and 20 ft in height to the east (Bowman Consulting Group, 2025).

1.3 Report Organization

This Safety Factor Assessment Report presents the subsequent periodic safety factor assessments for the Slurry Pond 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 from June 2025 (Bowman Consulting Group, 2025), 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 the Slurry Pond is presented in Section 3;
- Seismic hazard evaluations for WGS, the site response analysis of the Slurry Pond 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 Slurry Pond 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. A review of the topographic survey from June 2025 (Bowman Consulting Group, 2025) and the topographic survey used in the 2021 Safety Factor Assessment indicated that CCR has been moved within the surface impoundment and the volume of CCR impounded within the surface impoundment has changed significantly since the last assessment.

Groundwater level measurements previously provided by Santee Cooper were reviewed and compared with the 2025 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 the Slurry Pond ceased in April 2021.

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 the Slurry Pond 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 “Probable maximum flood (PMF) for a high 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.”

Since the Slurry Pond is classified as a high hazard potential surface impoundment (Geosyntec, 2016), the Inflow Design Flood (IDF) is the Probable Maximum Flood (PMF). H&H analyses were performed to demonstrate that the Slurry Pond inflow design flood control system is able to adequately manage flow during and following the IDF 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 surface water elevation within the Slurry Pond computed from the H&H analysis.

3.1.2 Methodology and Assumptions

Stormwater runoff volumes and associated discharges to Slurry Pond 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).

Stormwater runoffs from the exterior of the Slurry Pond area are collected in Detention Ponds No. 1 and No. 2 located outside the Slurry Pond. These detention ponds were designed to manage the 25-year, 24-hour storm event (Santee Cooper, 2004). Pump Station No. 1 receives stormwater from Detention Pond No. 1, as well as Cooling Tower blowdown, and discharges to the West Low Volume Waste Pond. Pump Station No. 2 receives stormwater from Detention Pond No. 2 and discharges to the Slurry Pond. However, for this analysis, Pump Stations No. 1 and No. 2 are assumed to be non-operational due to temporary loss of power during a large storm event.

A Floating Pump Station equipped with two Tsurumi GSZ-4-45-4 submersible pumps, normally conveys water from the Slurry Pond directly to the West Low Volume Waste Pond. The capacity of these pumps operating in parallel is 3,100 gallons per minute (gpm) at the normal operating elevation of 19.6 ft National Geodetic Vertical Datum of 1929 (NGVD 29) when pumping directly to the West Low Volume Waste Pond. Piping is valved such that the Floating Pump Station may convey water to Pump Station No. 1. However, for this analysis, the Floating Pump Station is assumed to be non-operational due to temporary loss of power during a large storm event.

The free water level within the Slurry Pond has been maintained below elevation 19.6 ft NGVD29 by the Floating Pump Station. Based on the annual inspection reports, the free water level has remained below elevation 19.6 ft NGVD29 since 2022.

Details of the H&H analysis are provided in a document titled “*Inflow Design Flood Control System Plan: Slurry Pond*” (Geosyntec, 2026c).

3.1.3 Analysis Results

Under the conditions and assumptions described in Section 3.1.2, the maximum surface water level during and following the IDF event (PMF) was computed as 28.3 ft NGVD29. The H&H analysis results (i.e., *HydroCAD* results) are included as Attachment 2 to 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 Slurry Pond 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 18 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 19 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 minor changes in the calculated seismic coefficients for Slurry Pond – Profile 2, with slightly larger increases calculated for Slurry Pond – Profile 1, however the factor of safety was still above the allowable limit. Overall, the revised k_h values remain similar to those 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 Slurry Pond 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 displacements for Slurry Pond were less than 12 inches, as shown in Table 2. The estimated displacements are below the allowable deformation criteria used for the seismic stability evaluations and indicate 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 Slurry Pond 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 Slurry Pond 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 Slurry Pond.

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 coefficients 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 Slurry Pond. The estimated deformations are below the allowable deformation criteria and indicate 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 Slurry Pond. The conclusions of the 2021 site response, liquefaction, and seismic stability assessments remain valid, and Slurry Pond 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 Slurry Pond 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 Slurry Pond 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 sections of the Slurry Pond 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

The models used for the 2021 Safety Factor Assessment were updated with a topographic surface within the pond (Section 2). Consistent with observations regarding the water level described in Section 2, the water levels (within the perimeter dike and beyond the downstream toe of the perimeter dike) selected for the analyses are those used for the 2021 Safety Factor Assessment. For the 2021 Safety Factor Assessment Report, only the critical cross sections identified in the 2016 Safety Factor Assessment were analyzed. The same three critical cross sections are analyzed for the 2026 Safety Factor Assessment Report.

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[®], 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[®] 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 the Slurry Pond 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[®].

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 sections of the Slurry Pond as shown on Figures 2, 4, and 6 of Attachment 3.

5.5 Static Safety Factor – Maximum Surge Pool

§257.73(e)(1)(ii) requires that the static factor of safety meets or exceeds 1.40 for the maximum surge pool conditions within the surface impoundment. The static safety factors were evaluated with an elevation of 28.3 ft NGVD29 for the surge pool level. The static safety factor was evaluated for the critical cross sections of the Slurry Pond as shown on Figures 3, 5, and 7 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 applied to each slice within SLIDE[®]. During the evaluation of the seismic safety factor, soil shear strengths were conservatively reduced to account for potential influence of cyclic degradation.

5.7 Summary of Results

As presented in Table 3, the minimum calculated factors of safety for the static case with the maximum normal storage pool, the static case with the maximum surge pool, and seismic case with the maximum normal storage pool are more than the required minimum target values. These analysis results indicate that the perimeter dikes of the Slurry Pond 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 the Slurry Pond 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 maximum surge pool within the Slurry Pond for the safety factor assessment was established based on the H&H performance of the Slurry Pond during the IDF.

- 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 the Slurry Pond 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 Slurry Pond 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 sections of the Slurry Pond perimeter dikes, the Slurry Pond 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

Location	2021 Analysis		2026 Analysis	
	MHEA (g)	k_h	MHEA (g)	k_h
Slurry Pond – Profile 1	0.096	0.048	0.110	0.055
Slurry Pond – Profile 2	0.088	0.044	0.083	0.041

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

Location	Section	Yield Coefficient (k_y)	Displacement (in)
Slurry Pond 3&4	Section C	0.11	0.6
	Section D	0.11	0.8
	Section E	0.11	0.8

Table 3. Summary of Safety Factor Analysis Results^[2]

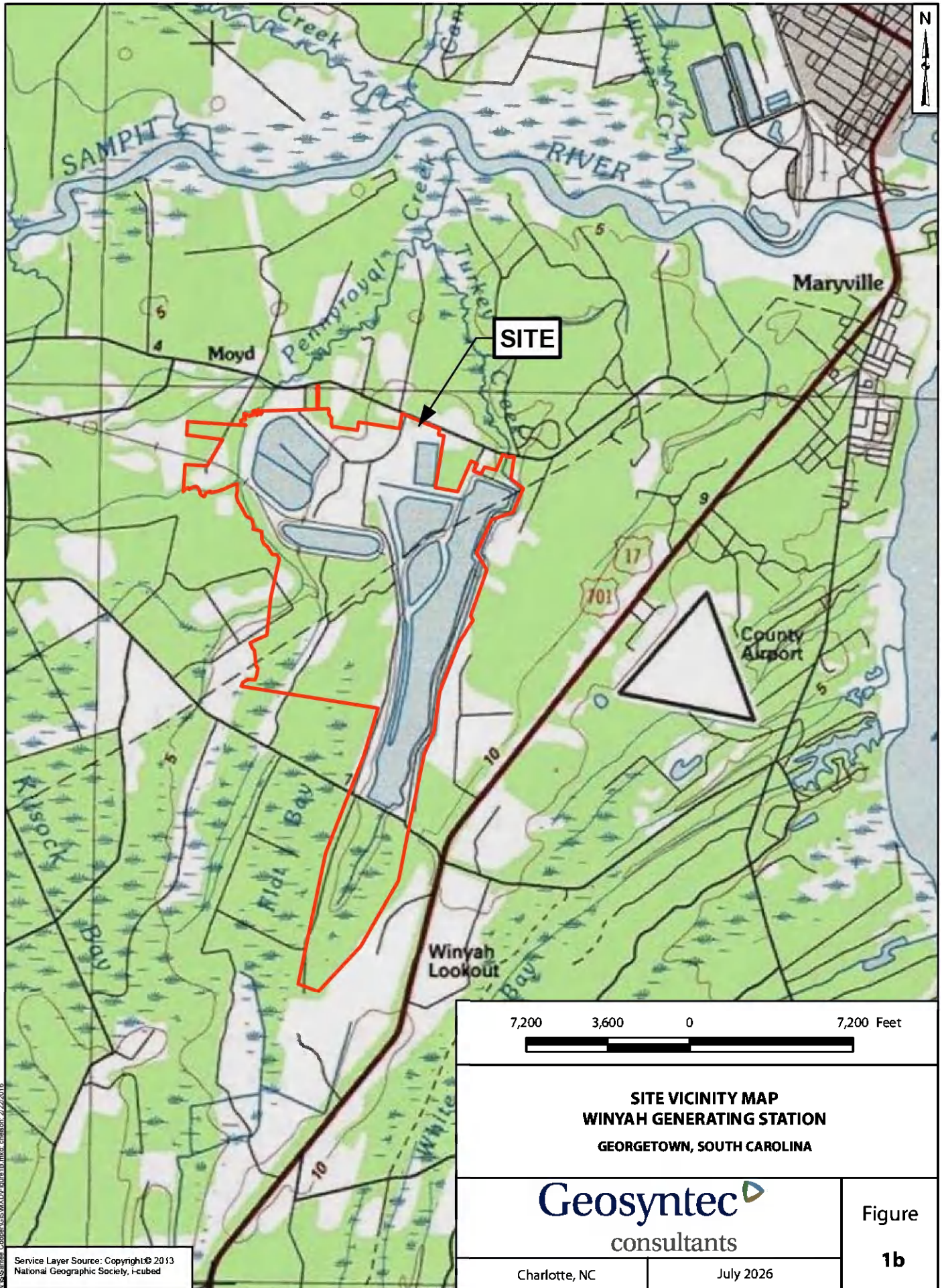
Safety Factor Case	Target FS	Cross Section C	Cross Section D	Cross Section E
Static - Maximum Normal Storage Pool	1.50	1.76	<i>1.69</i>	1.85
Static - Maximum Surcharge Pool	1.40	1.76	1.55	<i>1.49</i>
Seismic - Maximum Normal Storage Pool	1.00	1.17	1.17	<i>1.11</i>
Liquefaction ^[1]	1.20	Not Applicable	Not Applicable	Not Applicable

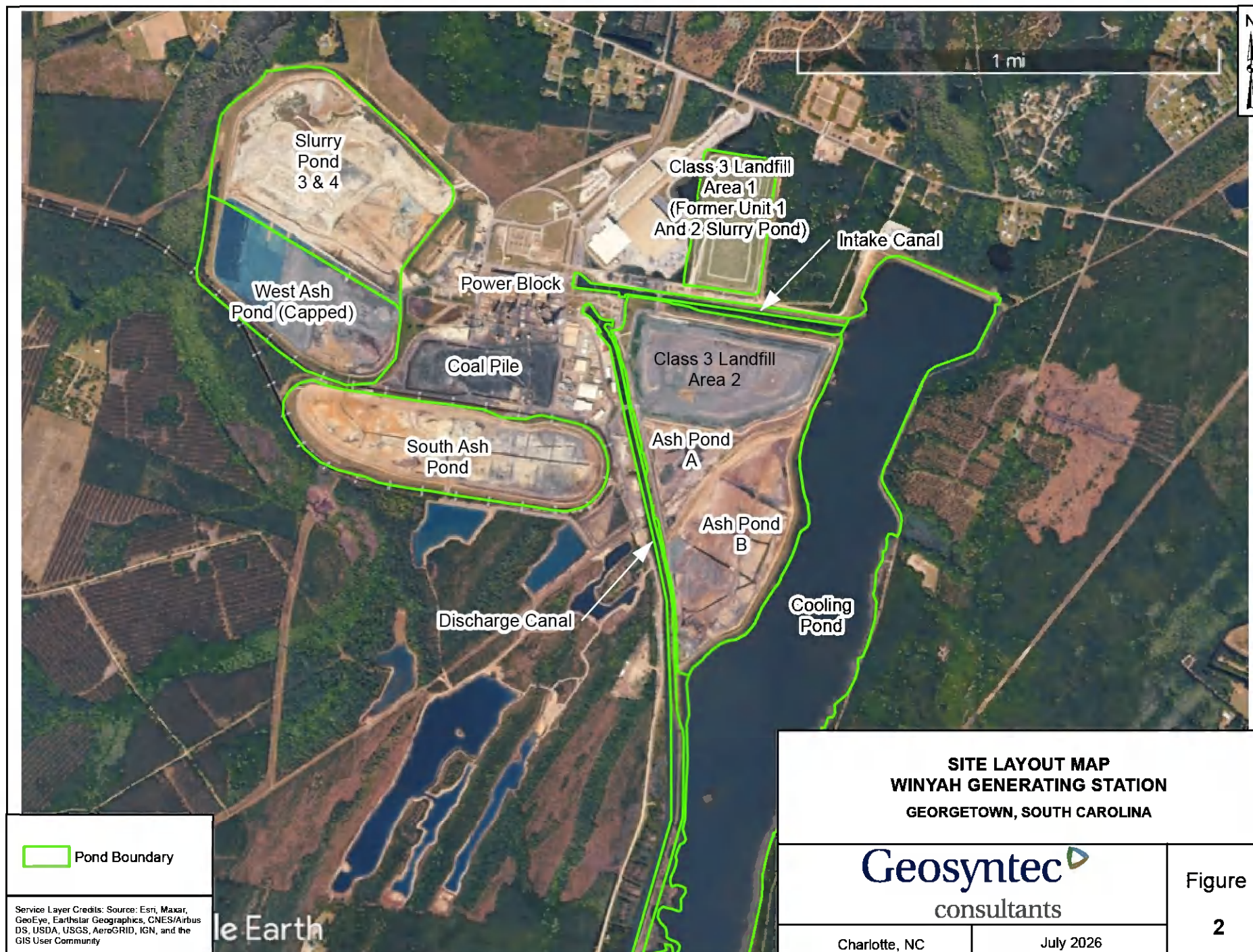
Notes:

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.
3. The lowest computed safety factor for each analysis case is italicized.

FIGURES







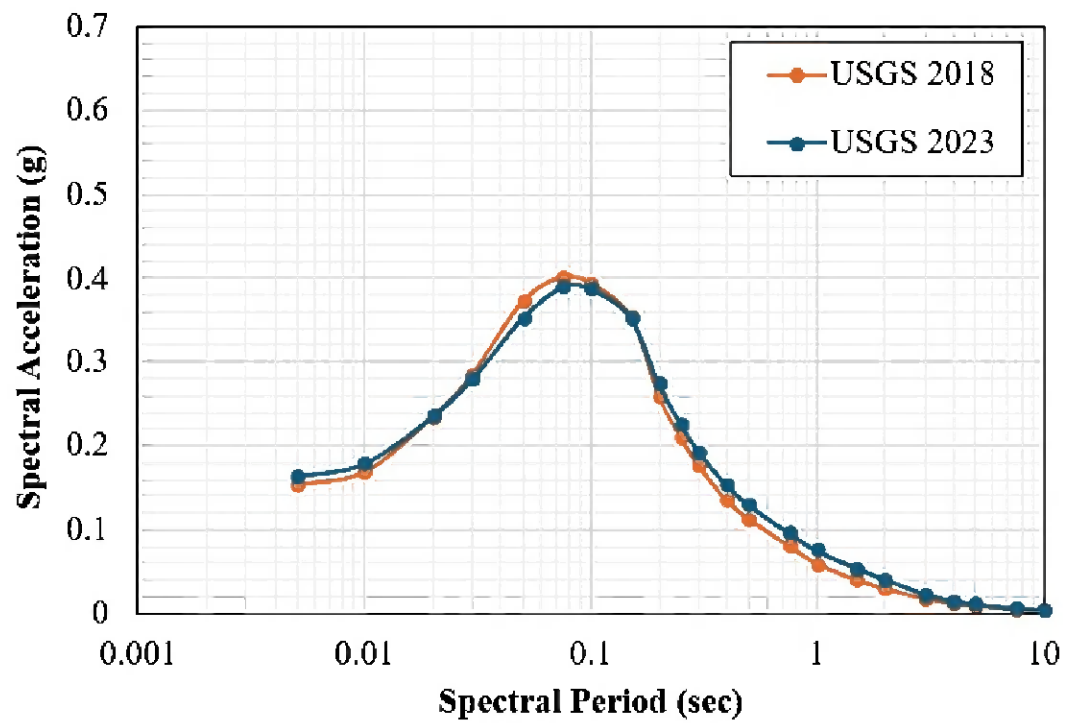


Figure 3. Target Response Spectrum Comparison for WGS

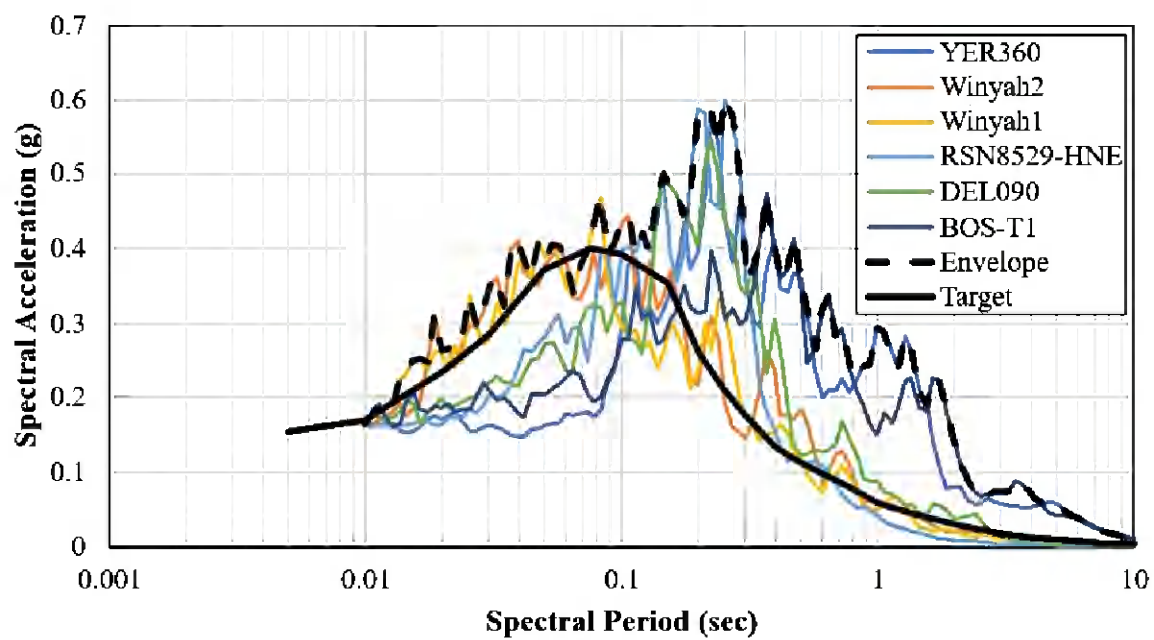


Figure 4. Input Motions Scaled to 2018 Target PGA

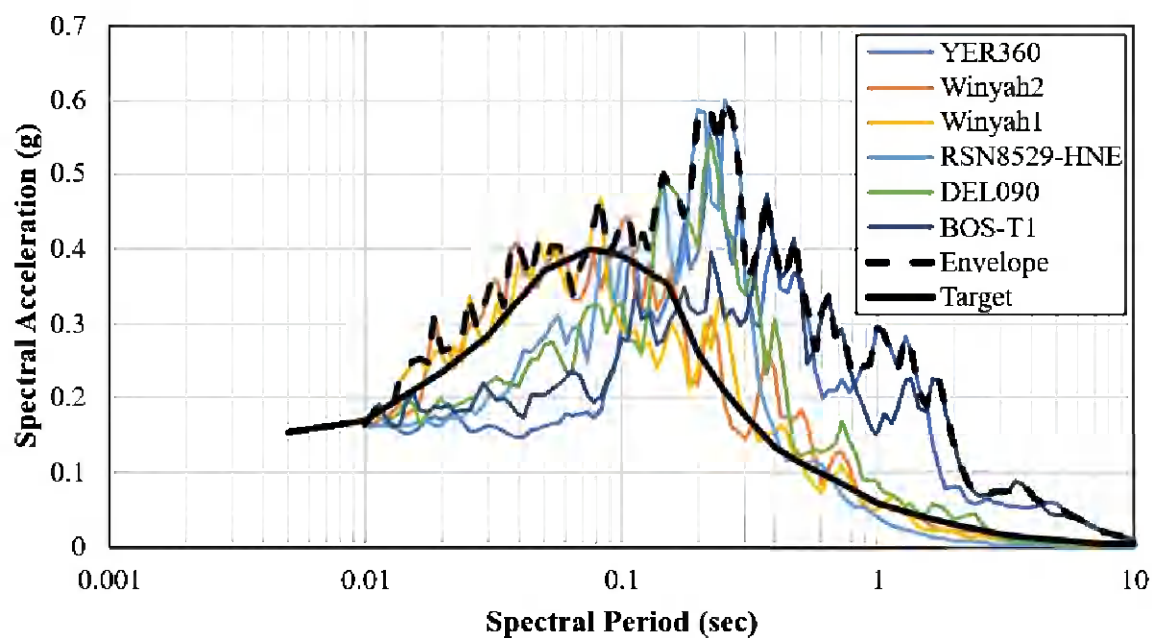


Figure 5. Input Motions Scaled to 2023 Target PGA

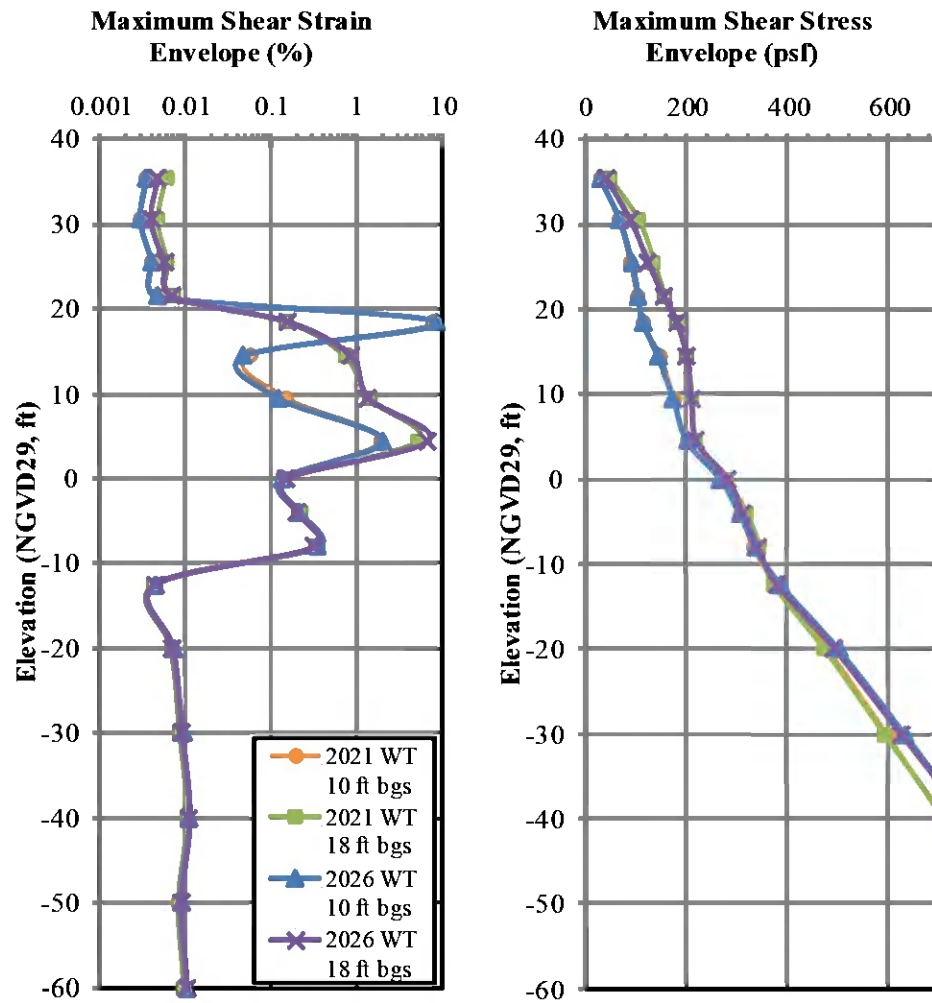


Figure 6. Comparison of Maximum Shear Strain and Shear Stress Envelopes at Slurry Pond 3&4

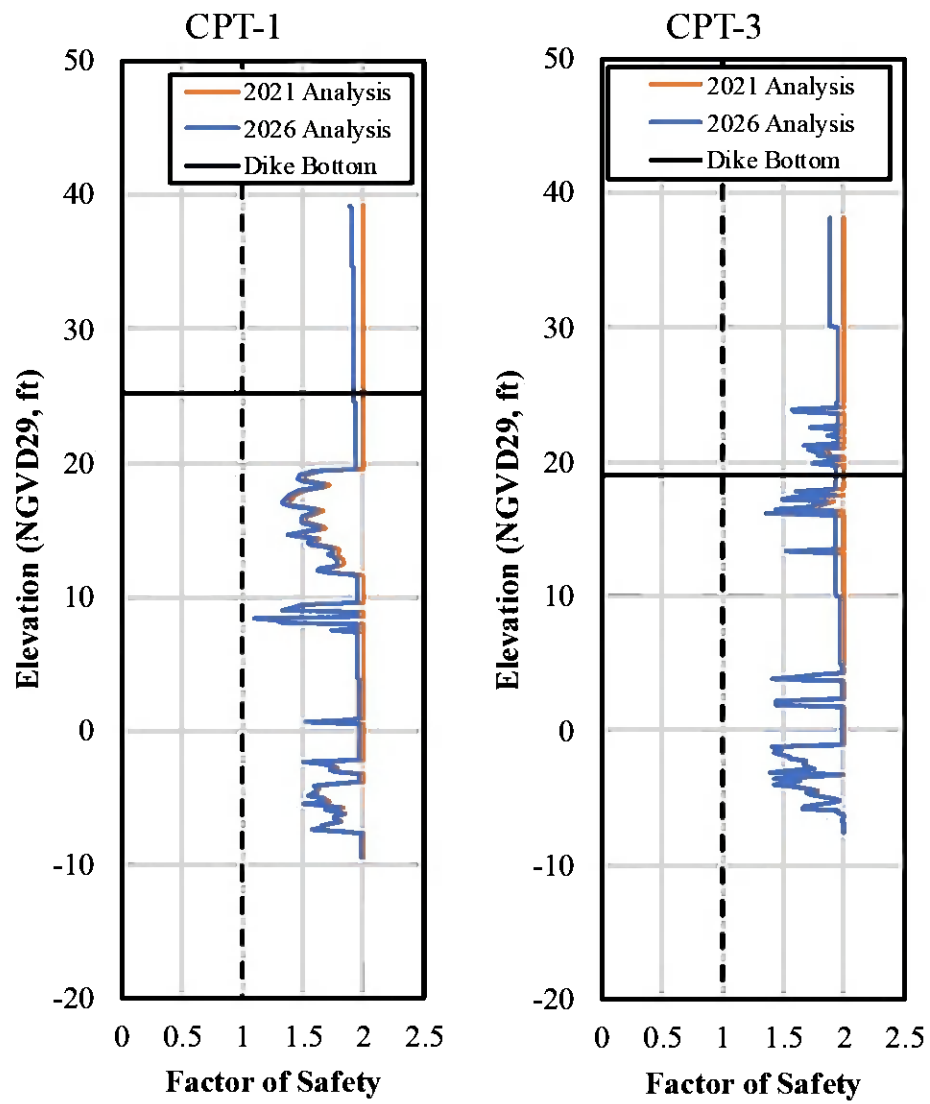


Figure 7. Comparison of Liquefaction Results for CPT-1 and CPT-3 at Slurry Pond 3&4

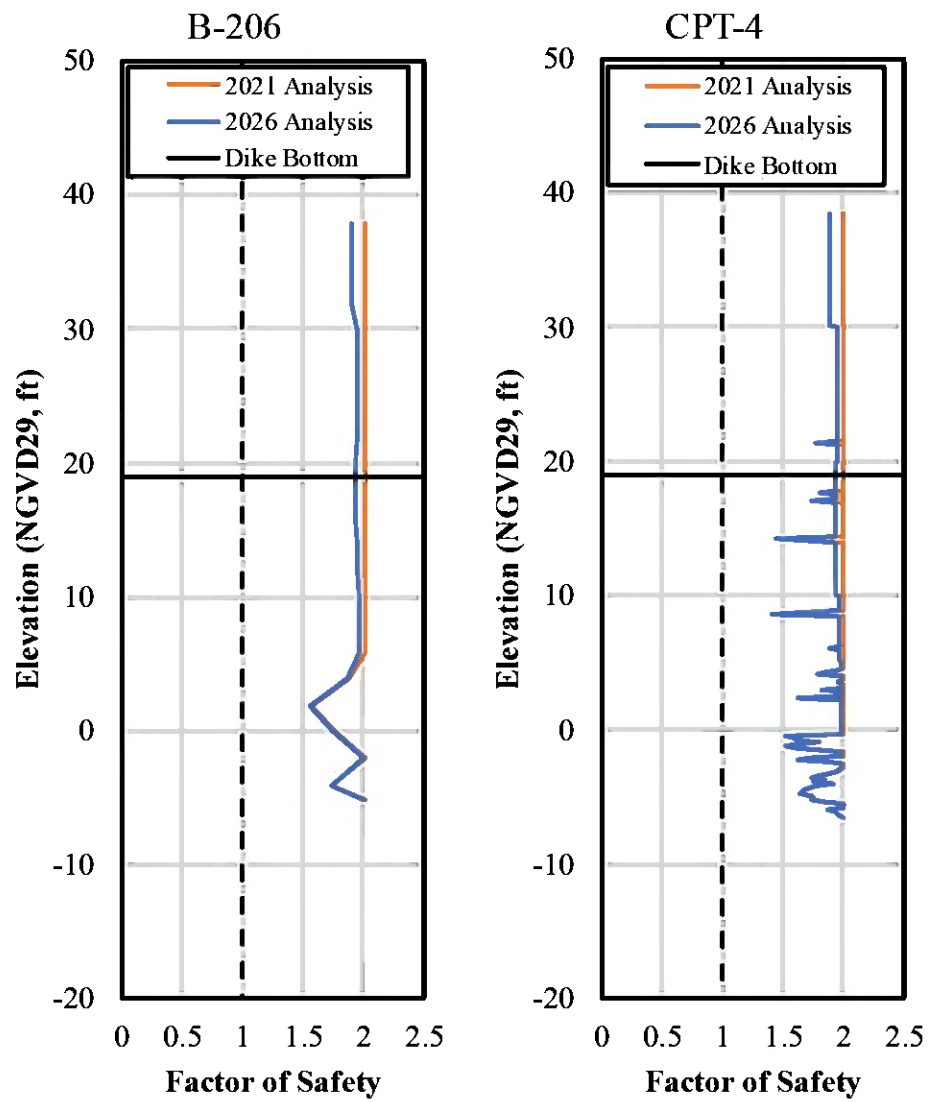


Figure 8. Comparison of Liquefaction Results for B-206 and CPT-4 at Slurry Pond 3&4

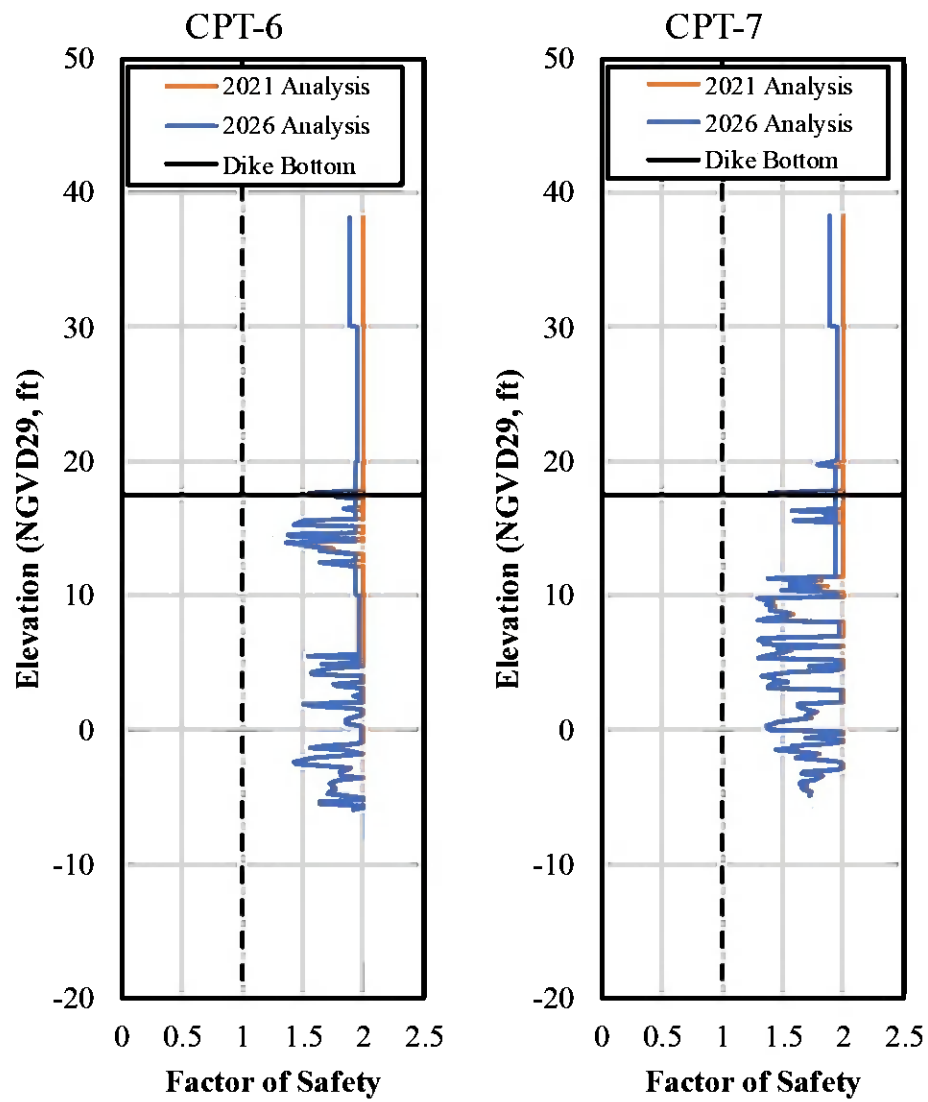


Figure 9. Comparison of Liquefaction Results for CPT-6 and CPT-7 at Slurry Pond 3&4

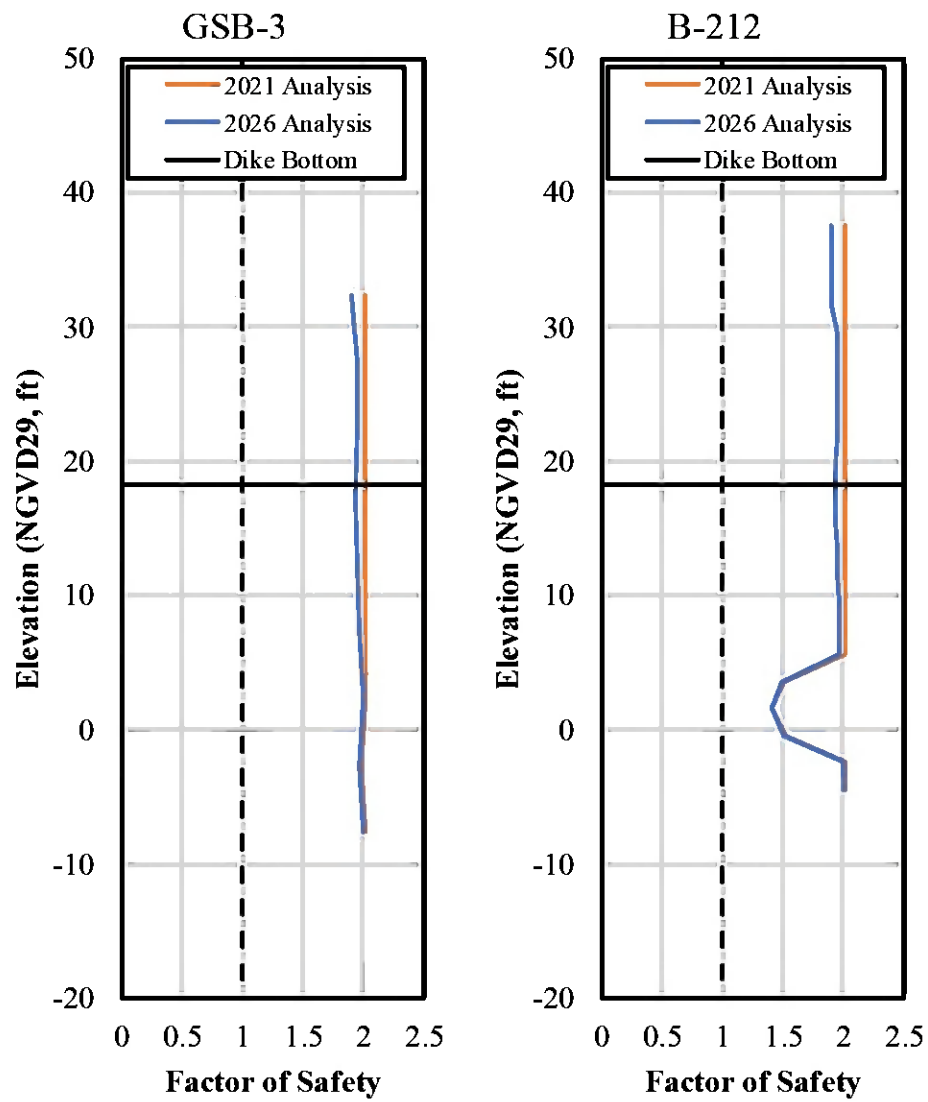


Figure 10. Comparison of Liquefaction Results for GSB-3 and B-212 at Slurry Pond 3&4

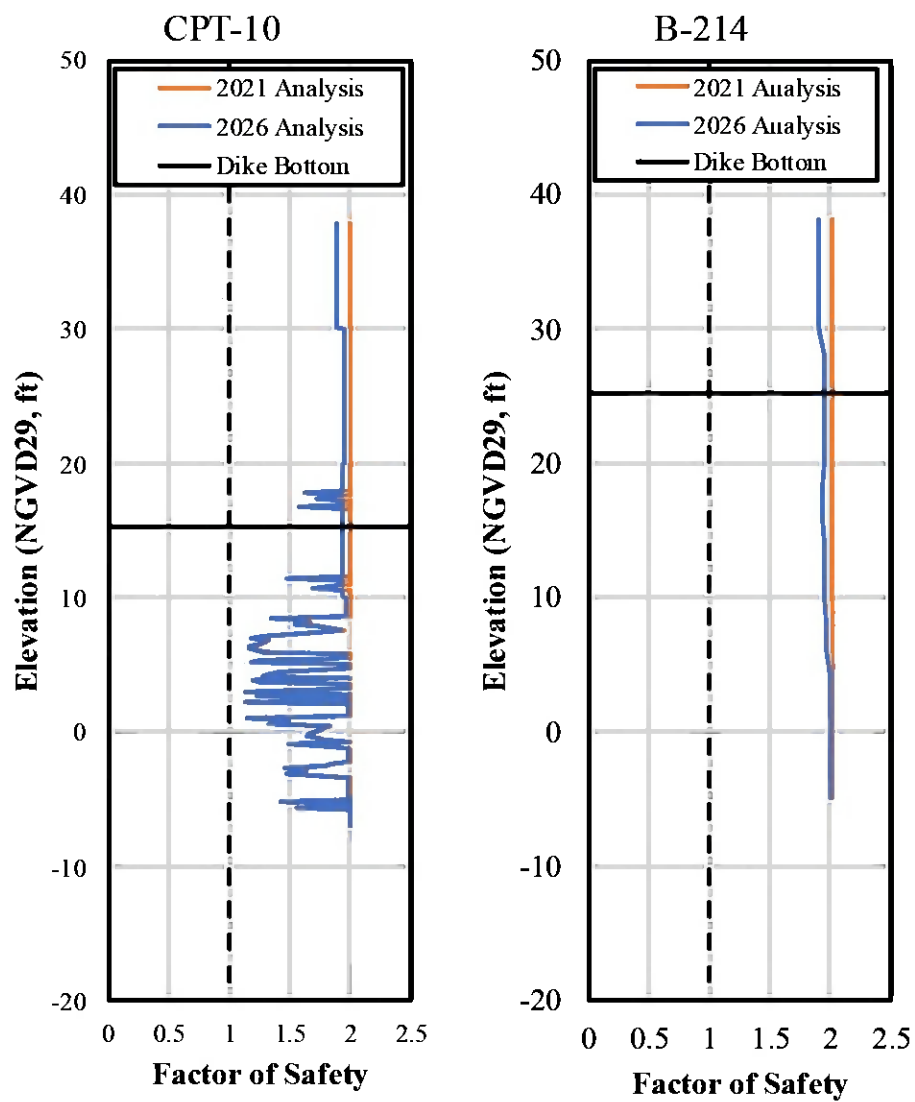


Figure 11. Comparison of Liquefaction Results for CPT-10 and B-214 at Slurry Pond 3&4

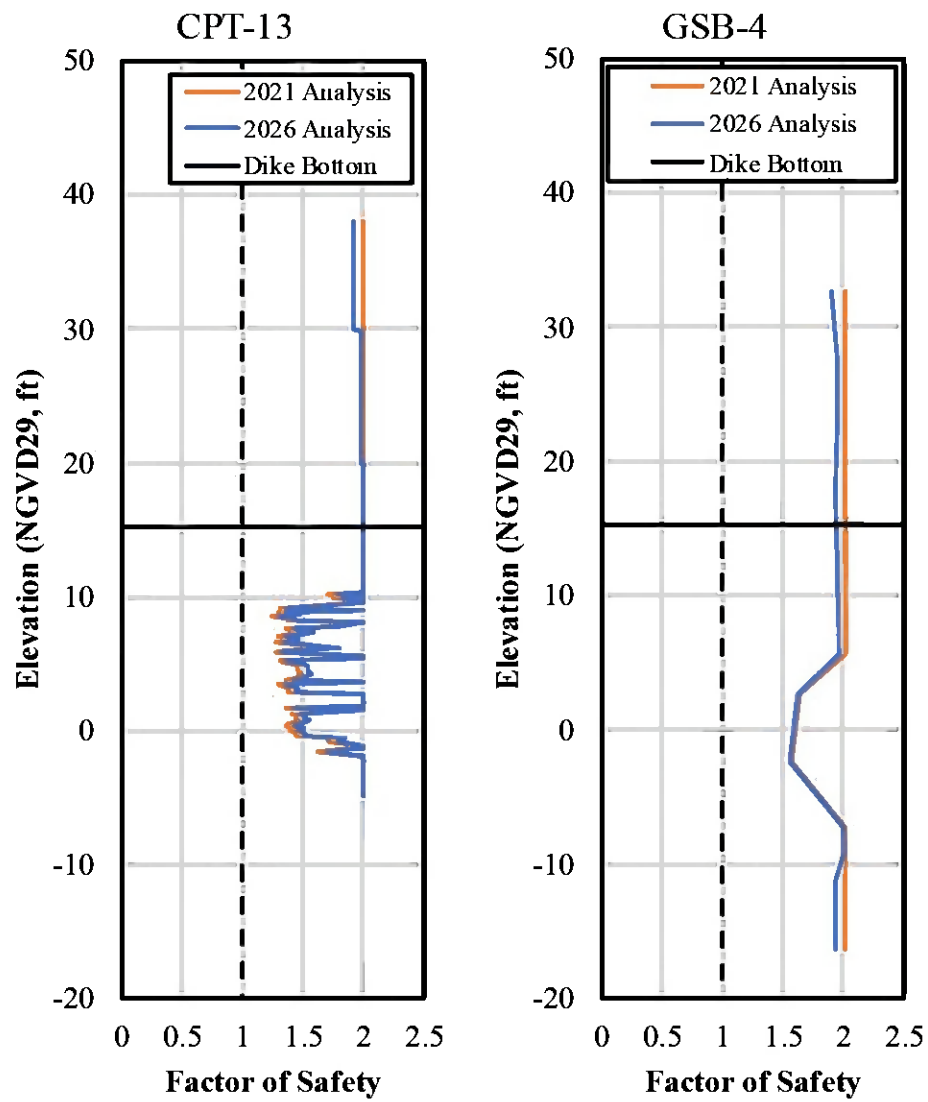


Figure 12. Comparison of Liquefaction Results for CPT-13 and GSB-4 at Slurry Pond 3&4

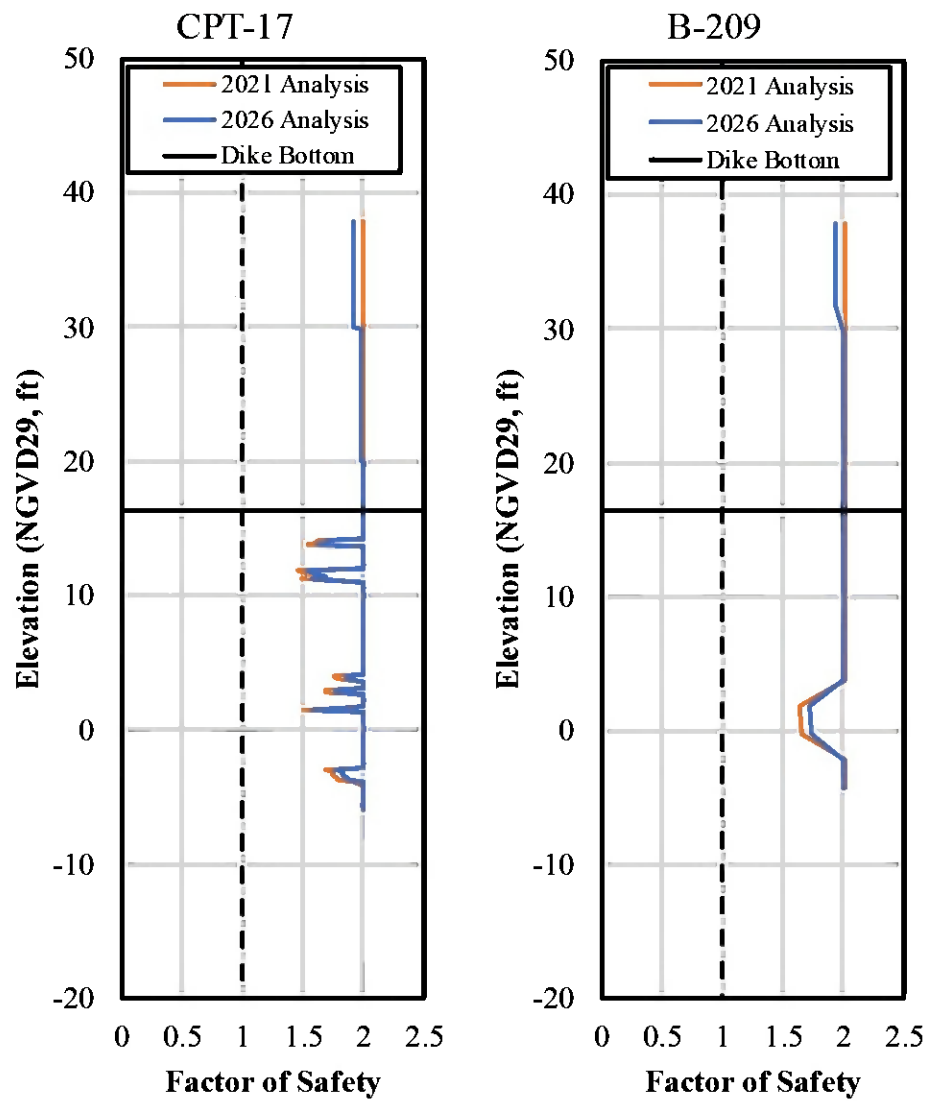


Figure 13. Comparison of Liquefaction Results for CPT-17 and B-209 at Slurry Pond 3&4

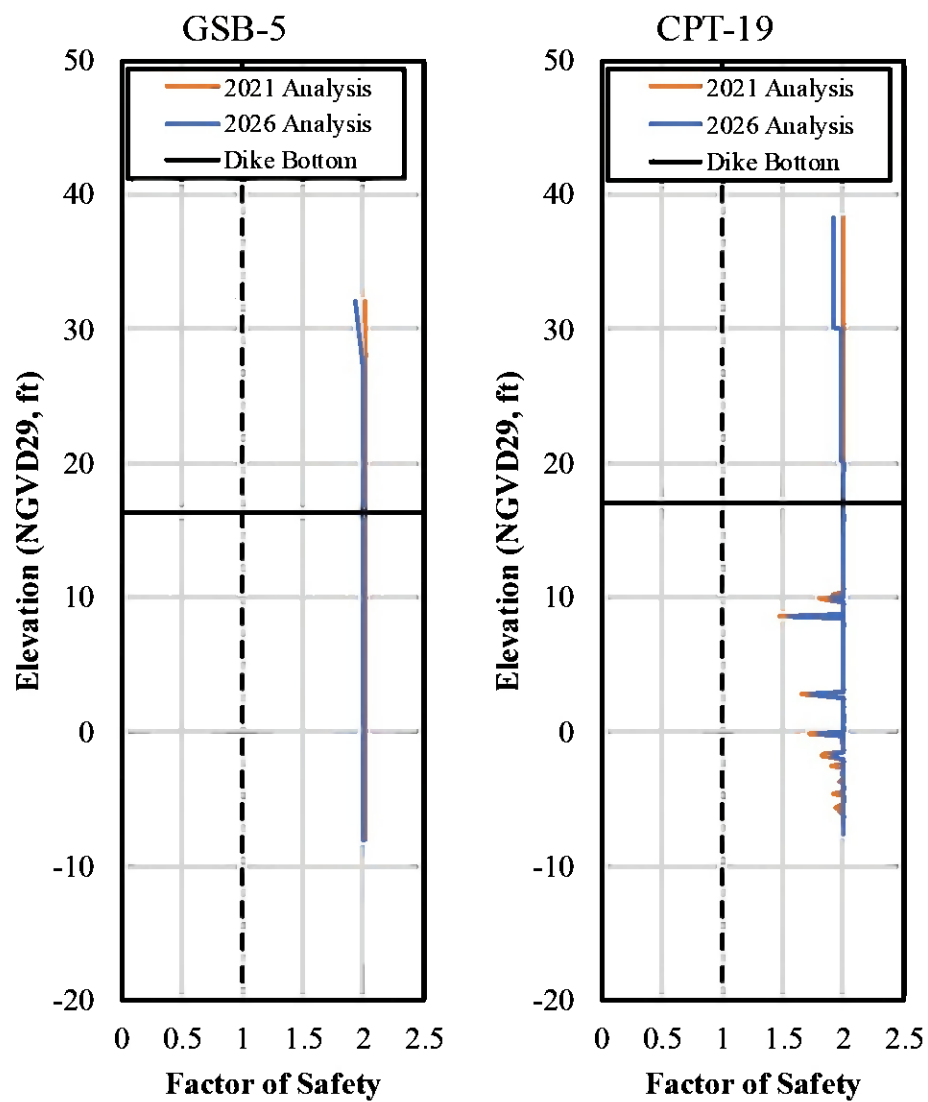


Figure 14. Comparison of Liquefaction Results for GSB-5 and CPT-19 at Slurry Pond 3&4

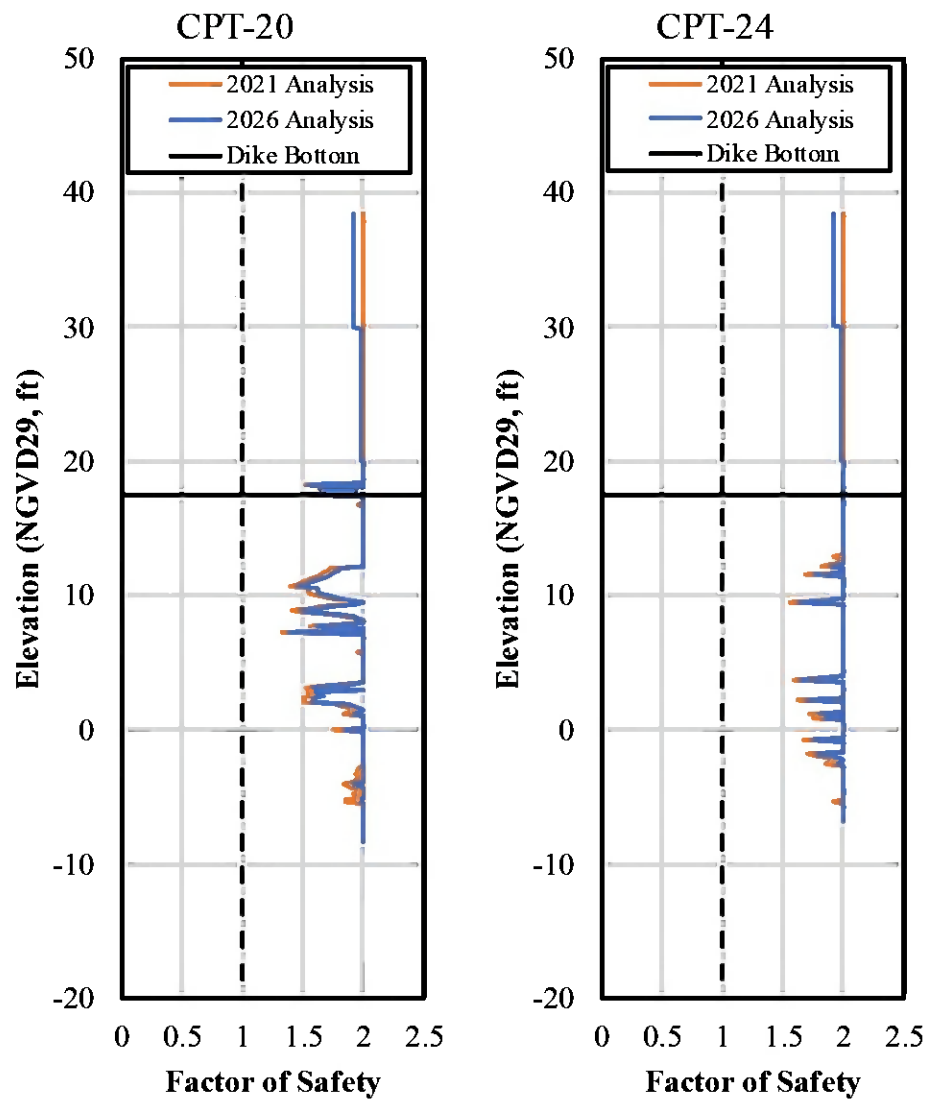


Figure 15. Comparison of Liquefaction Results for CPT-20 and CPT-24 at Slurry Pond 3&4

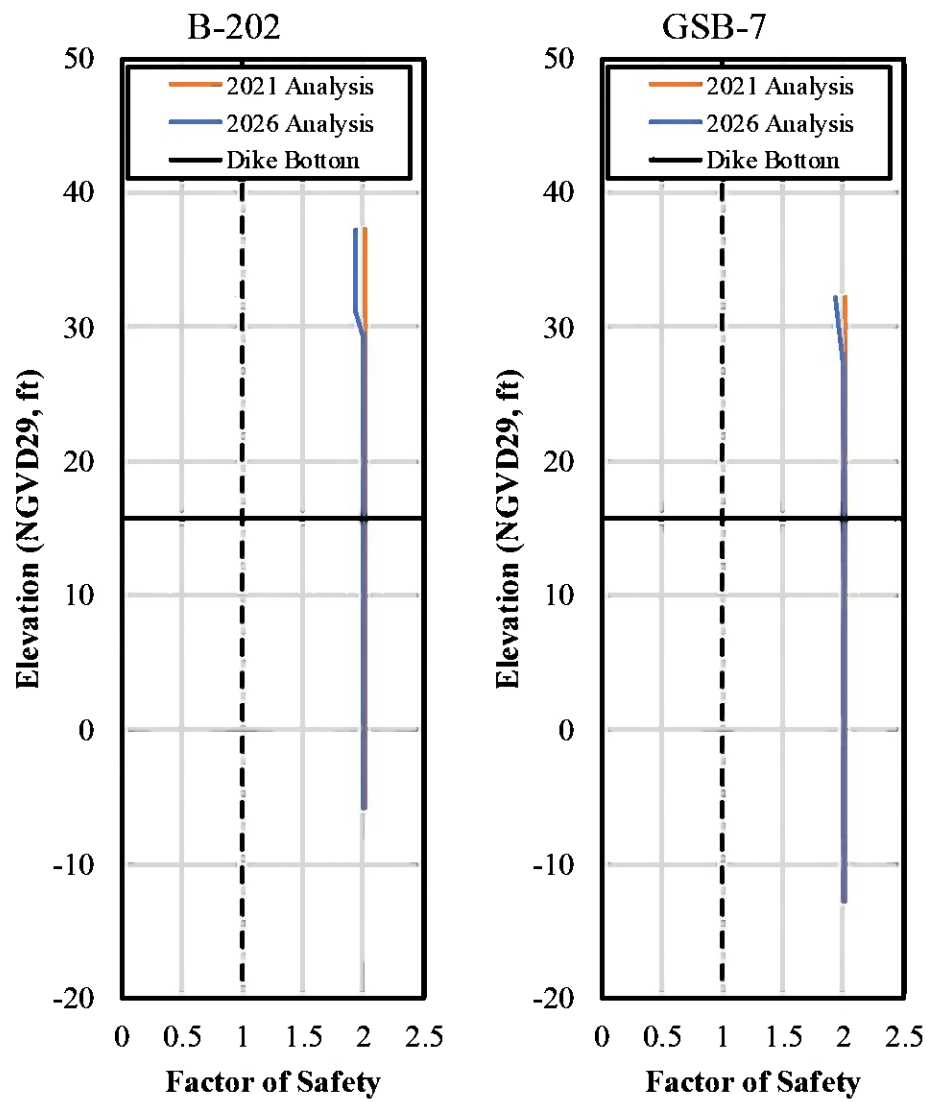


Figure 16. Comparison of Liquefaction Results for B-202 and GSB-7 at Slurry Pond 3&4

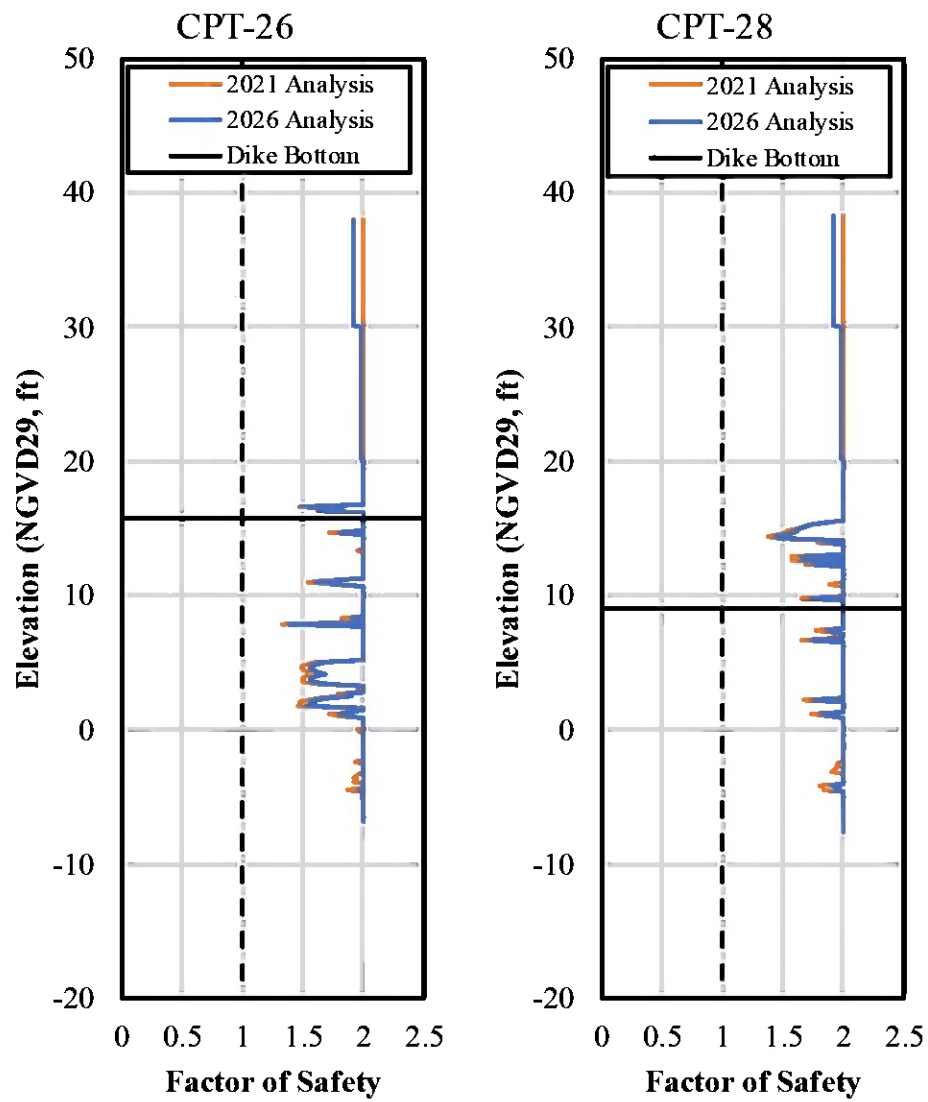


Figure 17. Comparison of Liquefaction Results for CPT-26 and CPT-28 at Slurry Pond 3&4

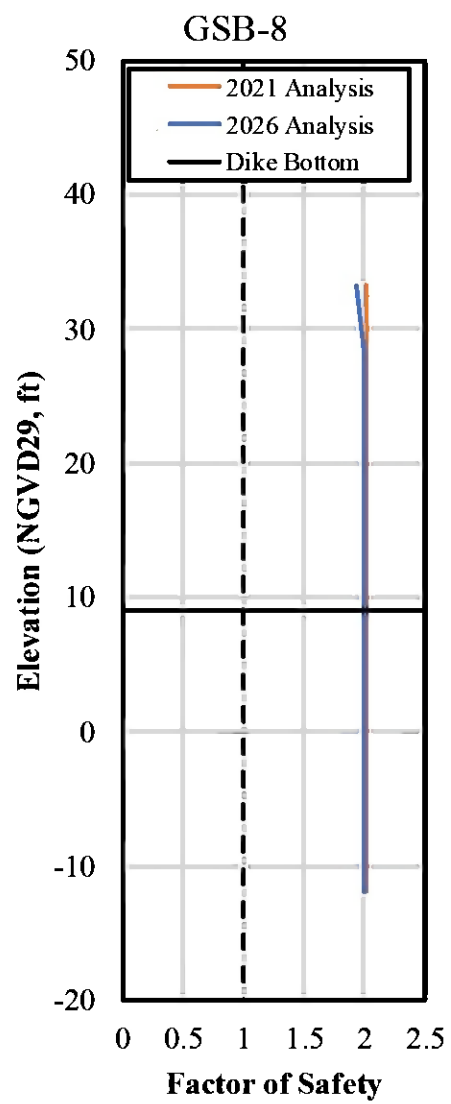


Figure 18. Comparison of Liquefaction Results for GSB-8 at Slurry Pond 3&4

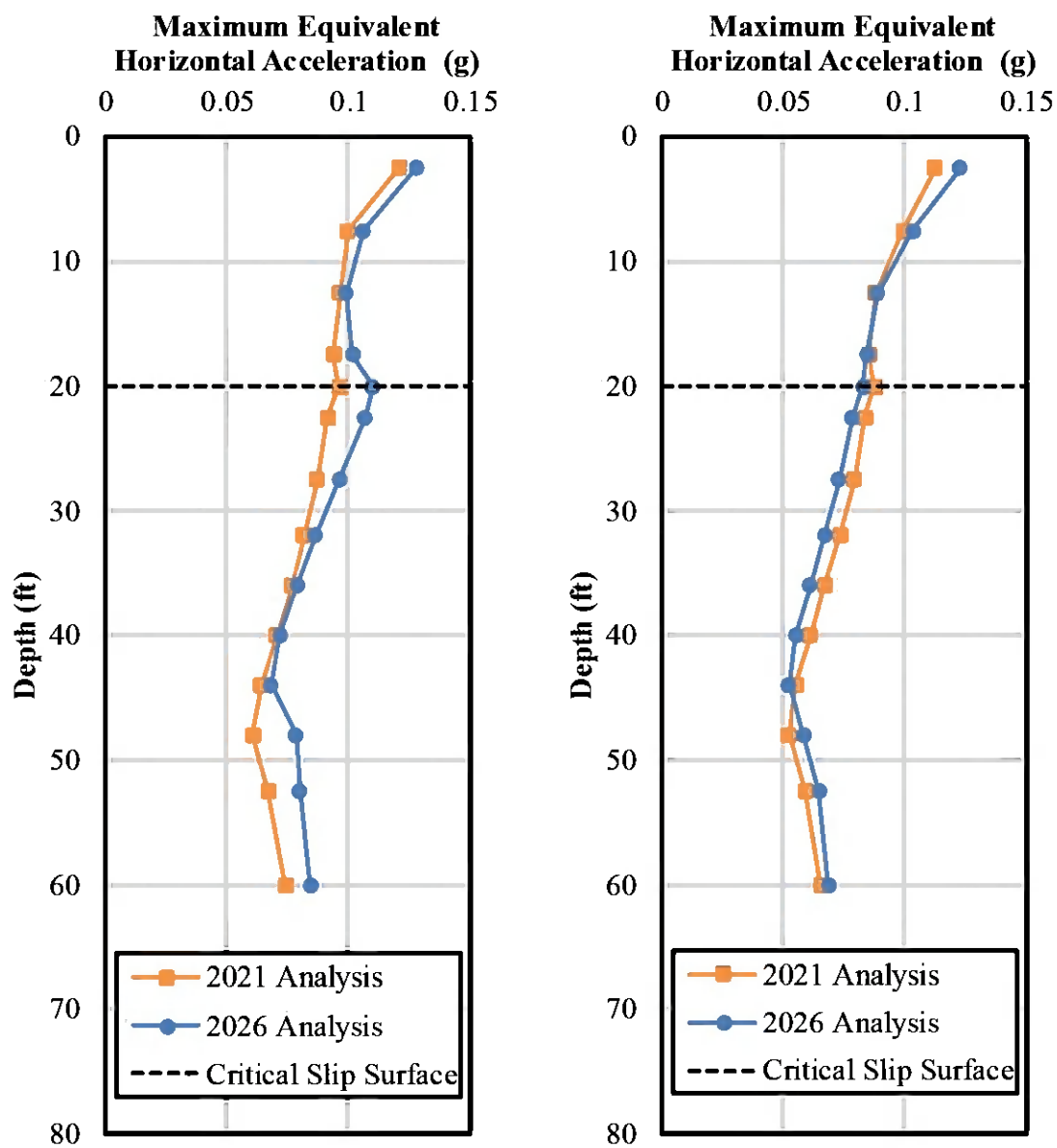


Figure 19. Comparison of MHEA at Slurry Pond 3&4

ATTACHMENTS

ATTACHMENT 1
TOPOGRAPHIC SURVEY (JUNE 2025)

NOTES

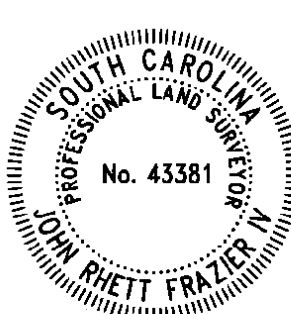
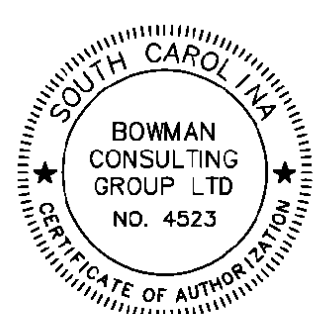
1. THE SURVEYED PROPERTY DELINEATED HEREON ARE LOCATED IN GEORGETOWN COUNTY, SOUTH CAROLINA AS TMS# 01-1009-04-00-00.
2. INFORMATION SHOWN HEREON IS BASED ON A LIDAR FLIGHT AND MAGERY PERFORMED BY BOWMAN CONSULTING GROUP, LTD. ON JUNE 30, 2025.
3. NORTH MERIDIAN INFORMATION AS SHOWN HEREON IS BASED ON SOUTH CAROLINA STATE PLANE COORDINATE SYSTEM, NAD 83 INTERNATIONAL FEET. DISTANCES SHOWN ARE HORIZONTAL "GROUND DISTANCES". VERTICAL DATUM IS BASED ON NGVD 29.
4. CONTOUR INTERVALS ARE 1 FOOT.

LEGEND

- TOPOGRAPHIC LIMITS
- INDEX CONTOUR
- INTERMEDIATE CONTOUR
- EDGE OF WATER
- LIDAR CONTROL POINT

I HEREBY STATE THAT TO THE BEST OF MY PROFESSIONAL KNOWLEDGE, INFORMATION, AND BELIEF, THE SURVEY SHOWN HEREIN WAS MADE IN ACCORDANCE WITH THE REQUIREMENTS OF THE STANDARDS OF PRACTICES MANUAL FOR SURVEYING IN SOUTH CAROLINA, AND MEETS OR EXCEEDS THE REQUIREMENTS AS SPECIFIED THEREIN.

JOHN R. FRAZIER IV
SOUTH CAROLINA PROFESSIONAL LAND SURVEYOR
LICENSE NO. 43381



SCALE: 1" = 100'

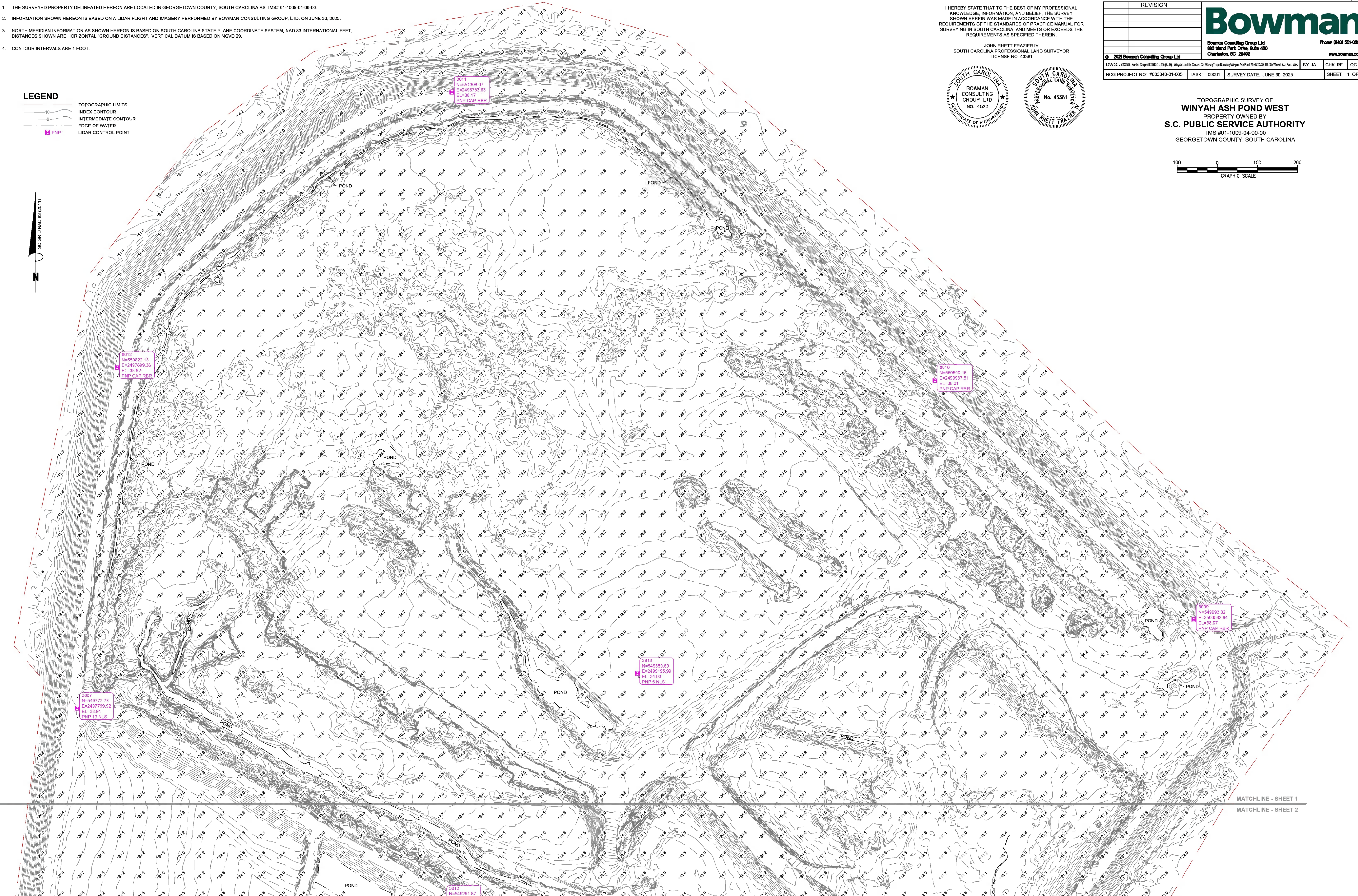
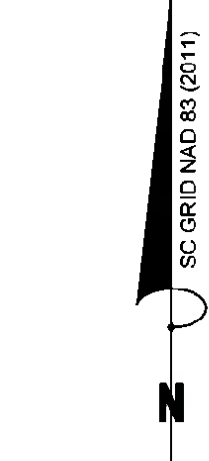
DATE: AUGUST 15, 2025

REVISION	

Bowman
Bowman Consulting Group Ltd
880 Island Park Drive, Suite 400
Charleston, SC 29402
Phone (843) 501-0333
www.bowman.com

© 2021 Bowman Consulting Group Ltd		BY: JA	CHK: RF	QC:
DWG: Y10300 - Santee Cooper/2043-31-005 (SAR) - Winyah Landfile Closure Certification/Top Boundary/Winyah Ash Pond West/3304-01-025 Winyah Ash Pond West				
BCG PROJECT NO: #003040-01-005	TASK: 00001	SURVEY DATE: JUNE 30, 2025	SHEET 1 OF 2	

TOPOGRAPHIC SURVEY OF
WINYAH ASH POND WEST
PROPERTY OWNED BY
S.C. PUBLIC SERVICE AUTHORITY
TMS #01-1009-04-00-00
GEORGETOWN COUNTY, SOUTH CAROLINA



MATCHLINE - SHEET 1
MATCHLINE - SHEET 2



ATTACHMENT 2
HYDROLOGIC AND HYDRAULIC
ANALYSIS RESULTS

Written by: A. Brand Date: 09/08/26 Reviewed by: S. Herr Date: 09/08/26
 Client: Santee Cooper Project: WGS SP 3&4 H&H Project No.: 26009458 Phase No.: 04

HYDROLOGIC AND HYDRAULIC ANALYSIS WINYAH GENERATING STATION, SLURRY POND 3 AND 4, 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 Slurry Pond 3 and 4 (Slurry Pond) 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. The Slurry Pond is regulated by the CCR Rule as an existing CCR surface impoundment. Under the CCR Rule, a high hazard ranking classification is associated with the Probable Maximum Flood (PMF). Since the Slurry Pond is a high hazard surface impoundment, the PMF is analyzed herein.

The Slurry Pond, encompassing approximately 107.2 acres (ac), is situated near the northwest corner of the Site, directly adjacent to Pennyroyal Creek. The Slurry Pond is bounded by perimeter dikes which are approximately 30 feet (ft) in height to the north, 26 ft in height to the west, and 20 ft in height to the east (Bowman Consulting Group, 2025). The Slurry Pond is separated from the West Ash Pond by a divider dike which forms the southern boundary of the Slurry Pond. Under the CCR Rule, the West Ash Pond is an inactive landfill. **Figure 1** depicts the boundary and hydraulic features associated with the Slurry Pond.

Stormwater from the exterior of the Slurry Pond is collected in Detention Ponds No. 1 and No. 2 located along the outside perimeter of the Slurry Pond. These Detention Ponds were designed to manage the 25-year (yr), 24-hour (hr) storm event (Santee Cooper, 2004). Pump Station No. 1 receives stormwater from Detention Pond No. 1, as well as Cooling Tower blowdown, and discharges to the West Low Volume Waste Pond. Pump Station No. 2 receives stormwater from Detention Pond No. 2 and discharges to the Slurry Pond. However, for this analysis, Pump Stations No. 1 and No. 2 are assumed to be non-operational due to temporary loss of power during a large storm event.

Written by: A. Brand Date: 09/08/26 Reviewed by: S. Herr Date: 09/08/26
 Client: Santee Cooper Project: WGS SP 3&4 H&H Project No.: 26009458 Phase No.: 04

A Floating Pump Station equipped with two Tsurumi GSZ-4-45-4 submersible pumps normally conveys water from the Slurry Pond directly to the West Low Volume Waste Pond. The capacity of these pumps operating in parallel is 3,100 gallons per minute (gpm) at the normal operating elevation of 19.6 ft National Geodetic Vertical Datum of 1929 (NGVD 29) when pumping directly to the West Low Volume Waste Pond. Piping is valved such that the Floating Pump Station may convey water to Pump Station No. 1. However, for this analysis, the Floating Pump Station is assumed to be non-operational due to temporary loss of power during a large storm event.

The West Ash Pond is no longer hydraulically connected to the Slurry Pond and is not included in this evaluation.

2 METHODOLOGY AND INPUT PARAMETERS

Stormwater runoff volumes and associated discharges to the Slurry Pond 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 the Slurry Pond.

Rainfall

The 72-hr duration precipitation event was used in this analysis. The rainfall depth corresponding to the 72-hr duration precipitation event for the PMF for the Site is 51 inches (in.) (NOAA, 1978). 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 the Slurry Pond is 107.2 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. The Slurry Pond was assumed to be approximately 94% ash (CN = 86) and 6% water (CN = 100) (Weighted CN = 87). The contributing watershed area and CN is summarized in **Table 1** and was directly input into the *HydroCAD* model.

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 Client: Santee Cooper Project: WGS SP 3&4 H&H Project No.: 26009458 Phase No.: 04

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 the Slurry Pond 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):

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

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 the Slurry Pond. 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 4.38 in. (NOAA, 2021). The parameters used to model sheet flow within the Slurry Pond 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).

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

Written by: A. Brand Date: 09/08/26 Reviewed by: S. Herr Date: 09/08/26
 Client: Santee Cooper Project: WGS SP 3&4 H&H Project No.: 26009458 Phase No.: 04

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 the Slurry Pond. The parameters used to describe shallow concentrated flow within the Slurry Pond are presented in **Table 2**.

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

Inflows

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

Storage Capacities

Pond 1P models the available storage volume within the Slurry Pond. The available stormwater storage volume in the Slurry Pond between elevations 4 ft and 36 ft NGVD 29 was calculated by developing an area-volume curve based on topographic data (Bowman Consulting Group, 2025). The lowest available contour within the Slurry Pond is 4 ft NGVD 29. The minimum crest elevation of the Slurry Pond perimeter dikes is 36 ft NGVD 29. 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 contour and multiplying by the change in elevation between each contour. The cumulative storage volume of Slurry Pond between these elevations is 1,256.5 acre-feet (ac-ft). The area-volume data are presented in **Table 3**.

The Slurry Pond is maintained at a normal operating elevation of 19.6 ft NGVD 29 by the Floating Pump Station. As a result, the starting elevation of Pond 1P is set to 19.6 ft NGVD 29.

Outlet Structures

For this analysis, the Floating Pump Station is assumed to be non-operational due to temporary loss of power during a large storm event.

Written by: A. Brand Date: 09/08/26 Reviewed by: S. Herr Date: 09/08/26
Client: Santee Cooper Project: WGS SP 3&4 H&H Project No.: 26009458 Phase No.: 04

3 RESULTS OF ANALYSES

The resulting peak water surface elevation and water volume for the PMF are shown in **Table 4**. The Slurry Pond will effectively contain the PMF and maintain a freeboard of 7.7 ft. This hydrologic and hydraulic analysis demonstrates that the Slurry Pond contains the PMF assuming the Slurry Pond is maintained at the normal operating elevation of 19.6 ft NGVD 29.

4 REFERENCES

- Bowman Consulting Group. (2025). *Topographic Survey of Winyah Ash Pond West*.
- HydroCAD. (2024). *HydroCAD Stormwater Modeling System*. Version 10.20-3h.
- NOAA. (1978). *Hydrometeorological Report No. 51: Probable Maximum Precipitation Estimates, United States East of the 105th Meridian*.
- NOAA. (2021). *Precipitation-Frequency Atlas of the United States*. Atlas 14, Volume 2, Version 3.0.
- NRCS. (2010). *National Engineering Handbook: Part 630 Hydrology*.
- Santee Cooper. (2004). *Pump Station Design Report – Winyah Generating Station Redirect Drainage Project*.
- SCS. (1982). *Technical Release Number 20 (TR-20)*. National Technical Information Service.
- SCS. (1986). *Technical Release Number 55 (TR-55)*. National Technical Information Service.

TABLES

Table 1. Land Use Conditions

Drainage Area ID	Land Use Description	CN	Area (acres)	Total Area (acres)	Weighted CN
Slurry Pond	CCR	86	100.8	107.2	87
	Water	100	6.4		

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 {Tc} (min)
	Surface Description	Manning's No. {n}	Flow Length {L} (ft)	2-Yr, 24-Hr Rainfall {I} (in.)	Land Slope {S} (ft/ft)	Time of Concentration from Sheet Flow {Tc, sheet} (min)	Surface Description	Velocity Factor (ft/s)	Flow Length {L} (ft)	Land Slope {S} (ft/ft)	Time of Concentration from Shallow Concentrated Flow {Tc, scf} (min)	
Slurry Pond	Smooth	0.020	100	4.38	0.0086	2.3	Unpaved	16.1	1,002	0.013	9.1	11.4

Table 3. Slurry Pond Stage Storage

Elevation (ft) (NGVD 29)	Area (ac)	Incremental Volume (ac-ft)	Cumulative Volume (ac-ft)
4	0.0	0.000	0.0
5	0.3	0.1	0.1
6	1.3	0.8	0.9
7	2.2	1.8	2.7
8	3.1	2.7	5.3
9	4.2	3.7	9.0
10	5.7	5.0	13.9
11	8.4	7.1	21.0
12	10.8	9.6	30.6
13	13.0	11.9	42.5
14	15.4	14.2	56.7
15	17.9	16.6	73.3
16	21.5	19.7	93.0
17	26.8	24.1	117.1
18	30.1	28.4	145.6
19	32.8	31.5	177.0
20	35.7	34.3	211.3
21	39.4	37.6	248.9
22	43.8	41.6	290.5
23	47.4	45.6	336.1
24	50.7	49.1	385.2
25	54.0	52.4	437.6
26	57.8	55.9	493.5
27	61.4	59.6	553.1
28	65.3	63.3	616.5
29	69.4	67.3	683.8
30	72.6	71.0	754.8
31	75.8	74.2	829.0
32	79.5	77.6	906.6
33	83.7	81.6	988.2
34	87.6	85.7	1,073.9
35	91.0	89.3	1,163.3
36	95.3	93.2	1,256.5

Table 4. H&H Calculation Results

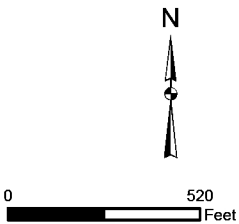
Pond ID	Storm Event	Normal Operating Elevation (ft) (NGVD 29)	Peak Water Surface Elevation (ft) (NGVD 29)	Water Volume (ac-ft)	Freeboard (ft)
Slurry Pond	PMF	19.6	28.3	440.0	7.7

FIGURES

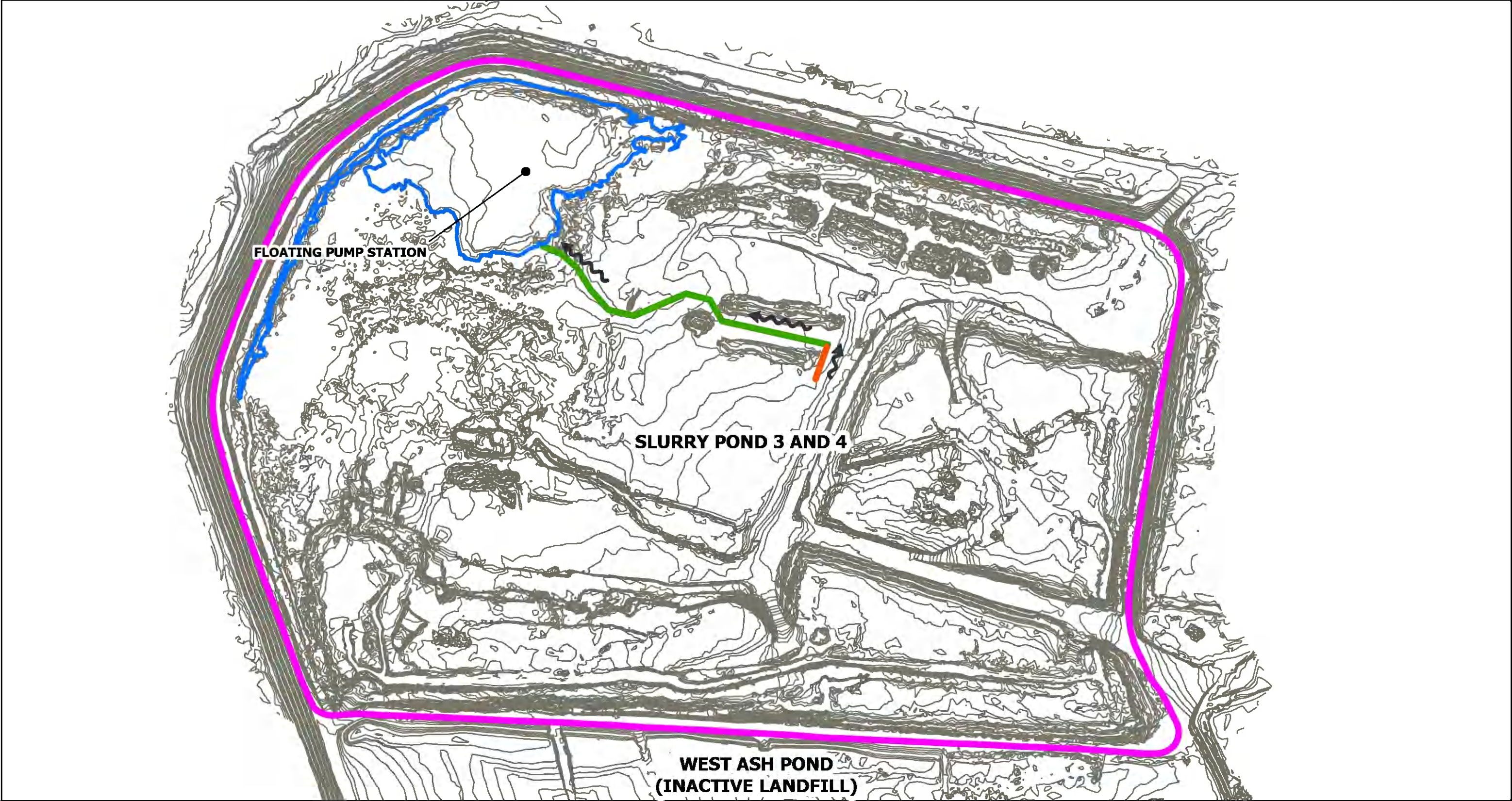


Legend
Pond Boundary

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



WINYAH GENERATING STATION SLURRY POND 3 AND 4	
Geosyntec consultants	Figure 1
26009458	September 2026



Legend

- Pond Boundary
- Normal Pool
- Shallow Concentrated Flow
- Sheet Flow
- Flow Direction

Note:
1. Topographic survey information collected by Bowman Consulting Group, dated 6-30-2025.



0 260
Feet

WINYAH GENERATING STATION
SLURRY POND 3 AND 4

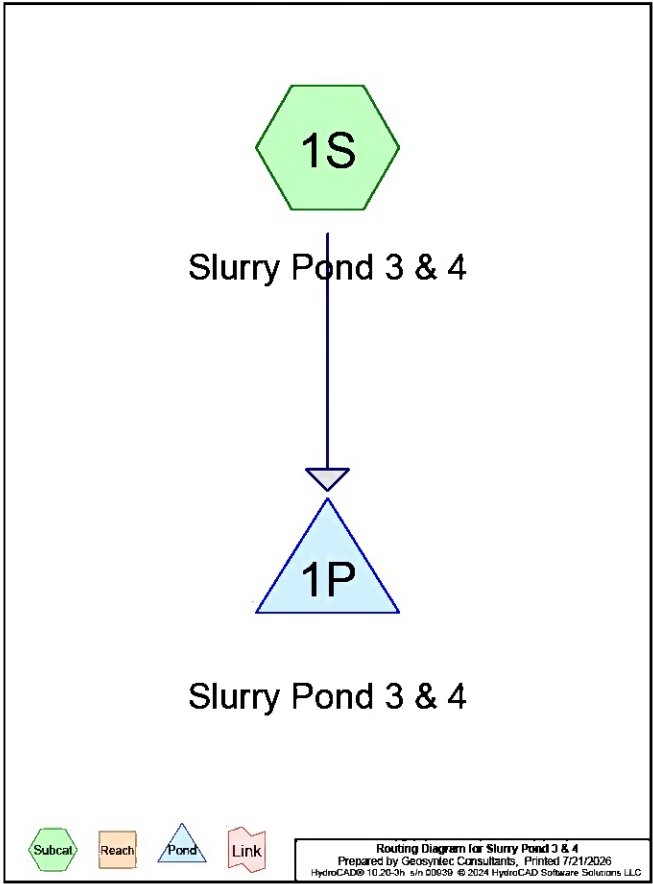
Geosyntec
consultants

26009458 September 2026

Figure
2

ATTACHMENTS

ATTACHMENT A



Area Listing (all nodes)		
Area (acres)	CN	Description (subcatchment-numbers)
107.205	87	(1S)
107.205	87	TOTAL AREA

Rainfall Events Listing (selected events)								
Event#	Event Name	Storm Type	Curve	Mode	Duration (hours)	B/B	Depth (inches)	AMC
1	PMF (72-HR, 10-SQ MI)	Type III 24-hr		Scale	72.00	1	51.00	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	
107.205	Other	1S
107.205		TOTAL AREA

Ground Covers (all nodes)							
HSG-A (acres)	HSG-B (acres)	HSG-C (acres)	HSG-D (acres)	Other (acres)	Total (acres)	Ground Cover	Subcatchment Numbers
0.000	0.000	0.000	0.000	107.205	107.205		1S
0.000	0.000	0.000	0.000	107.205	107.205	TOTAL AREA	

Time span=0.00-600.00 hrs, dt=0.050 hrs, 12001 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Sim-Route method - Pond routing by Sim-Route method

Subcatchment1S: Slurry Pond 3 & 4Runoff Area=107.205 ac 0.00% Impervious Runoff Depth=49.25"
Flow Length=1,102' Tc=11.4 min CN=87 Runoff=1,930.76 cfs 439.984 af

Pond 1P: Slurry Pond 3 & 4Peak Elev=28.31' Storage=637.580 af Inflow=1,930.76 cfs 439.984 af
Outflow=0.00 cfs 0.000 af

Total Runoff Area = 107.205 ac Runoff Volume = 439.984 af Average Runoff Depth = 49.25"
100.00% Pervious = 107.205 ac 0.00% Impervious = 0.000 ac

Summary for Subcatchment 1S: Slurry Pond 3 & 4

Runoff = 1,930.76 cfs @ 36.16 hrs, Volume= 439.984 af, Depth=49.25"
Routed to Pond 1P: Slurry Pond 3 & 4

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-600.00 hrs, dt= 0.050 hrs
Type III 24-hr scaled to 72.00 hrs PMF (72-HR, 10-SQ MI) Rainfall=51.00"

Area (ac)	CN	Description			
107.205	87				
107.205		100.00% Pervious Area			
Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
2.3	100	0.0086	0.71		Sheet Flow, Sheet Flow n= 0.020 P2= 4.38"
9.1	1,002	0.0130	1.84		Shallow Concentrated Flow, Shallow Concentrated Flow Kv= 16.1 lps
11.4	1,102	Total			

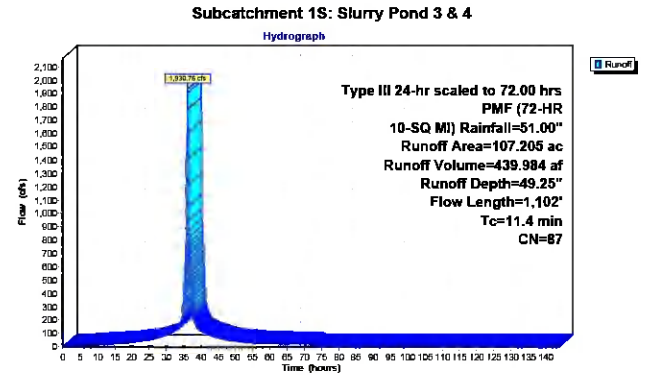
Summary for Pond 1P: Slurry Pond 3 & 4

Inflow Area = 107.205 ac, 0.00% Impervious, Inflow Depth = 49.25" for PMF (72-HR, 10-SQ MI) event
Inflow = 1,930.76 cfs @ 36.16 hrs, Volume= 439.984 af
Outflow = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af, Atten= 100%, Lag= 0.0 min

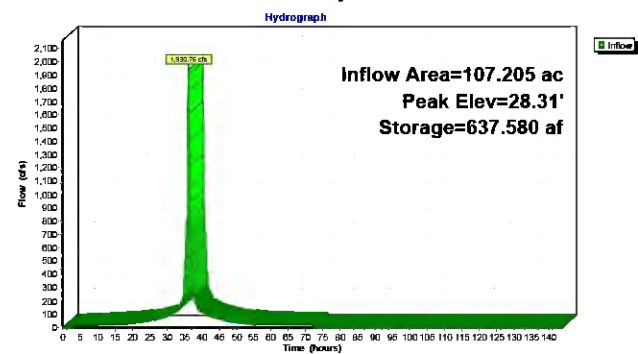
Routing by Sim-Route method, Time Span= 0.00-600.00 hrs, dt= 0.050 hrs
Starting Elev= 19.60' Surf Area= 34.588 ac Storage= 197.596 af
Peak Elev= 28.31' @ 72.70 hrs Surf Area= 66.550 ac Storage= 637.580 af (439.984 af above start)

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	4.00'	1,256.476 af	Custom Stage Data (Prismatic), listed below
Elevation (feet)	Surf Area (acres)	Inc. Store (acre-feet)	Cum. Store (acre-feet)
4.00	0.008	0.000	0.000
5.00	0.264	0.136	0.136
6.00	1.261	0.762	0.899
7.00	2.243	1.752	2.650
8.00	3.121	2.682	5.333
9.00	4.184	3.653	8.985
10.00	5.743	4.963	13.949
11.00	8.389	7.066	21.015
12.00	10.764	9.577	30.591
13.00	13.027	11.895	42.487
14.00	15.381	14.204	56.691
15.00	17.904	16.643	73.333
16.00	21.466	19.685	93.018
17.00	26.788	24.127	117.145
18.00	30.057	28.423	145.568
19.00	32.844	31.450	177.018
20.00	35.750	34.297	211.315
21.00	39.448	37.599	248.914
22.00	43.780	41.614	290.528
23.00	47.449	45.615	336.143
24.00	50.731	49.090	385.232
25.00	54.025	52.378	437.610
26.00	57.817	55.921	493.531
27.00	61.420	59.619	553.150
28.00	65.254	63.337	616.487
29.00	69.389	67.321	683.808
30.00	72.592	70.990	754.799
31.00	75.828	74.210	829.009
32.00	79.452	77.640	906.649
33.00	83.748	81.600	988.249
34.00	87.643	85.695	1,073.944
35.00	91.036	89.339	1,163.284
36.00	95.348	93.192	1,256.476



Pond 1P: Slurry Pond 3 & 4



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: SLURRY POND 3 AND 4

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
(peer reviewer) Printed Name Prabesh Bhandari Date
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. Tasthan Date: 9-18-2026
 Client: Santee Cooper Project: Winyah Generating Station Project No.: 26009458 Phase No.: 003

SAFETY FACTOR ASSESSMENT: SLURRY POND 3 AND 4

INTRODUCTION

This calculation package was prepared as Attachment 3 to the *2026 Periodic Safety Factor Assessment: Slurry Pond 3 and 4* (2026 Safety Factor Assessment Report) and presents the slope stability analyses for the critical portion of the Slurry Pond 3 & 4 (Slurry Pond) perimeter dikes at Winyah Generating Station (WGS), Georgetown County, South Carolina. On 17 April 2015, the United States Environmental Protection Agency (USEPA) published the CCR Rule (40 Code of Federal Regulations [CFR] Parts 257 and 261). Under the CCR Rule, the Slurry Pond 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 critical portion of the Slurry Pond perimeter dikes satisfies 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 Slurry Pond 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 Slurry Pond 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[®], 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 (e.g., non-circular slip surfaces) and the non-rotational mode (i.e., block slip surfaces) were considered for the stability analyses presented in this calculation package. SLIDE[®] generates potential slip surfaces, calculates the FS for each of these surfaces, and identifies the critical slip surface with the lowest calculated FS. The critical slip surfaces are reported in the results of this calculation package. 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

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expected to experience less deformation than the selected allowable displacement and meet the requirements of the CCR Rule.

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 section evaluated as part of this safety factor assessment.

External Geometry

The heights of the Slurry Pond perimeter dikes are approximately 25 to 30 feet (ft) to the north and west, approximately 20 to 25 ft to the east, and approximately 15 ft to the south. 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 22-ft wide (Thomas and Hutton, 2012). To the northeast, east, and southeast of the perimeter dikes, a drainage channel has been excavated as part of WGS's stormwater management plan.

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 developed based on the topographic survey prepared by Thomas and Hutton (2012), the original design contours, and the design of the western rim ditch.

For the 2021 Safety Factor Assessment Report, only the critical cross sections identified in the 2016 Safety Factor Assessment were analyzed. The same three critical cross sections are analyzed for the 2026 Safety Factor Assessment Report. In the 2016 Safety Factor Assessment for the Slurry Pond, Cross Sections C, D, and E had relatively lower calculated FSs for the static and seismic slope stability analyses. Therefore, updated slope stability analyses were performed for Cross Sections C, D, and E only as part of the 2026 Safety Factor Assessment Report (Figure 1). Updated topographic survey data from June 2025 was also incorporated into these cross sections (Bowman Consulting Group, 2025).

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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 and 2016 subsurface investigations. Santee Cooper personnel indicated that no additional geotechnical investigations were conducted in the area of the Slurry Pond 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 retained ash and perimeter dike, and downstream toe were determined as described below.

Maximum Normal Storage Pool Condition: As described within the 2016 Safety Factor Assessment Report (Geosyntec, 2016), the surface water level in the Slurry Pond was maintained at an elevation of 19.6 ft National Geodetic Vertical Datum of 1929 (NGVD29) by a Floating Pump Station. Currently, the free water level within the Slurry Pond has been maintained below elevation 19.6 ft NGVD29 by the Floating Pump Station. Based on the annual inspection reports, the free water level has remained below elevation 19.6 ft NGVD29 since 2022. Nevertheless, for conservatism, elevation 19.6 ft NGVD29 was selected as the “Maximum Normal Storage Pool” within the Slurry Pond for the static and seismic slope stability analyses herein. Details of the H&H analyses are provided in a document titled “*Inflow Design Flood Control System Plan - Slurry Pond 3 and 4*” and the H&H analysis results are included as Attachment 2 to the 2026 Safety Factor Assessment Report. Groundwater level measurements previously provided by Santee Cooper were reviewed and compared with the 2025 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 the Slurry Pond has been classified as a “High Hazard Potential” surface impoundment (Geosyntec, 2021), the Probable maximum flood (PMF) 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 surge pool” elevation within the Slurry Pond was

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Client: <u>Santee Cooper</u>	Project: <u>Winyah Generating Station</u>	Project No.: <u>26009458</u>	Phase No.: <u>003</u>

established at 28.33 ft NGVD 29. Details of the H&H analyses are provided in a document titled “*Inflow Design Flood Control System Plan: Slurry Pond 3 and 4*” and the H&H analysis results are included as Attachment 2 to the 2026 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.

Final Cross Section Geometry

The final geometric models implemented within SLIDE[®] for Cross Sections C, D, and E are provided in Figures 2 through 7 for the maximum normal and surge storage pool conditions.

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 were evaluated in the 2016 Safety Factor Assessment Report (Geosyntec, 2016) using in-situ and laboratory data collected in the vicinity of the Slurry Pond. Geosyntec assumed the site subsurface conditions have generally remained unchanged. Table 1 provides a summary of the material properties selected for the evaluated cross sections as part of the 2021 Safety Factor Assessment. The interpretation and selection of properties for Cross Sections C through E are summarized in the 2016 Safety Factor Assessment Report. Drained shear strength parameters for cross section-specific dike fill and sandy foundation soil were predominantly developed from in-situ measurements (i.e., SPT N-values, etc.).

It was assumed that seismic waves generated during the design seismic event would load dike fill and foundation soils rapidly enough to develop elevated pore pressures and induce an undrained loading condition within the clayey soils. In accordance with recommendations made by Hynes-Griffin and Franklin (1984), the selected undrained shear strength values were conservatively reduced by 20 percent for the seismic slope stability analyses to account for potential cyclic degradation during an earthquake at the Site.

Seismic Loading and Allowable Displacement

An evaluation of the seismic hazard for WGS and the site response analysis for the Slurry Pond perimeter dikes are presented in Section 4 of the 2026 Safety Factor Assessment Report. Maximum

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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 coefficients for the seismic slope stability analyses. MHEA values of 0.11g and 0.083g were selected for Profile 1 (Cross Sections D and E) and Profile 2 (Cross Section C), respectively.

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.) (30.5 centimeters [cm]) was selected for the Slurry Pond 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 0.5, as shown in Figure 8. Thus, k_h values of 0.041, 0.055, and 0.055 were computed for Cross Sections C, D, and E, respectively.

RESULTS

The safety factor evaluation for Cross Sections C, D, and E was performed according to the methodology and parameters described within this calculation package, and the results are summarized within Table 2. Computed safety factors were found to exceed the minimum safety factors required by §257.73(e)(1) of the CCR Rule. Figures 9 through 17 depict the safety factors for Cross Sections C through E. While both rotational mode (i.e., circular slip surface mode) and non-rotational (i.e., block slip surface mode) were considered in the analyses, the rotational failure mode was consistently more critical than the non-rotational failure mode.

CONCLUSIONS

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

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TABLES

Table 1. Selected Material Parameters for Analysis

Material	Total Unit Weight (pcf)	Drained Parameters		Undrained Parameters ^[2]	
		ϕ' (°)	c' (psf)	S_u/σ'_{vo}	$S_{u,min}$ (psf)
Dike Fill Material ^[1]	125	33	100	0.65	100
Foundation Materials (Clayey) ^[1]	100	28	0 / 50 ^[5]	0.35 to 0.40 ^[3]	0
Foundation Materials (Sandy)	115	32	0	-	-
Riprap Buttress	150	45	0	-	-
Chicora Member	130	50	0	-	-
Williamsburg Formation Clay	105	50	0	-	-
FGD Residuals ^[1,4]	95	40	0	0.5	0

Notes:

1. In accordance with recommendations made by Hynes-Griffin and Franklin (1984), the shear strengths of Dike Fill Material, Foundation Soils (Clayey), and FGD Residuals were conservatively reduced during pseudo-static analyses by 20% to account for cyclic degradation during an earthquake.
2. Undrained strength parameters for clayey foundation soils were applied for seismic slope stability case only.
3. Cross Section C was assigned a $S_u/\sigma'_{vo} = 0.35$ while Cross Sections D and E were assigned a $S_u/\sigma'_v = 0.40$ based on the interpretation of CPT soundings in the vicinity of each cross section.
4. FGD residuals were modeled using undrained shear strength ratio in each case.
5. A cohesion intercept of 50 psf was applied to “clayey foundation soils” for Cross Section E under “Maximum Normal Pool” and “Maximum Surcharge Pool” conditions to address a localized slip surface, which was identified as an artifact of conservatively modeling the clayey foundation soil as a purely frictional material in this area.

Table 2. Summary of Calculated Factors of Safety (FS) ^[2]

Factor of Safety Case	Target FS	Cross Section C	Cross Section D	Cross Section E
Static - Maximum Normal Storage Pool	1.50	1.76	<i>1.69</i>	1.85
Static - Maximum Surcharge Pool	1.40	1.76	1.55	<i>1.49</i>
Seismic - Maximum Normal Storage Pool	1.00	1.17	1.17	<i>1.11</i>
Liquefaction Slope Stability ^[1]	1.20	N/A	N/A	N/A

Notes:

1. The liquefaction safety factor was not evaluated since dike fill soils and foundation soils beneath the dike fill were not found to be susceptible to liquefaction under the design earthquake.
2. Only critical failure surfaces passing through the perimeter dikes were considered.
3. The lowest computed safety factor for each analysis case is italicized.

FIGURES

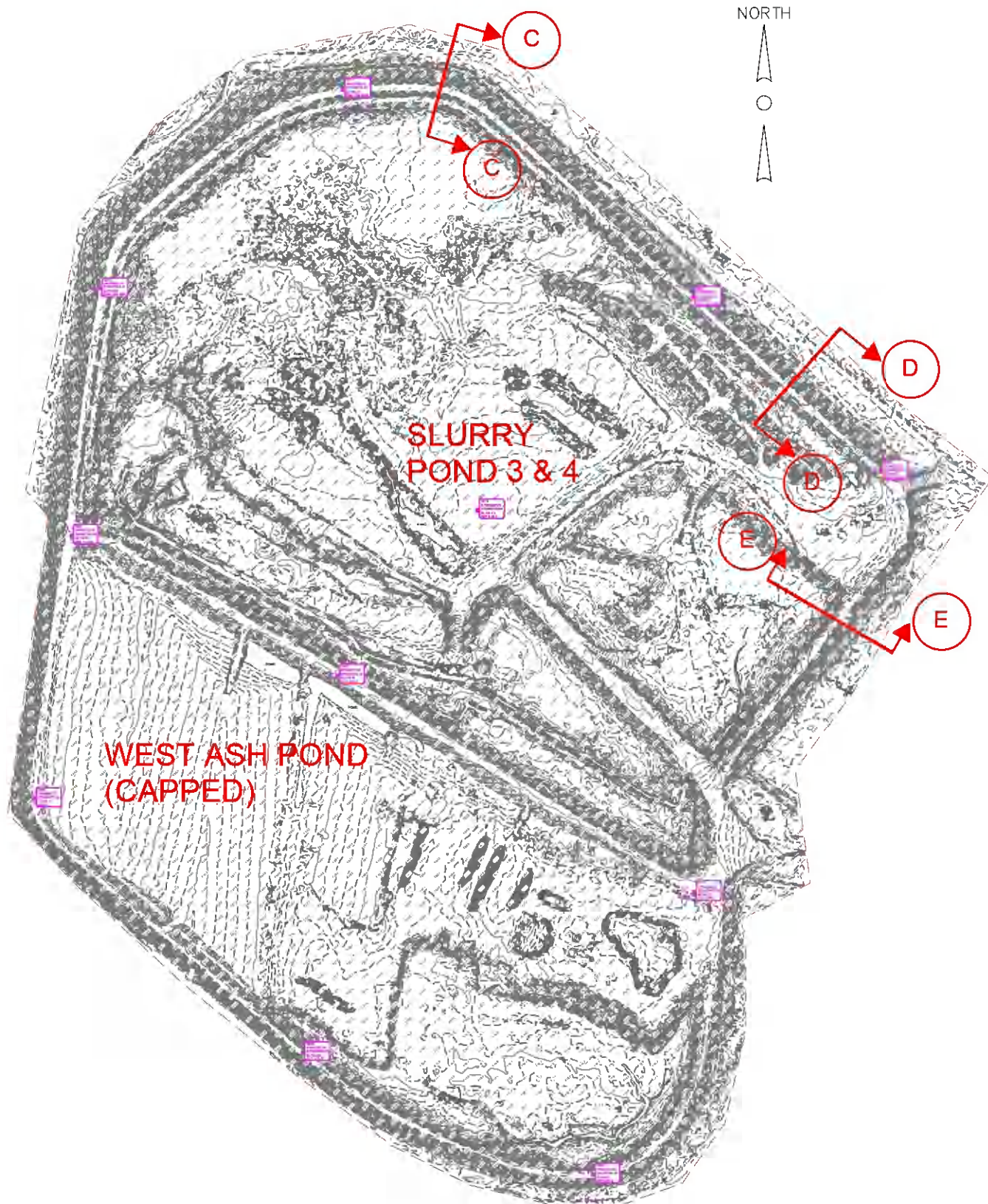


FIGURE 1. CROSS SECTION LOCATION PLAN

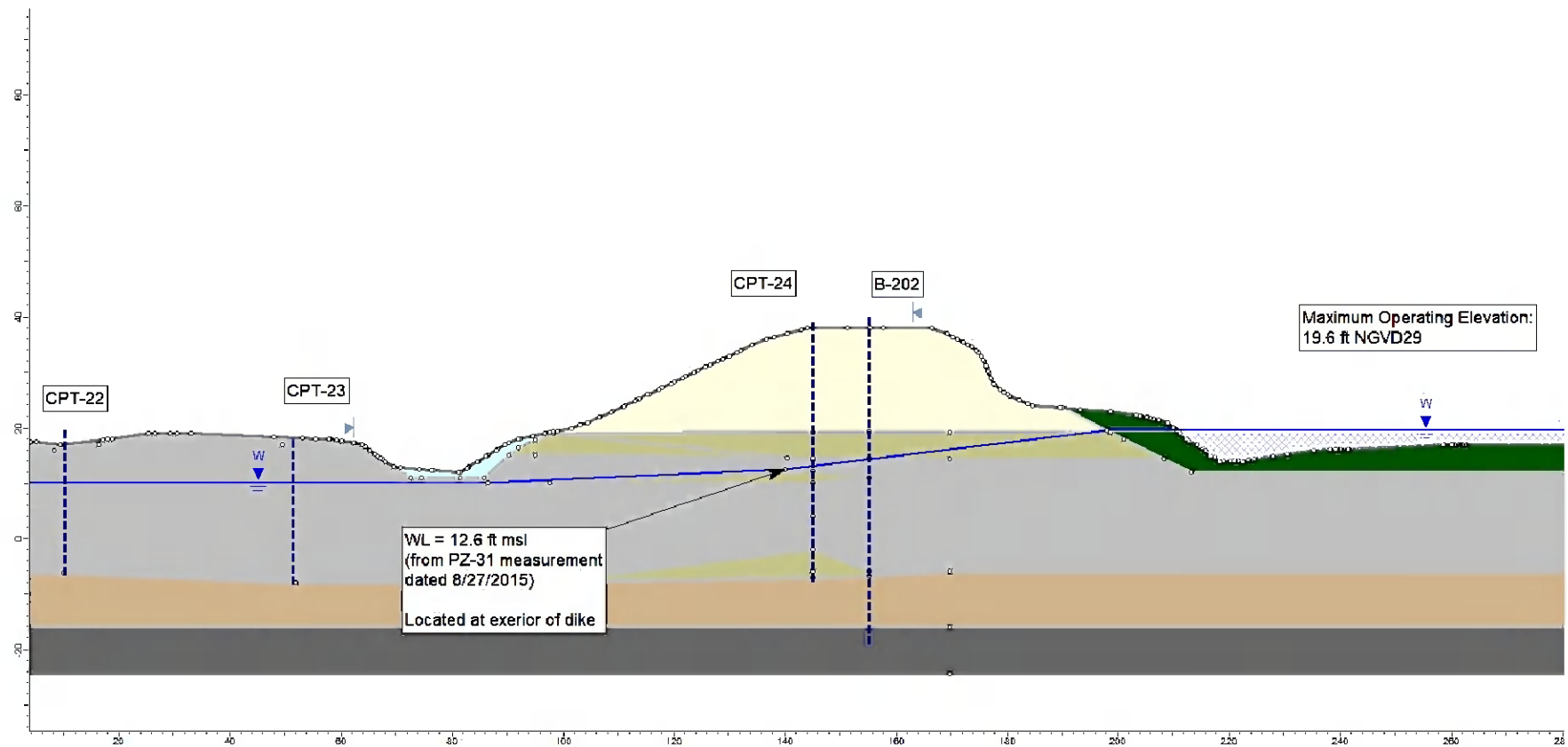


Figure 2. Cross Section C Geometry for Maximum Normal Storage Pool

Note:

1. A 2H:1V rip-rap buttress is constructed within the stormwater channel at the base on the perimeter dikes within this area. The base of the stormwater channel is also lined with 1-ft of riprap.

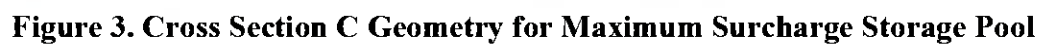


Figure 3. Cross Section C Geometry for Maximum Surcharge Storage Pool

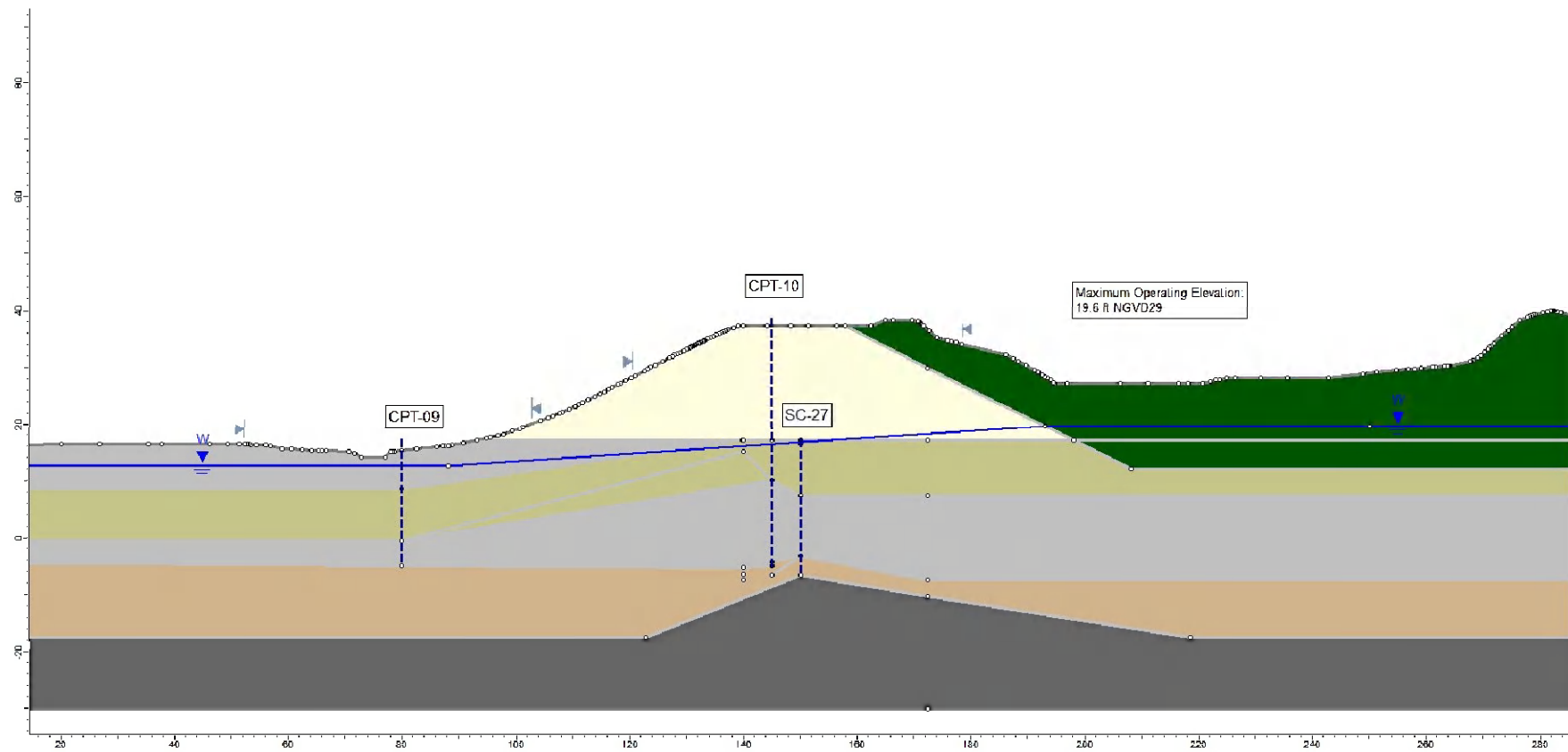


Figure 4. Cross Section D Geometry for Maximum Normal Storage Pool

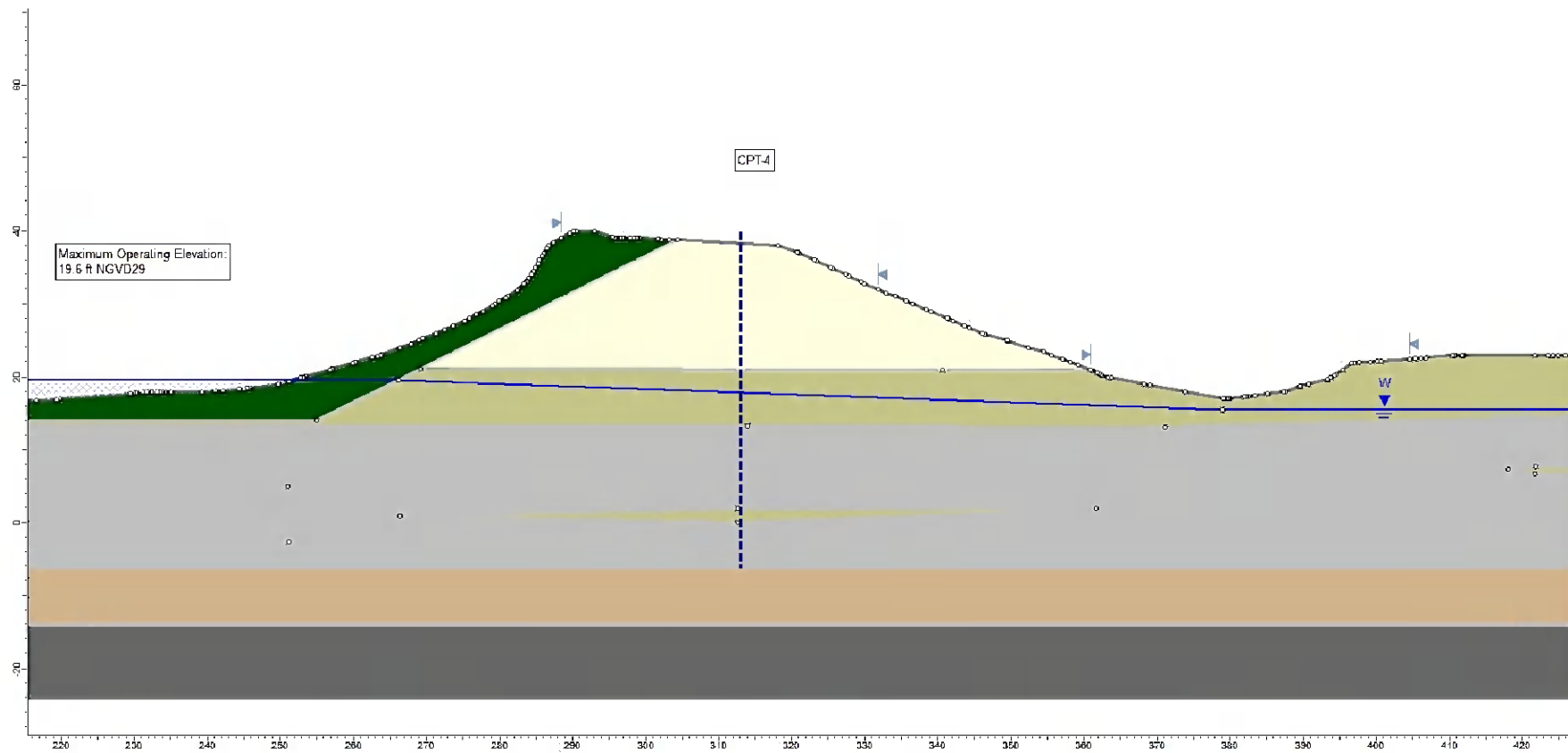


Figure 6. Cross Section E Geometry for Maximum Normal Storage Pool

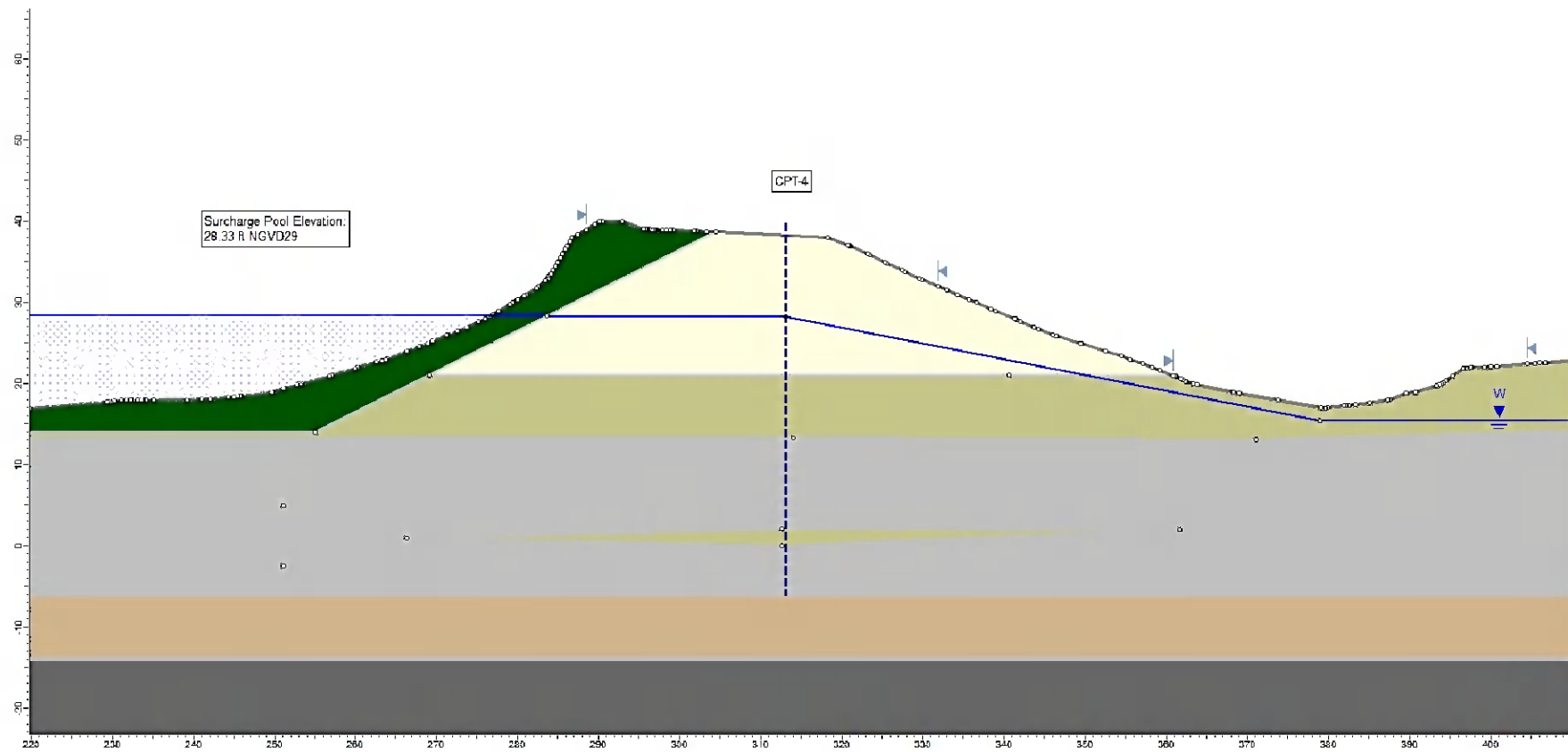


Figure 7. Cross Section E Geometry for Maximum Surcharge Storage Pool

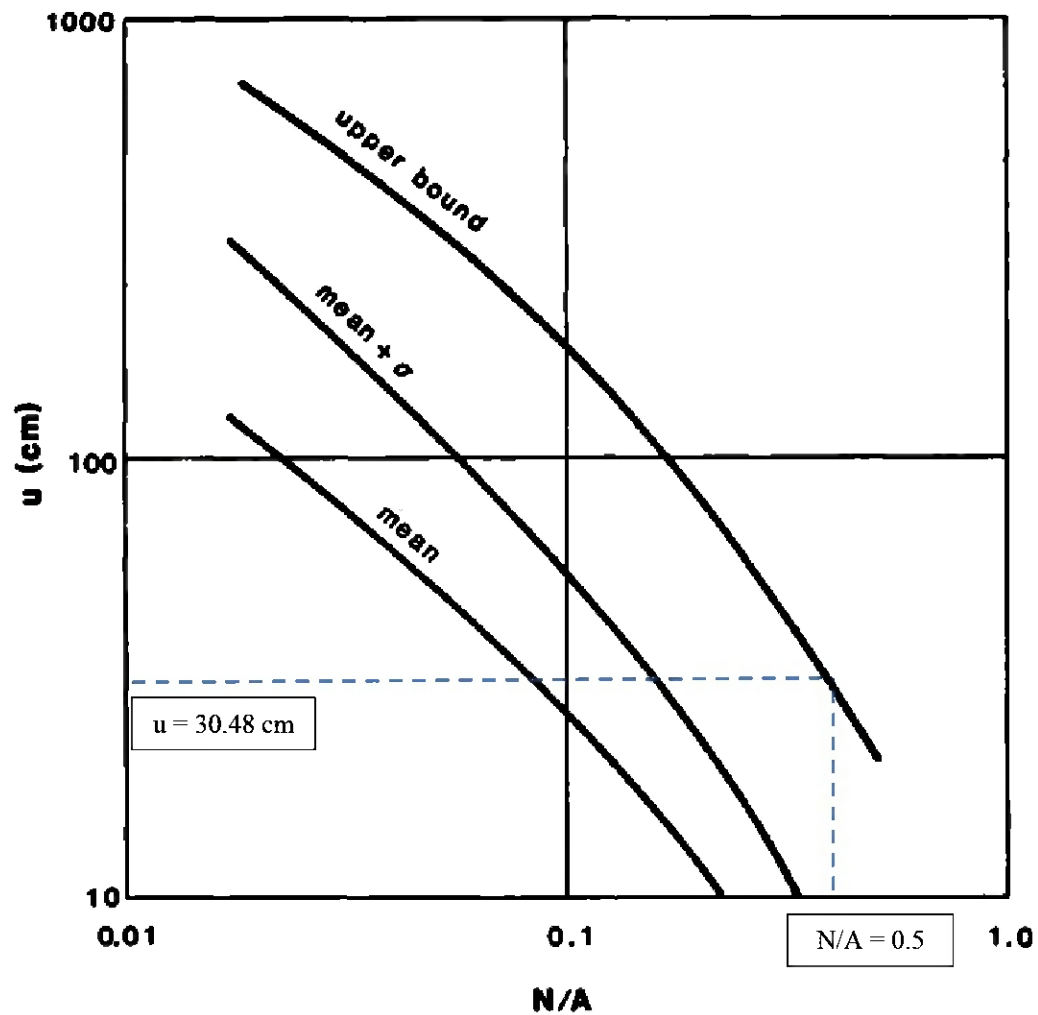


**Figure 8. Allowable Deformation (u) vs. N/A
(from Figure 7 of Hynes-Griffin and Franklin, 1984)**

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.

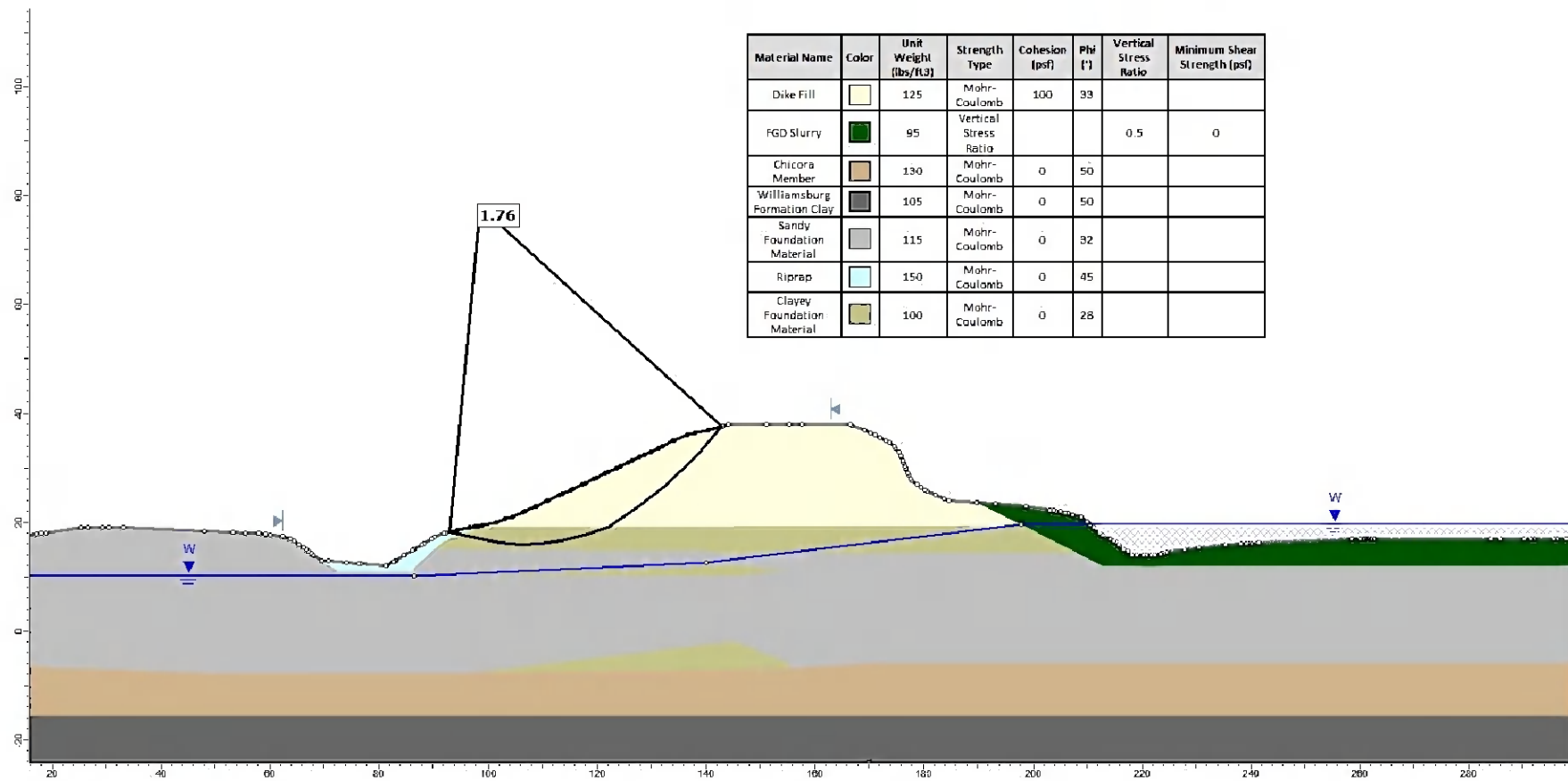


Figure 9. Calculated Factor of Safety for Cross Section C: Static Factor of Safety – Maximum Normal Storage Pool

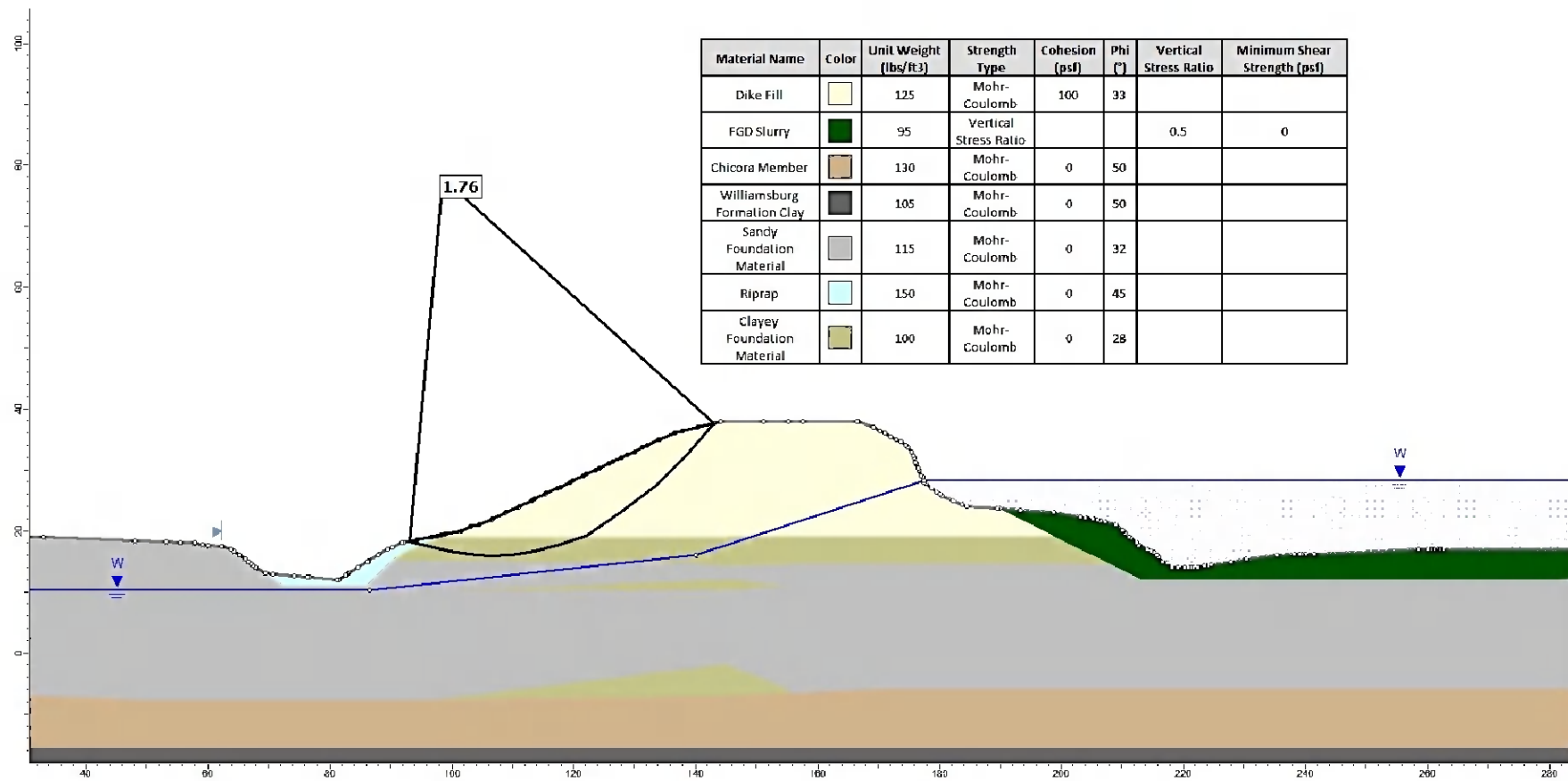


Figure 10. Calculated Factor of Safety for Cross Section C: Static Factor of Safety - Maximum Surcharge Pool

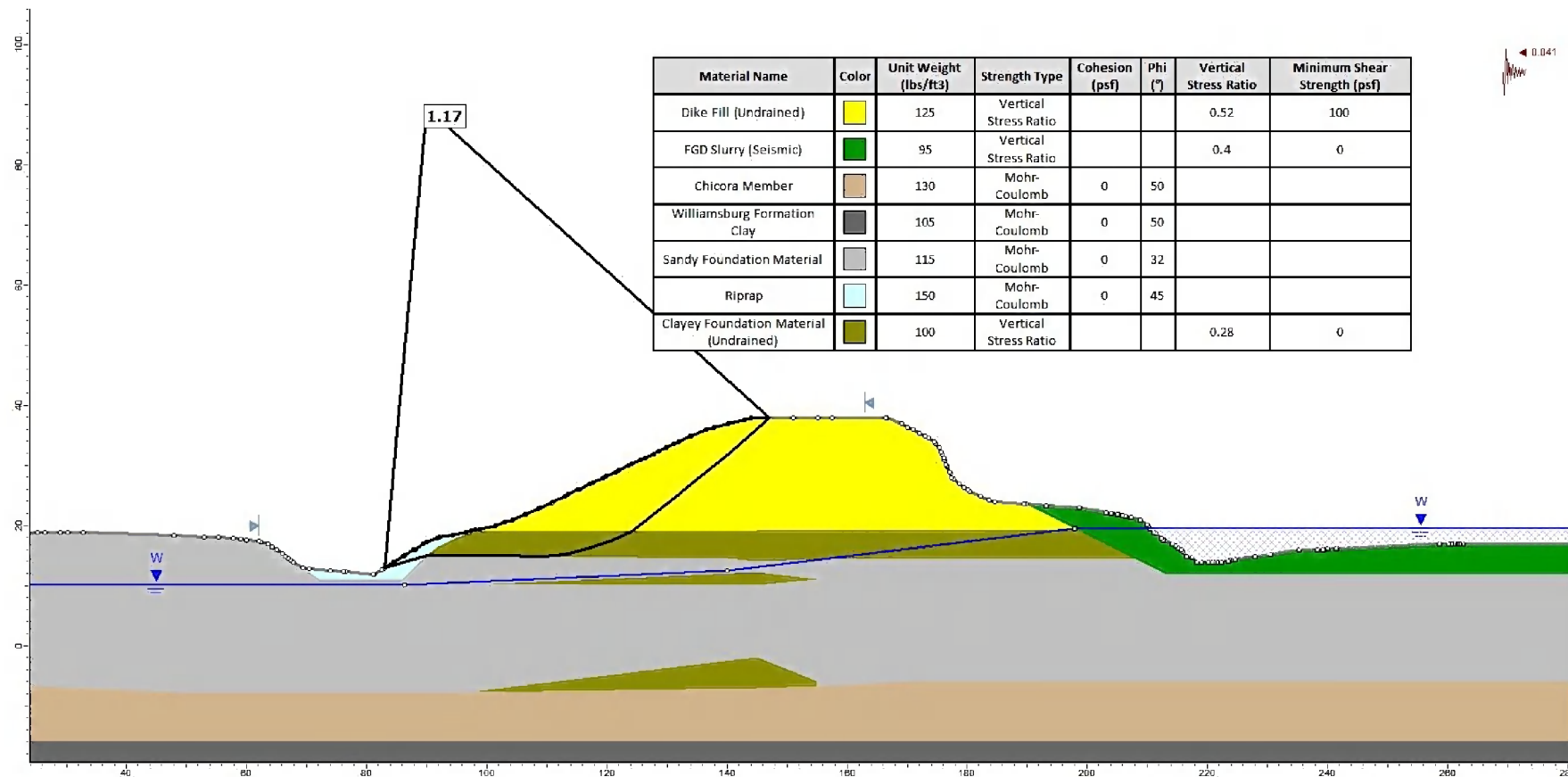


Figure 11. Calculated Factor of Safety for Cross Section C: Seismic Factor of Safety – Maximum Normal Storage Pool

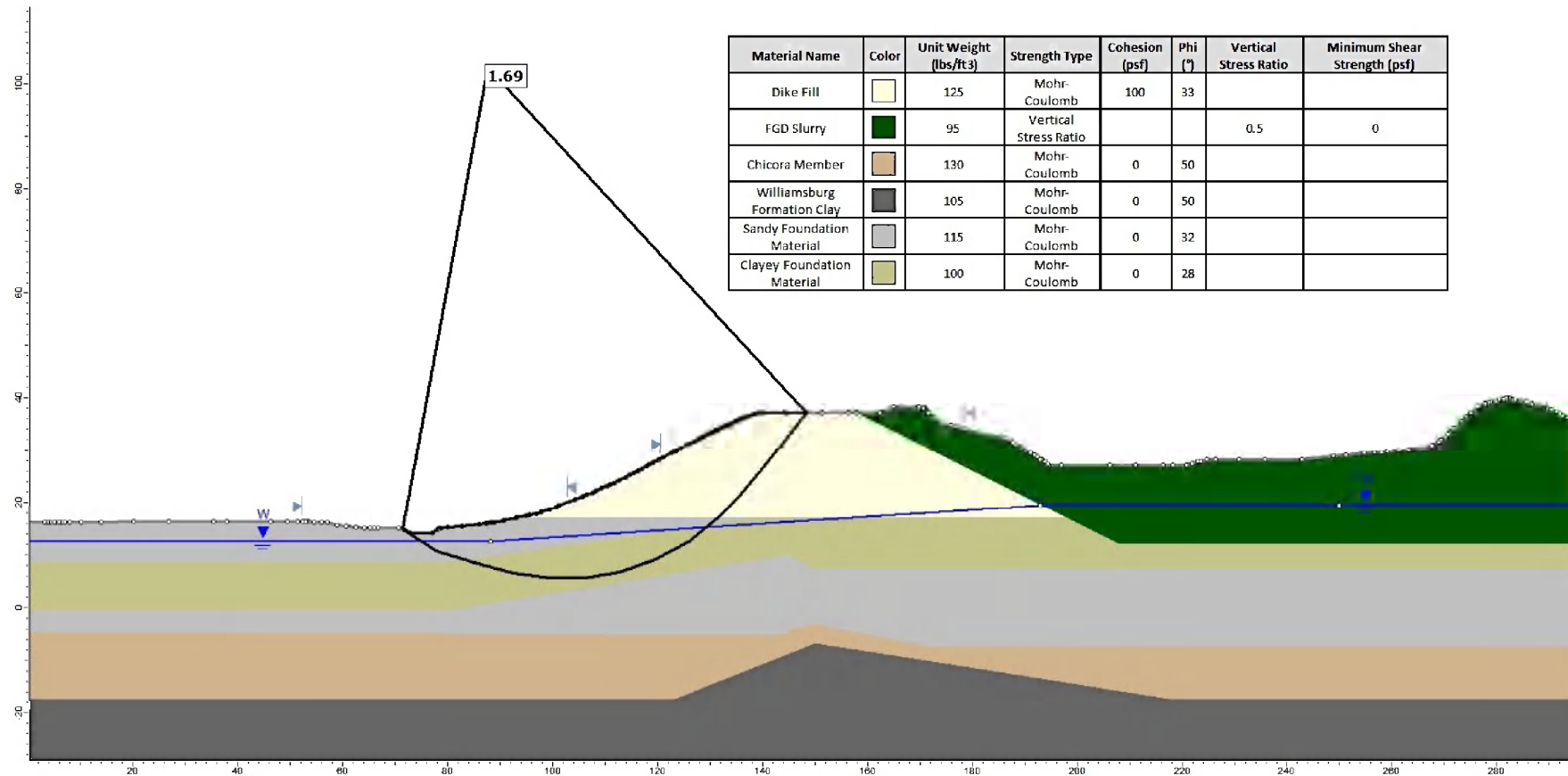


Figure 12. Calculated Factor of Safety for Cross Section D: Static Factor of Safety – Maximum Normal Storage Pool

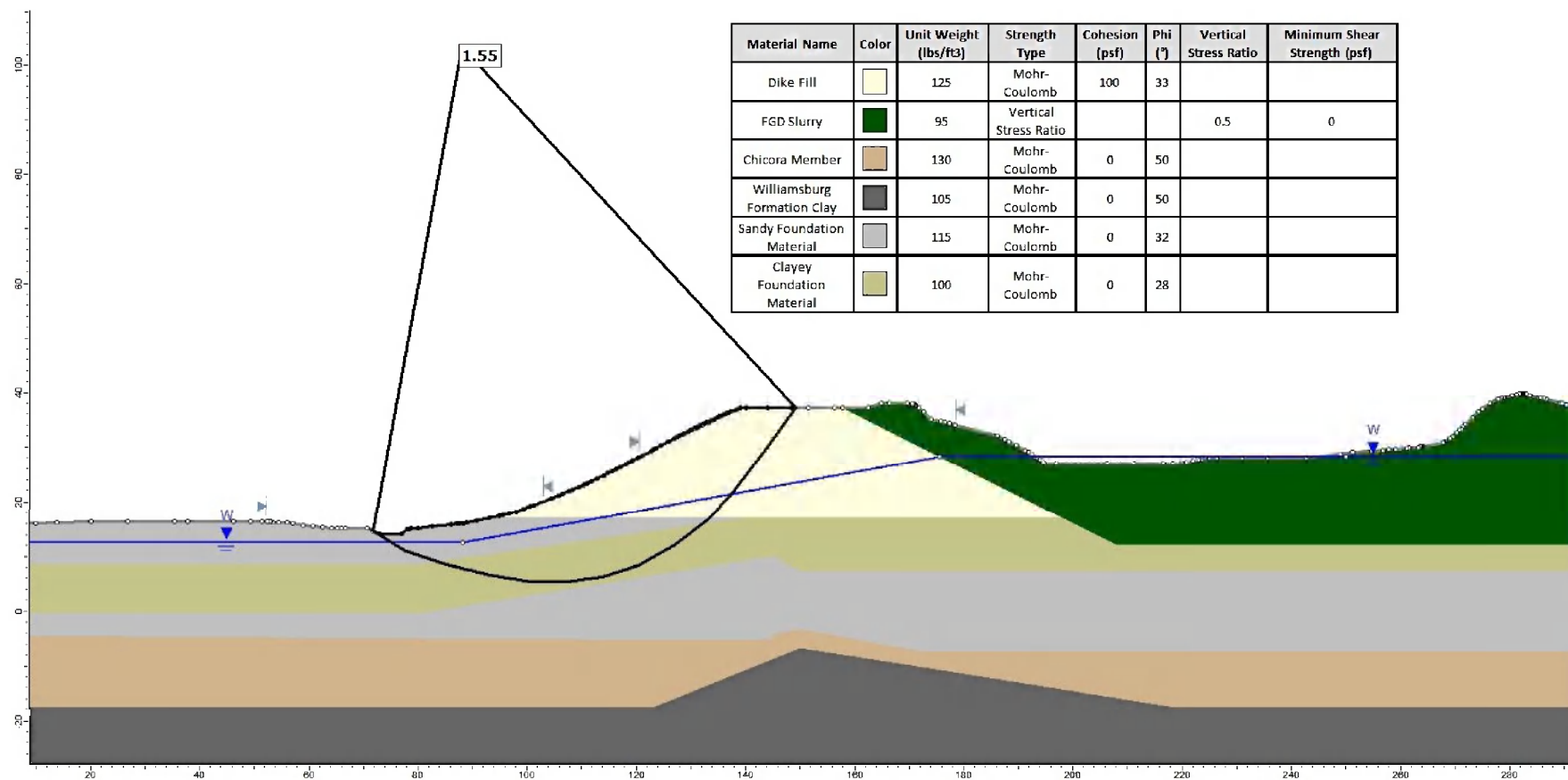


Figure 13. Calculated Factor of Safety for Cross Section D: Static Factor of Safety - Maximum Surcharge Pool

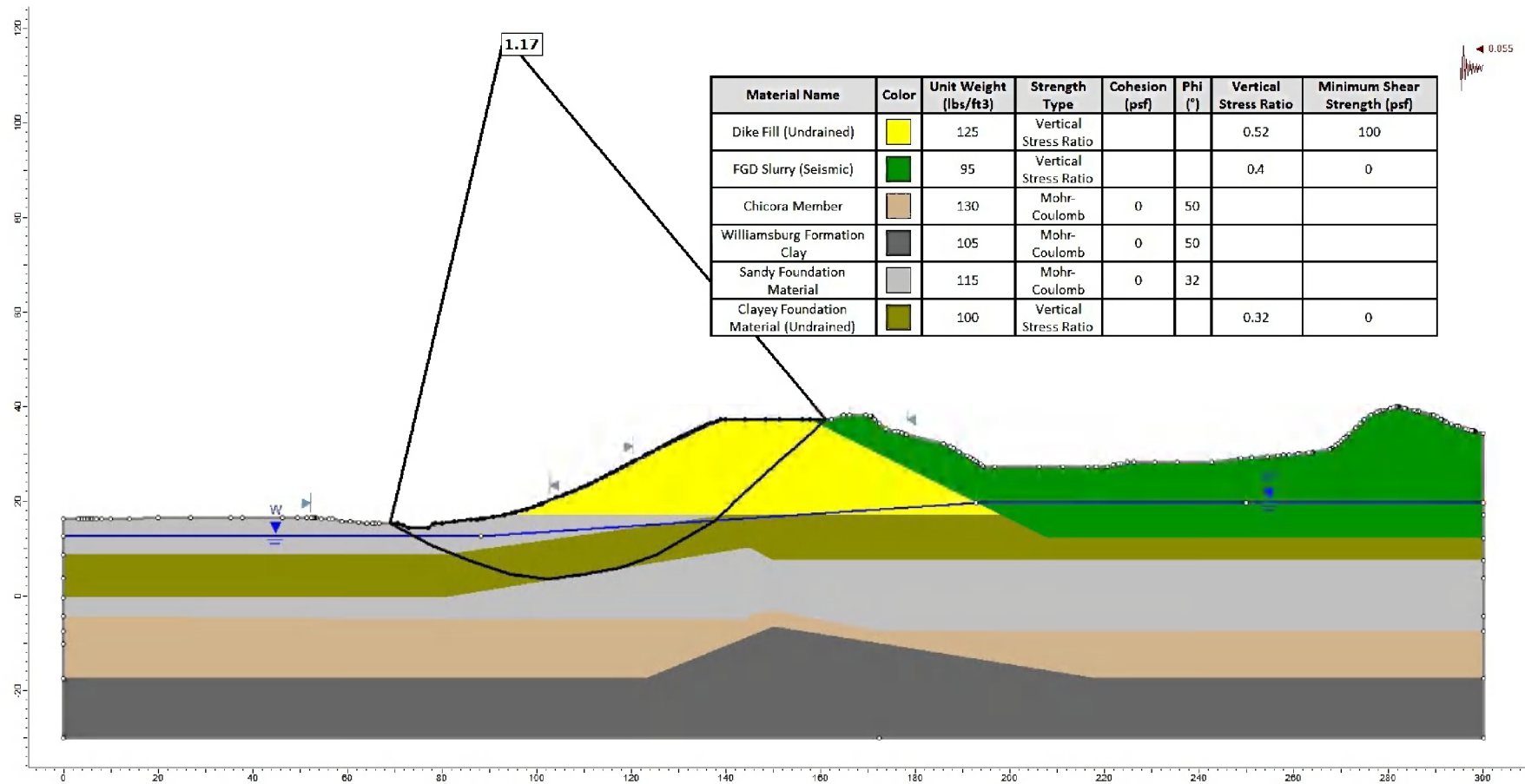


Figure 14. Calculated Factor of Safety for Cross Section D: Seismic Factor of Safety – Maximum Normal Storage Pool

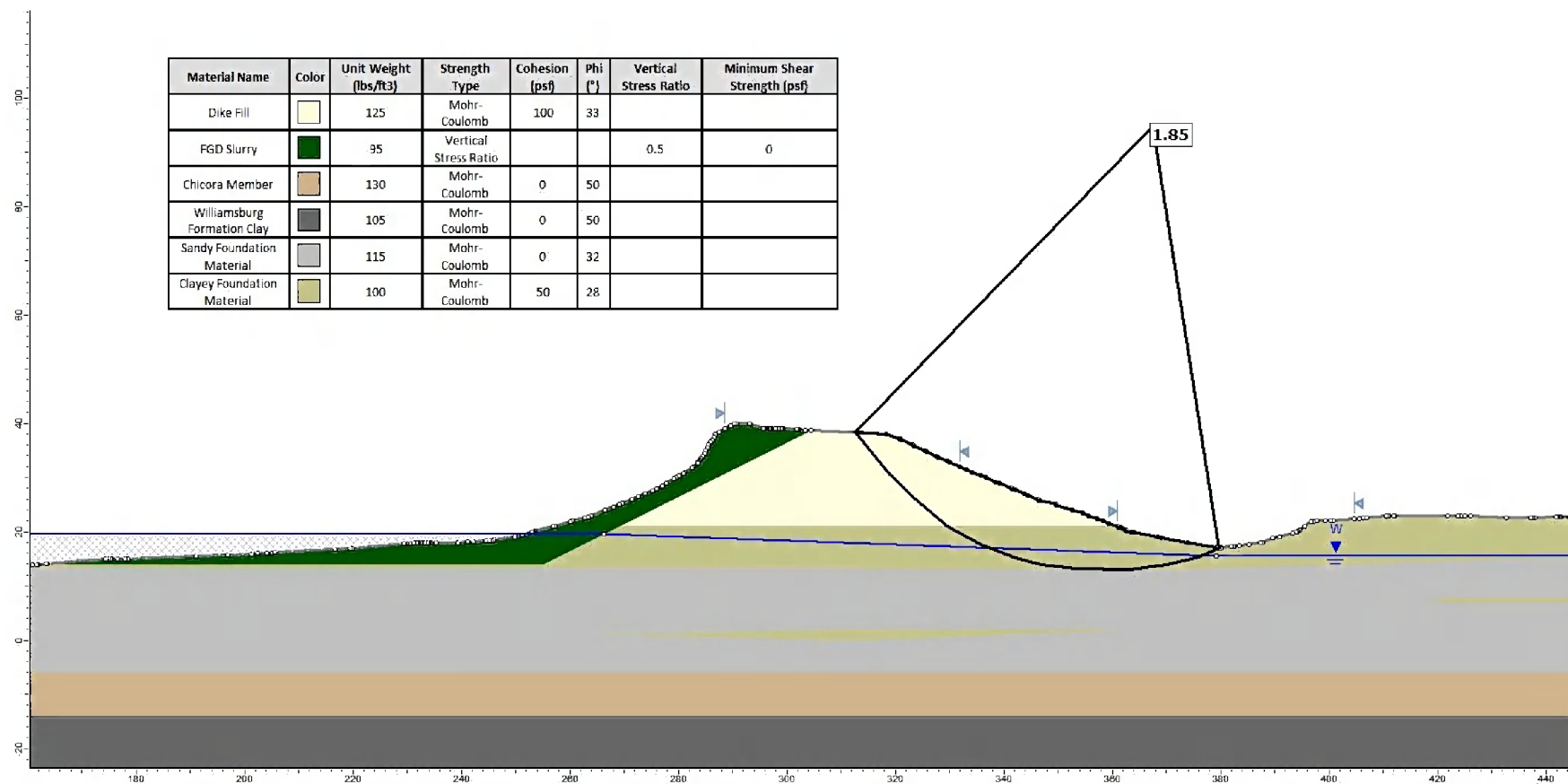


Figure 15. Calculated Factor of Safety for Cross Section E: Static Factor of Safety – Maximum Normal Storage Pool

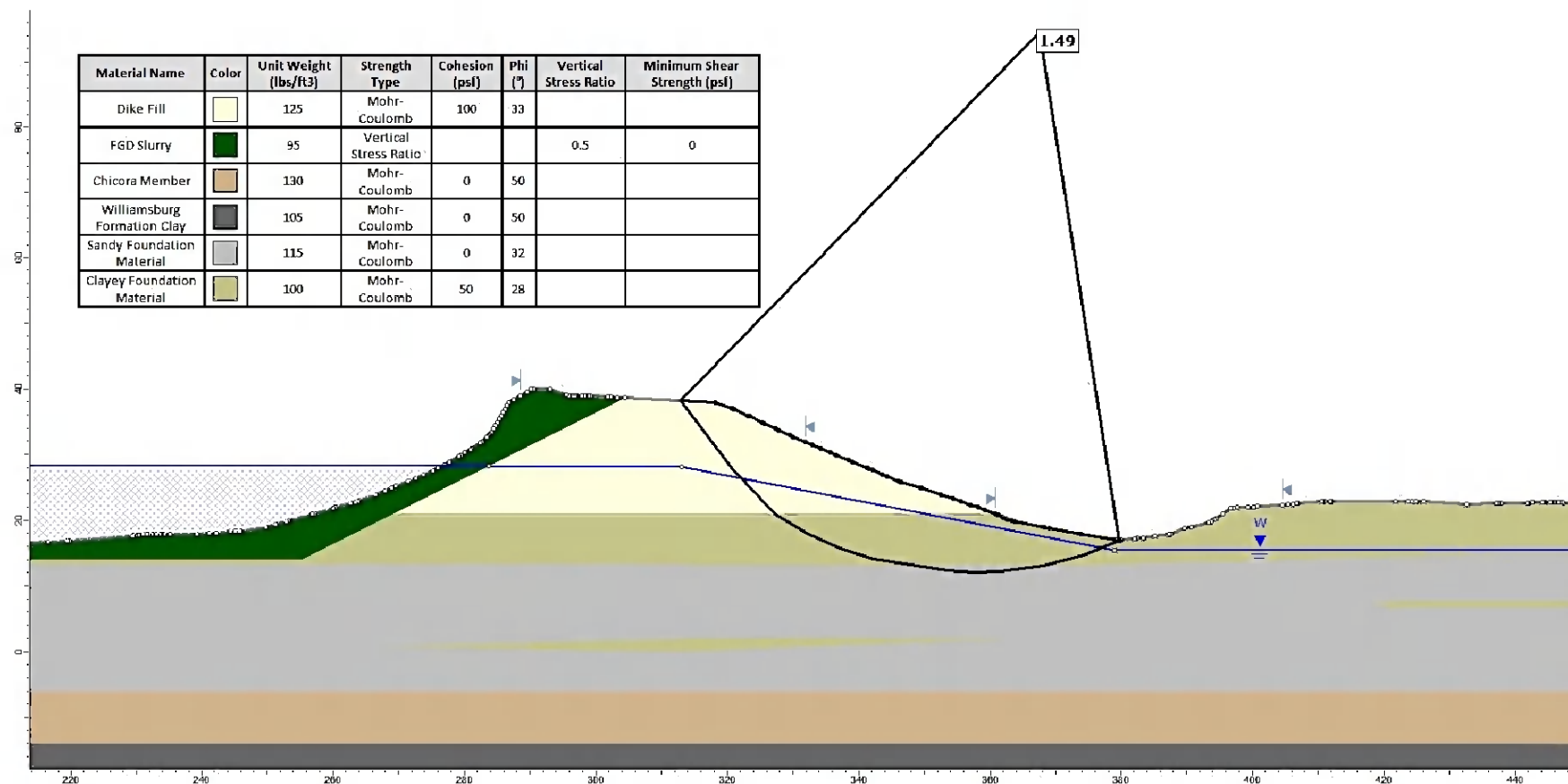


Figure 16. Calculated Factor of Safety for Cross Section E: Static Factor of Safety – Maximum Surge Pool

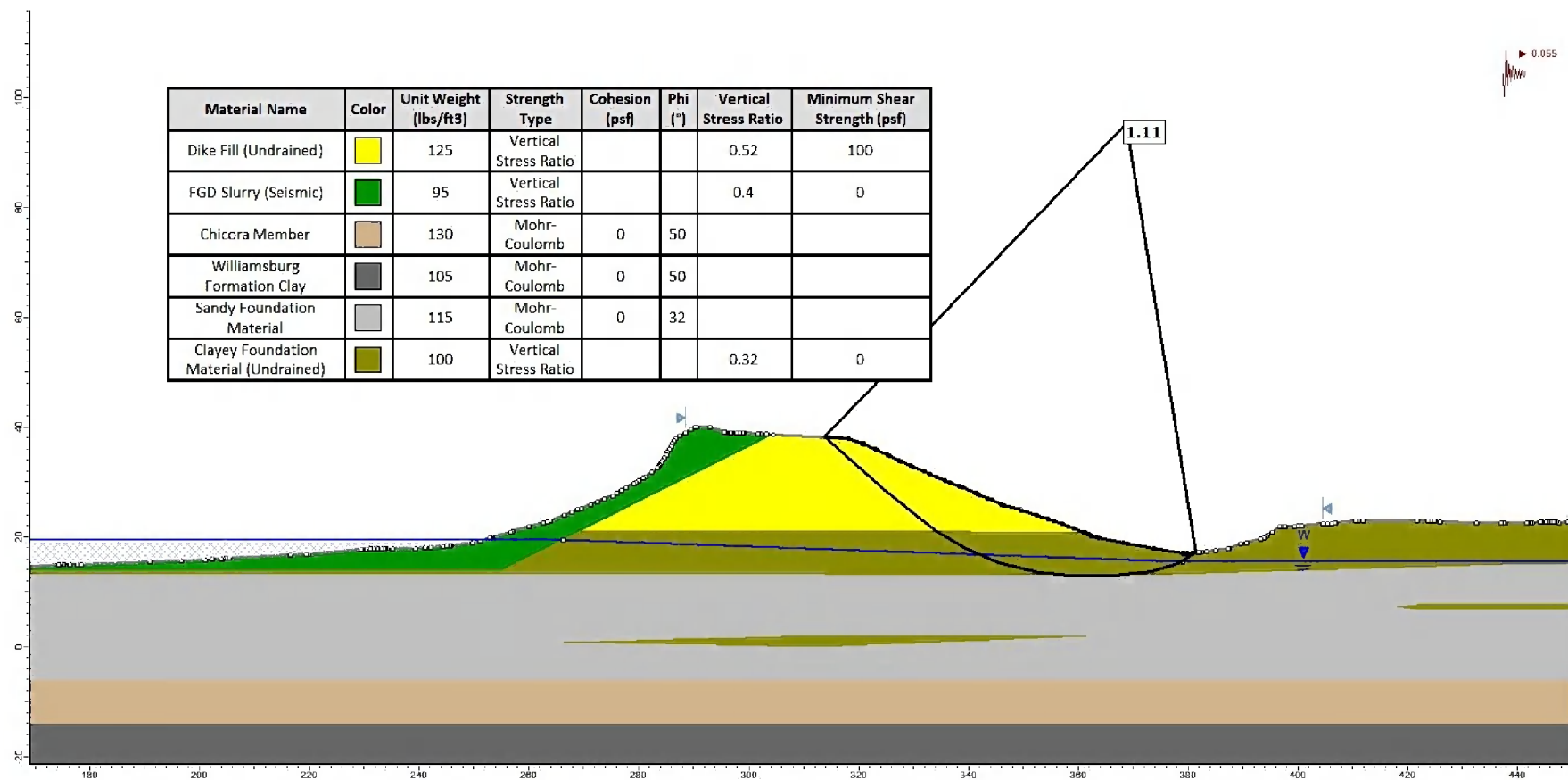


Figure 17. Calculated Factor of Safety for Cross Section E: Seismic Factor of Safety – Maximum Normal Storage Pool