Prepared for

Santee Cooper
One Riverwood Drive
Moncks Corner, SC 29461

2021 PERIODIC SAFETY FACTOR ASSESSMENT, Revision 1
SLURRY POND

WINYAH GENERATING STATION
Georgetown, South Carolina

Prepared by

Geosyntec Consultants, Inc.
1300 South Mint Street, Suite 300
Charlotte, North Carolina 28203

Project No. GC8100

November 2021

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF ATTACHMENTS

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.

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 36052 WOO-KUEN SHIN

Woo-Kuen Shin, Ph.D, P.E.
South Carolina License No. 36052

10 November 2021

Date

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).

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 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.

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 five years.

This *2021 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 spans approximately 106 acres. 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 resulted 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, 2021a).

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 the 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 perimeter dikes are approximately 25 to 30 ft in height in the northern and western sections, approximately 20 to 25 ft in height in the eastern section, and approximately 15 ft in height in the southern section (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 dike crest is approximately 12- to 15-ft wide and typically at elevations 37.0 to 39.0 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 the Slurry Pond at WGS based on the results of the initial periodic safety factor assessments (2016 Safety Factor Assessment) (Geosyntec, 2016), recent survey dated September 2021 (McKim & Creed, 2021), 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 2016 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 and the site response analysis of the Slurry Pond perimeter dikes are presented in Section 4;
- Liquefaction potential evaluation is presented in Section 5;
- Slope stability analyses performed for the safety factor assessment are discussed in Section 6; and
- The summary and general conclusions are presented in Section 7.

2. CHANGES IN SITE CONDITIONS

Santee Cooper personnel indicated that no changes were made for the Slurry Pond perimeter dikes and adjacent areas outside the dikes since preparation of the 2016 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 dated September 2021 (McKim & Creed, 2021) and the topographic survey used in the 2016 Safety Factor Assessment indicated that CCR have been moved within the surface impoundment but the volume of CCR impounded within the surface impoundment has changed insignificantly since the last assessment.

Santee Cooper provided available water level measurements from wells in the Slurry Pond area, located outside the downstream toe of the pond perimeter dike. The recorded water levels in these wells have generally been steady over the last five years. Based on the review of the topographic survey and limited water level measurements adjacent to the Slurry Pond perimeter dikes, the water level within the perimeter dike and beyond the downstream toe of dike is expected to be similar to the water level used in the 2016 Safety Factor Assessment. 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."

Because the Slurry Pond was classified as a "High Hazard Potential" surface impoundment, the PMF event with a rainfall duration of 72 hours was selected as the inflow design flood (IDF).

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

HydroCAD® Version 10.0 software (HydroCAD, 2019) was utilized to compute the stormwater volume using the Soil Conservation Service (SCS) Technical Release 20 (TR-20) method (SCS, 1982) and to model the performance of the hydraulic structures of the Slurry Pond during the IDF. The PMF event with a 72-hour duration precipitation event resulted in a rainfall depth of approximately 51 inch (NOAA, 1980) and was modeled within HydroCAD® using a SCS Type III rainfall distribution.

Stormwater runoffs from 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. 2 receives water from Detention Pond No. 2 and discharges to the Slurry Pond. Detention Pond No. 2 is equipped with a spillway to Pennyroyal Creek which may only be activated during storm events greater than the 25-year, 24-hour storm. Non-contact stormwater collected on top of the geosynthetic cover in the West Ash Pond drains to the Slurry Pond by gravity through two 36-inch diameter culverts. There is also an emergency spillway that hydraulically connects between the Slurry Pond and the West Ash Pond. A Floating Pump Station equipped with two Tsurumi GSZ-4-45-4 submersible pumps, installed in the Slurry Pond in 2015, 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 maximum head, normal pool operating elevation of 19.6 NGVD 29. Piping is valved such that the Floating Pump Station may convey water to the Pump Station No. 1 sump located immediately east of the Slurry Pond. Pump Station No. 1 conveys water to the West Low Volume Waste Pond to be further treated prior to discharging to the Cooling Pond.

Details of the H&H analysis are provided in a document titled *"Inflow Design Flood Control System Plan: Slurry Pond"* (Geosyntec, 2021b). Note that the vertical datum conversion between NGVD 29 and North American Vertical Datum of 1988 (NAVD 88) is -1.0 ft (i.e., NGVD 29 – 1.0 ft = NAVD 88) (FEMA, 2015).

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 with a 72-hour duration) was computed as 34.1 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

This section presents the results of seismic hazard evaluation and site response analysis of the Slurry Pond perimeter dikes. Seismic hazard evaluation includes the selection of an appropriate hazard level and associated hazard parameters (e.g., peak ground acceleration, or PGA). Site response analysis was performed to evaluate the local site effects on selected time history records propagated from the hypothetical, firm ground outcrop to the ground surface at the Site. Details and results for these analyses are presented in Attachment 3 to this Safety Factor Assessment Report and summarized herein.

4.1 Seismic Hazard Evaluation

A seismic hazard evaluation typically consists of the selection of appropriate hazard level and associated seismic parameters, which include the target acceleration response spectra, PGA, and the controlling earthquake magnitude. The seismic hazard analysis also involves the selection of ground motions that envelop the target response spectrum.

4.1.1 Seismic Hazard Level

The appropriate hazard level is often expressed in probabilistic terms as a specific hazard level that has a certain probability of exceedance in a given time period. The CCR Rule states in §257.63(a) that:

"New CCR landfills, existing and new CCR surface impoundments, and all lateral expansions of CCR units must not be located in seismic impact zones, unless the owner or operator demonstrates by the dates specified in paragraph (c) of this section that all structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site."

§257.53 defines the maximum horizontal acceleration in lithified earth material as:

"... the maximum expected horizontal acceleration at the ground surface as depicted on a seismic hazard map, with a 98 percent or greater probability that the acceleration will not be exceeded in 50 years, or the maximum expected horizontal acceleration based on a site-specific seismic risk assessment."

A 98 percent or greater probability of not being exceeded in 50 years (or two percent probability of exceedance in 50 years) corresponds to a return period of approximately 2,500 years. The Preamble of the CCR Rule indicates that USEPA selected this return period by considering a typical operating life for CCR surface impoundments (i.e., 50 years) and its common use in seismic design criteria throughout engineering (e.g., American Society of Civil Engineers [ASCE] 7-16 [2016]). For the CCR surface impoundments at WGS, pond closure was initiated in 2021 and is expected to be complete in less than 15 years. Therefore, an earthquake return period of approximately 750 years was conservatively selected for the 2021 Safety Factor Assessment of the Slurry Pond (i.e., two percent probability of exceedance in 15 years) following the basis for selecting the return period of approximately 2,500 years for typical CCR surface impoundments.

4.1.2 Peak Ground Acceleration (PGA)

PGA values corresponding to different hazard levels and different site conditions, including firm ground outcrops, are published as seismic hazard maps or curves. The 2016 Safety Factor Assessment Report (Geosyntec, 2016) referenced seismic hazard maps presented in the South Carolina Department of Transportation (SCDOT) Geotechnical Design Manual (GDM) (SCDOT, 2010) for selection of a PGA to incorporate local site effects for the Charleston Seismic Zone researched by Chapman and Talwani (2006). The GDM was updated in 2019 (SCDOT, 2019) and does not present the seismic hazard maps referenced in the 2016 Safety Factor Assessment Report. Moreover, SCDOT is updating seismic hazard maps at the time of this seismic hazard evaluation.

As an alternative, United States Geological Survey (USGS) hazard curves for two percent probability of exceedance in 15 years (i.e., approximately 750-year return period event) at the BC boundary (i.e., boundary between National Earthquake Hazard Reduction Program [NEHRP] site classes B and C with a mean shear wave velocity of 2,500 ft/s) were used to estimate the PGA and spectral accelerations for a hypothetical firm ground outcrop, similar to "geologically realistic" site conditions, at the Site. The data available at the USGS website (Petersen et al., 2019) use pre-calculated hazard values at nearby grid locations and interpolate the hazard value for a given site location. As discussed in Attachment 3, the interpolated PGA from USGS Hazard Curves is 0.15g for the Site.

4.1.3 Earthquake Magnitude

In a probabilistic seismic hazard analysis, the PGA cannot be associated with a single earthquake event due to the hazard contribution from multiple possible events. An earthquake moment magnitude (M_W) value is required to conduct liquefaction potential analyses and to select earthquake time histories. A process called deaggregation can be performed for sites that have multiple hazard sources using the most up-to-date USGS (2014) deaggregation tool. As discussed in Attachment 3, a 7.3 moment magnitude was selected for liquefaction potential analyses and time history selection for the Site by applying this deaggregation tool.

4.1.4 Target Acceleration Response Spectra and Time History Selection

A target acceleration response spectrum was established using the USGS seismic hazard curves at different spectral periods (or frequencies). Time histories of ground motions are selected such that their response spectra match or envelope the target acceleration response spectrum. Six acceleration time histories used for the 2016 Safety Factor Assessment were still considered adequate as input for site response analyses since the scaled time histories provide a conservative, reasonable match with the target acceleration response spectrum. The response spectra of scaled time histories selected for the site response analyses are presented on Figure 4 of Attachment 3.

4.2 Site Response Analysis

Site response analyses computed the cyclic shear stresses within the select representative soil profile located along the perimeter dike centerline. Computed cyclic shear stresses were applied for the liquefaction potential analysis, and were also utilized to evaluate the seismic safety factor as part of this Safety Factor Assessment.

4.2.1 Analysis Model Setup

Site response analyses presented herein were conducted using DEEPSOIL® (Hashash et al., 2020), a one-dimensional, nonlinear site response analysis program. The program assumes that all the soil layers are perfectly horizontal (i.e., "layer cake") and that ground response is mainly caused by vertically-propagating, horizontally polarized shear waves. This assumption is valid for many geotechnical cases including the response analyses of the Site. Under these assumptions, the subsurface stratigraphy is modeled as a one-dimensional column of soil layers for the analyses. Two representative profiles were selected for the site response analyses of the Slurry Pond perimeter dikes based on the 2016 Safety Factor Assessment results and are shown on Figure 6 of Attachment 3.

DEEPSOIL® employs a viscoelastic material model, described by its shear modulus (G), mass density (ρ) or unit weight (γ), and damping (D). Preliminary equivalent-linear site response analyses yielded relatively large maximum cyclic shear strains in some layers, which are greater than the cyclic shear strains for which equivalent-linear analyses are considered applicable (i.e., one to two percent). Therefore, nonlinear site response analyses were performed. Additional discussion of input parameters, such as the V_s profile, soil plasticity, and shear modulus reduction/damping curves applied in the DEEPSOIL® program, are provided in Attachment 3.

As discussed in Section 2, the water level within the perimeter dike is expected to be similar to the water level used in the 2016 Safety Factor Assessment. Therefore, the site response analyses for the 2021 Safety Factor Assessment were performed with the water table used in the 2016 Safety Factor Assessment Report, as discussed in Attachment 3.

4.2.2 Site Response Analysis Results

Maximum shear stresses within the representative soil profiles were computed and presented on Figures 9a, 9b, and 10 of Attachment 3. Additional site response analysis results are presented in Attachment 3. The maximum cyclic shear stresses at depths were calculated and these values were used to calculate a measure of shear stress developed during the design earthquake (cyclic stress ratios, or CSR) in the evaluation of liquefaction potential, presented in Section 5 of this Safety Factor Assessment Report. The site response analysis results were also used to calculate the horizontal seismic coefficient (k_h) as presented in Section 6 of this Safety Factor Assessment Report.

5. EVALUATION OF LIQUEFACTION POTENTIAL

This section presents the liquefaction potential evaluation for the critical section of the Slurry Pond perimeter dikes. The evaluation applies the cyclic shear stress computed as part of the site response analysis (Section 4). Further details of the liquefaction potential evaluation are presented in Attachment 4 of this Safety Factor Assessment Report.

5.1 Regulatory Framework

A periodic safety factor assessment is required by the CCR Rule to evaluate whether the existing CCR surface impoundments meet minimum safety factors (also referred to as "factors of safety") for slope stability provided in §257.73(e)(1). Specifically, §257.73(e)(1)(iv) requires that:

"embankments constructed of soils that have susceptibility to liquefaction, the calculated liquefaction factor of safety must equal or exceed 1.20."

The purpose of this section is to discuss the methodology, analysis, and results of the liquefaction potential analysis to evaluate if the Slurry Pond dike fill and foundation soils are susceptible to liquefaction triggering under the design earthquake. If the dike fill and foundation soils are not found to be susceptible to liquefaction, then the liquefaction factor of safety is not required and is not evaluated as part of this Safety Factor Assessment.

5.2 Methodology

Liquefaction potential analysis was performed based on the Simplified Procedure recommended by Seed and Idriss (1971) and an update by Boulanger and Idriss (2014). This approach is based on comparing in-situ test results with case histories of occurrences and non-occurrences of liquefaction due to past earthquakes. The analyses presented herein were conducted for soil borings and CPT soundings along the critical sections of the Slurry Pond. The factor of safety against liquefaction (FS_{liq}) was computed as the ratio of a measure of a soil's resistance to triggering of liquefaction (cyclic resistance ratio, or CRR) to CSR.

5.2.1 Dike Phreatic Surface Conditions

As described in Section 2, the water level within the perimeter dike is anticipated to be similar to the water level used in the 2016 Safety Factor Assessment. The phreatic surface at the time of the boring/CPT sounding was used to estimate CRR profiles. CSR profiles were estimated for the time at which the design earthquake event occurs using the phreatic surface used for the 2016 Safety Factor Assessment.

5.2.2 Age Correction Factor

Correlations associated with liquefaction potential analysis were developed based on case histories of relatively young soil deposits (i.e., Holocene age). As described in SCDOT (2019), liquefaction resistance, as represented by the CRR, may be adjusted to account for aging effects in older soils based on time from deposition (i.e., geologic age) and time from last occurrence of liquefaction (i.e., geotechnical age). As described in Attachment 4, an age correction factor (K_{dr}) of 1.2 was applied to the Pleistocene-aged soils at the WGS site (typically foundation soils below the base of the dike), and an age correction factor of 1.0 was applied to the dike fill soils.

5.3 Evaluation Results

The FS_{liq} was computed at every depth interval where data were collected for soil test borings (2-ft or 5-ft intervals) and CPT sounding (0.16-ft intervals) advanced in the vicinity of the Slurry Pond perimeter dikes. Analysis results for each soil boring and CPT sounding analyzed are provided on Figures 6 through 16 of Attachment 4 to this Safety Factor Assessment Report. FS_{liq} values computed for dike fill and foundation soils were found to exceed 1.0 for the conditions described within this Safety Factor Assessment Report (i.e., no zones expected to undergo triggering of liquefaction under the design earthquake were identified for borings and CPT soundings advanced through the critical sections of the Slurry Pond perimeter dikes).

6. 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 5 of this Safety Factor Assessment Report and summarized herein.

6.1 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 6 describes the geometric model, methodology, and analysis results for each case.

6.2 Analysis Models

The models used for the 2016 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 2016 Safety Factor Assessment. Three representative cross sections were selected for the 2021 Safety Factor Assessment based on factors of safety calculated in the 2016 Safety Factor Assessment.

6.3 Methodology

6.3.1 Static Slope Stability

Global slope stability analyses were performed using Spencer's method (Spencer, 1973), as implemented in the computer program SLIDE®, version 6.039 (Rocscience, 2016). 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.

6.3.2 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®.

6.4 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 through 4 of Attachment 5.

6.5 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 factors were evaluated with a more conservative water level (35.1 ft NGVD29) within the Slurry Pond than the maximum surface water level (34.1 ft NGVD 29) from the H&H analyses (Section 3). The static safety factor was evaluated for the critical cross section of the Slurry Pond.

6.6 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.

6.7 Summary of Results

As presented in Table 3 of Attachment 5, the minimum 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 1.66, 1.41, and 1.06, respectively. 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 5.

7. SUMMARY AND GENERAL CONDITIONS

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

- The maximum surcharge 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 0.15g 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) as shown below.

Factor of Safety Case	**Target FS**	**Cross Section C***	**Cross Section D***	**Cross Section E***
Static - Maximum Normal Storage Pool	1.50	1.72	1.62	1.68
Static - Maximum Surcharge Pool	1.40	1.69	1.41	1.41
Seismic - Maximum Normal Storage Pool	1.00	1.09	1.14	1.06
Liquefaction Slope Stability	1.20	Not Applicable	Not Applicable	Not Applicable

*Note: The cross section locations are shown on Figure 1 of Attachment 5.

8. REFERENCES

Bishop, A. (1955), "The Use of the Slip Circle in the Stability Analysis of Slopes," Géotechnique, Vol. 5, No. 1, Jan 1955, pp. 7-17.

Boulanger, R. W. and Idriss, I. M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures," Report No. UCD/CGM-14/01, Center for Geotechnical Monitoring, University of California, Davis, CA

Chapman, M.C. and Talwani, P. (2006), "Seismic Hazard Mapping for Bridge and Highway Design in South Carolina," South Carolina Department of Transportation, FHWA-SC-06-09.

Federal Emergency Management Agency (FEMA) (2015), "Flood Insurance Study: Georgetown County, South Carolina and Incorporated Areas," Preliminary, 13 November 2015.

Geosyntec Consultants, Inc. (2016), "2016 Surface Impoundment Periodic Safety Factor Assessment Report: Slurry Pond," Project No. GSC5242.

Geosyntec Consultants, Inc. (2021a), "Periodic Hazard Potential Classification Assessment: Slurry Pond," Project No. GC8100.

Geosyntec Consultants, Inc. (2021b), "Inflow Design Flood Control System Plan: Slurry Pond," Project No. GC8100.

Hashash, Y.M.A., Musgrove, M.I., Harmon, J.A., Ilhan, O., Xing, G., Numanoglu, O., Groholski, D.R., Phillips, C.A., and Park, D. (2020), "DEEPSOIL 7.0, User Manual", Board of Trustees of University of Illinois at Urbana-Champaign, Urbana, Illinois.

HydroCAD (2019), "HydroCAD Stormwater Modeling," HydroCAD Software Solutions, LLC.

Hynes-Griffin, M. and Franklin, A. (1984), "Rationalizing the Seismic Coefficient Method", Department of the Army, Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi, Miscellaneous Paper GL-84-14, July.

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

Lockwood Greene (1978), "South Carolina Public Service Authority – Georgetown Generating Station."

McKim & Creed (2021), "Topographic Survey for Winyah Generating Station."

National Oceanic and Atmospheric Administration (NOAA) (2021), "Precipitation-Frequency Atlas of the United States," Atlas 14, Volume 2, Version 3.

Petersen, M.D., Shumway, A.M., Powers, P.M., Mueller, C.S., Moschetti, M.P., Frankel, A.D., Rezaeian, S., McNamara, D.E., Luco, N., Boyd, O.S., Rukstales, K.S., Jaiswal, K.S., Thompson, E.M., Hoover, S.M., Clayton, B.S., Field, E.H., and Zeng, Y. (2019), "The

2018 update of the US national seismic hazard model: Overview of model and implications", Earthquake Spectra, Vol. 36, No. 1, 2019, pp. 5-41.

Rocscience (2016), "SLIDE® – 2-D Limit Equilibrium Slope Stability for Soil and Rock Slopes," User's Guide, Rocscience Software, Inc., Toronto, Ontario, Canada.

Santee Cooper (2004), "Pump Station Design Report - Winyah Generating Station Redirect Drainage Project."

Seed, H.B, and Idriss, I.M. (1971), "Simplified Procedure for Evaluation Soil Liquefaction Potential", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 107, NO. SM9.

Soil Conservation Service (SCS) (1982), "Technical Release Number 20 (TR-20)," National Technical Information Service.

South Carolina Dept. of Transportation (SCDOT) (2010), Geotechnical Design Manual, available: http://www.scdot.org/doing/structural_Geotechnical.aspx

South Carolina Dept. of Transportation (SCDOT) (2019), Geotechnical Design Manual Version 2.0, available: https://www.scdot.org/business/pdf/geotech/SCDOT-Geotechnical-Design-Manual.pdf

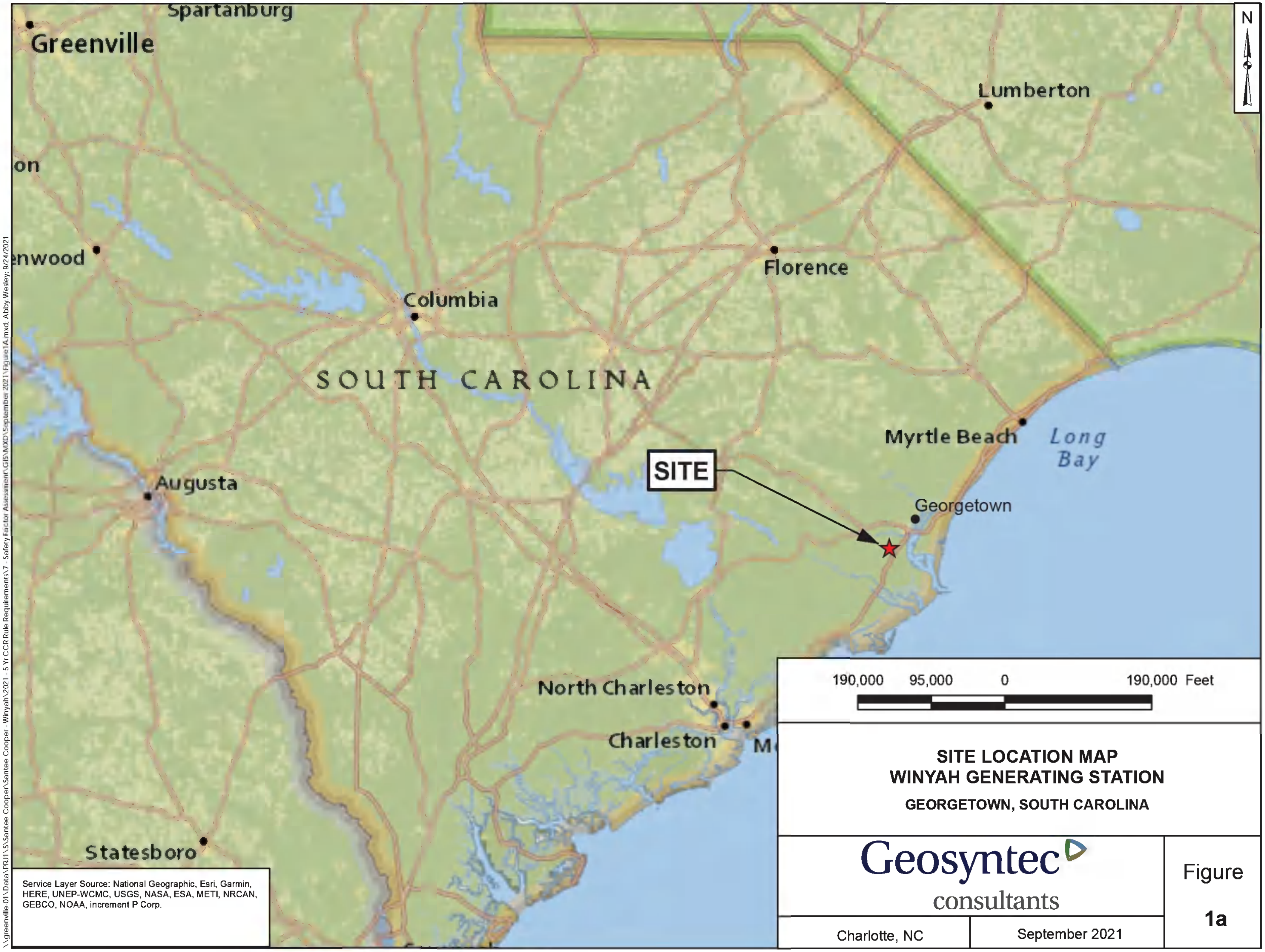
Spencer, E. (1973), "The Thrust Line Criterion in Embankment Stability Analysis," Géotechnique, Vol. 23, No. 1, pp. 85-100, March.

Thomas and Hutton (2012). "Topographic Survey of A Portion of Santee Cooper Winyah Generating Station", prepared for Santee Cooper, 14 January 2014.

Thomas and Hutton. (2015). "Topographic Survey of the Cooling Pond at Santee Cooper Winyah Generating Station."

U.S. Geological Survey (USGS), (2014), "Dynamic: Conterminous U.S. 2014 (v4.2.0) Interactive Deaggregations", 2014. https://earthquake.usgs.gov/hazards/interactive/

FIGURES



Service Layer Source: National Geographic, Esri, Garmin, HERE, UNEP-WCMC, USGS, NASA, ESA, METI, NRCAN, GEBCO, NOAA, increment P Corp.

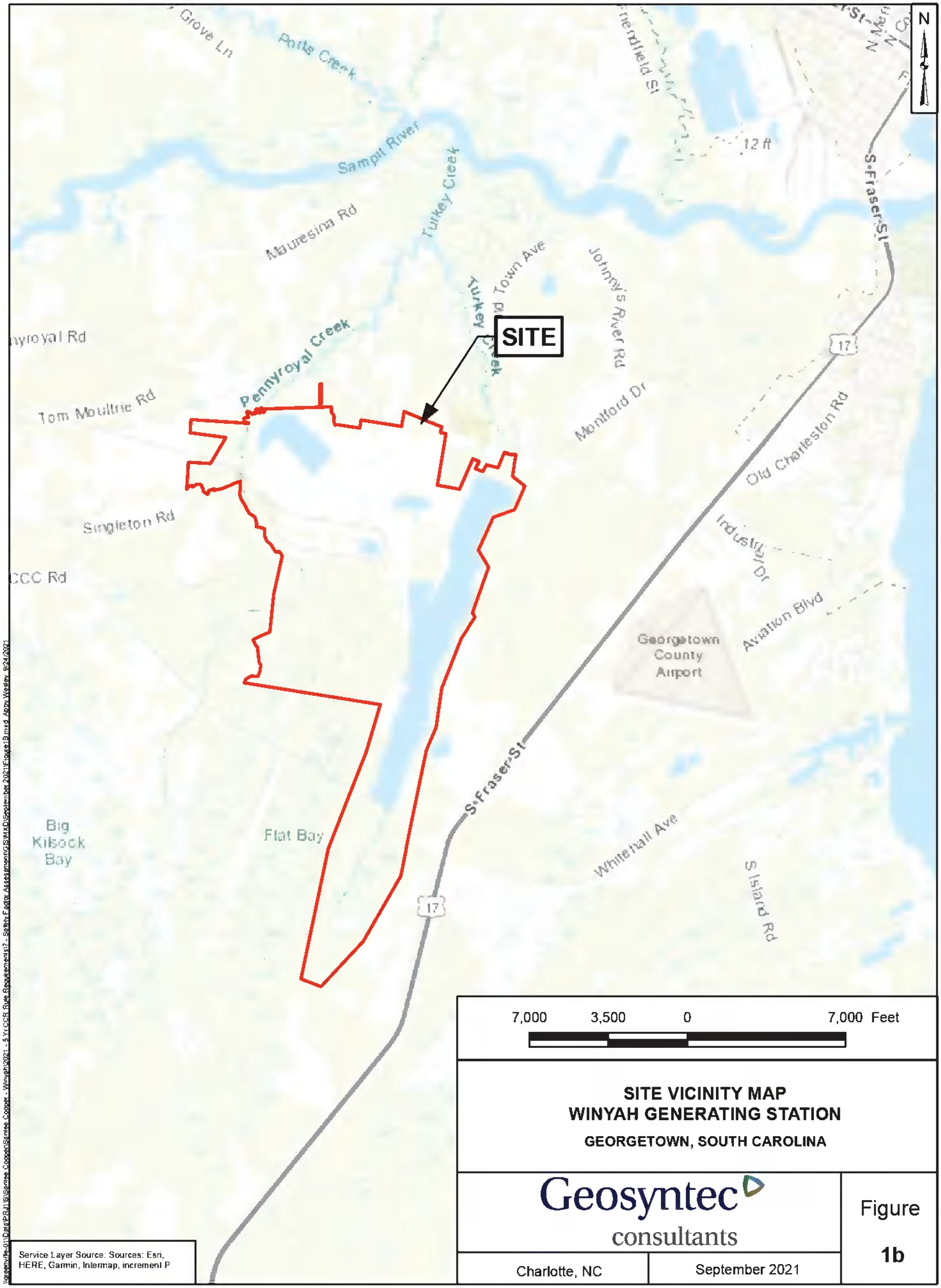
SITE LOCATION MAP
WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

Geosyntec consultants

Charlotte, NC | September 2021

Figure 1a

\\greenville-01\Data\PRJ1\S\Santee Cooper\Santee Cooper - Winyah\2021 - 5 Yr CCR Rule Requirements\7 - Safety Factor Assessment\GIS\MXD\September 2021\Figure1A.mxd; Abby.Wesley; 9/24/2021



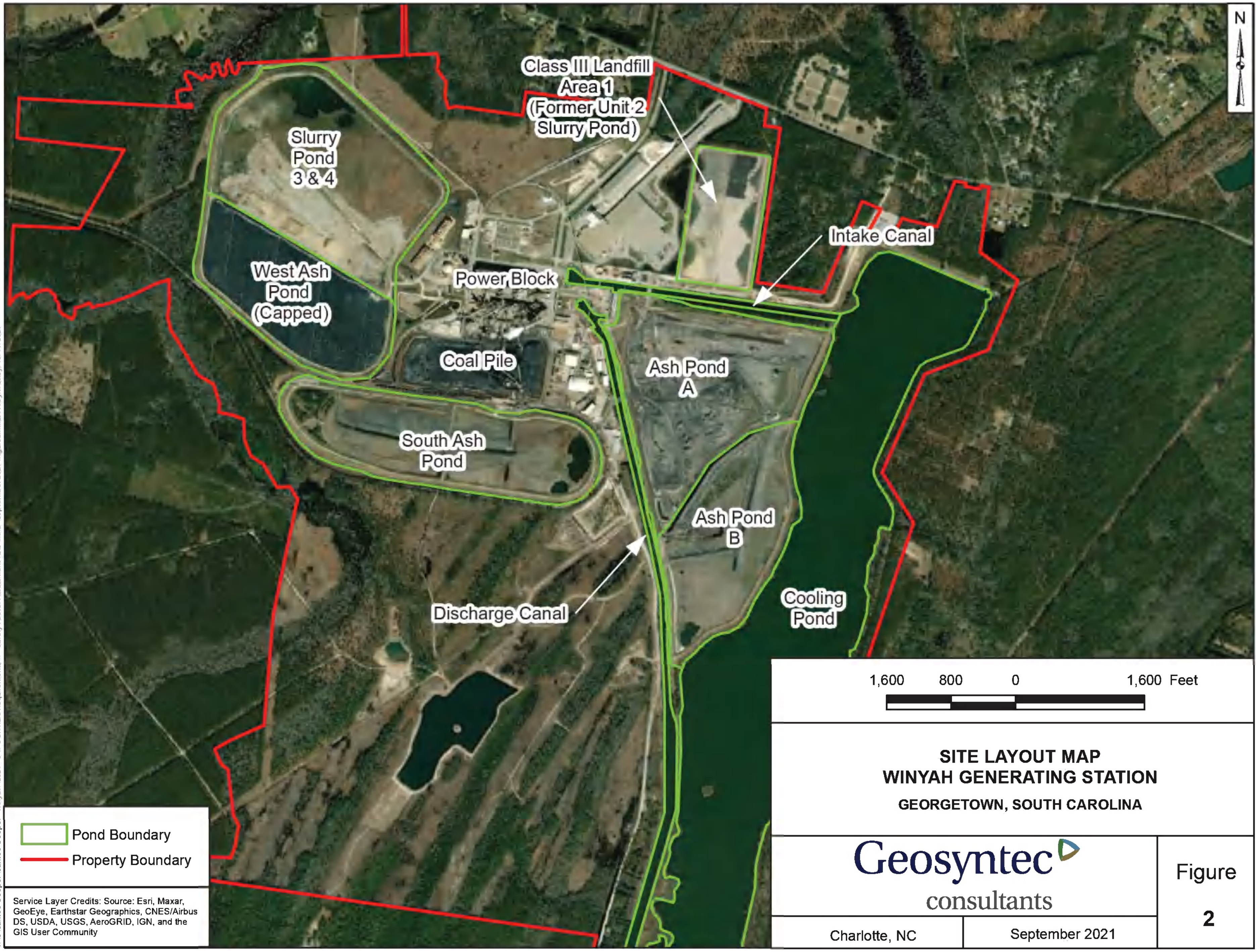
SITE VICINITY MAP
WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

Geosyntec consultants

Charlotte, NC | September 2021

Figure **1b**

Service Layer Source: Sources: Esri, HERE, Garmin, Intermap, increment P



SITE LAYOUT MAP
WINYAH GENERATING STATION

GEORGETOWN, SOUTH CAROLINA

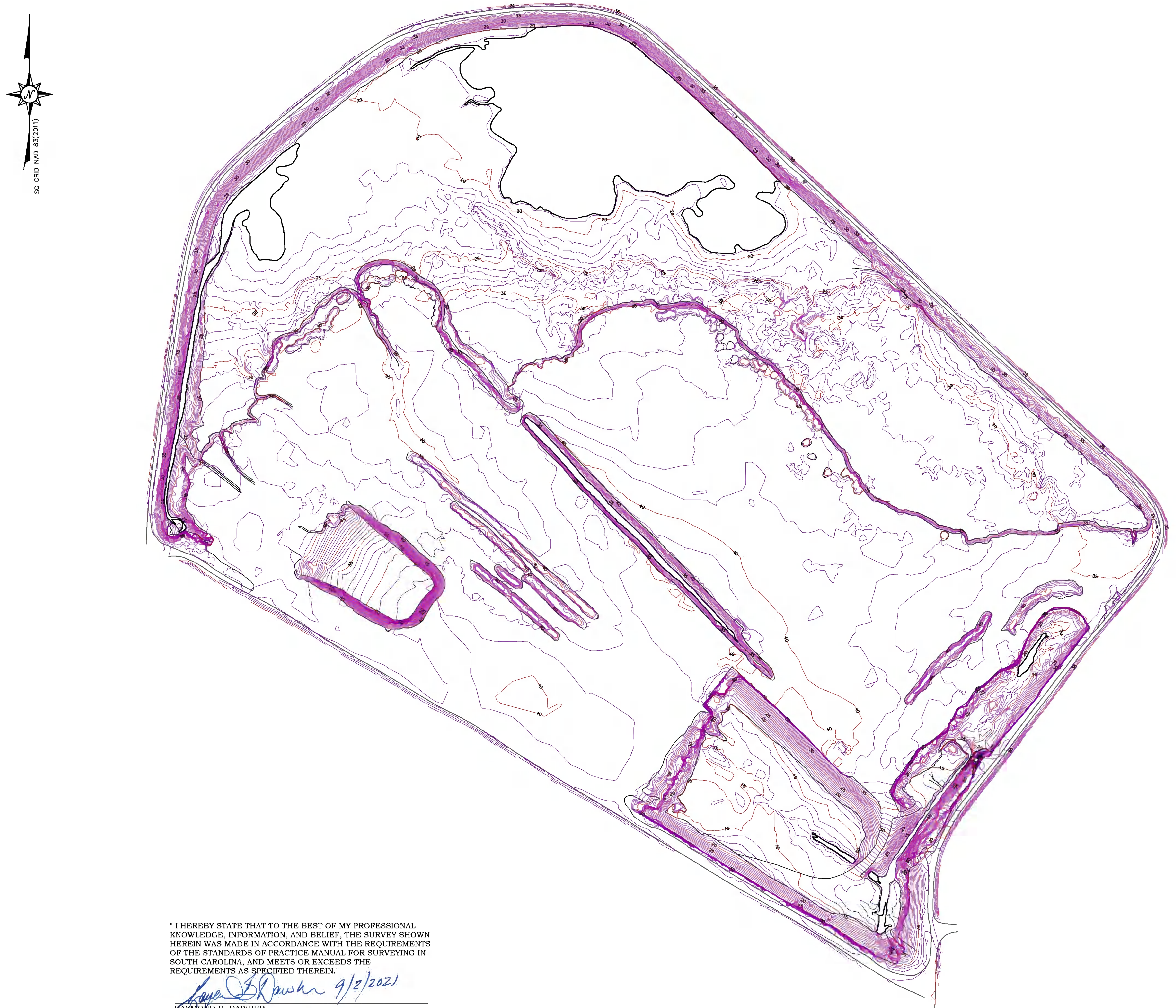
Geosyntec consultants

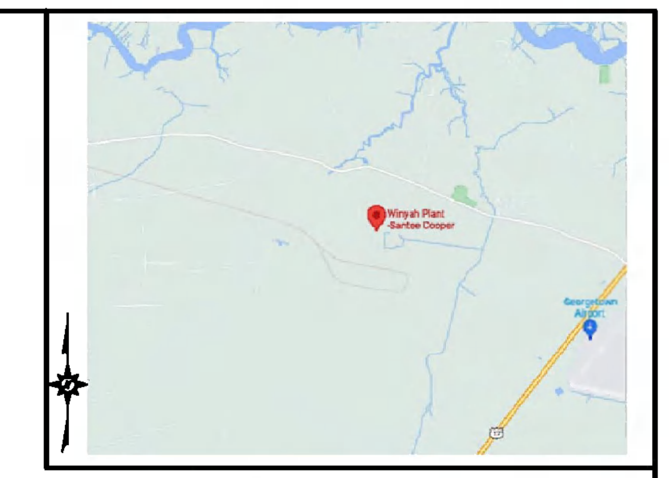
Charlotte, NC | September 2021

Figure **2**

ATTACHMENTS

ATTACHMENT 1
TOPOGRAPHIC SURVEY (SEPTEMBER 2021)





VICINITY MAP – NOT TO SCALE

SURVEYOR'S NOTES

1. ALL DISTANCES ARE HORIZONTAL GROUND IN INTERNATIONAL FEET UNLESS OTHERWISE SHOWN.
2. BEARINGS BASED ON SOUTH CAROLINA NAD83/2011.
3. ELEVATIONS AND CONTOURS SHOWN HERON ARE BASED ON NAVD88. CONTOURS ARE SHOWN AT 1' INTERVALS.
4. SUBSURFACE AND ENVIRONMENTAL CONDITIONS WERE NOT EXAMINED OR CONSIDERED AS A PART OF THIS SURVEY. NO STATEMENT IS MADE CONCERNING THE EXISTENCE OF UNDERGROUND OR OVERHEAD CONTAINERS OR FACILITIES THAT MAY AFFECT THE USE OR DEVELOPMENT OF THIS TRACT.
5. THE EXISTENCE OR NONEXISTENCE OF WETLANDS ON SUBJECT PROPERTY HAS NOT BEEN DETERMINED BY THIS SURVEY.
6. SUBJECT TO ALL EASEMENTS, RIGHT OF WAYS, AND OR ENCUMBRANCES THAT MAY EFFECT THIS PROPERTY.

SIMULTANEOUS AERIAL LIDAR ('50PPSM, AND IMAGERY (~5.7CM GSD) WAS COLLECTED ON 07–24–2021 WITH A FIXED–WING AIRCRAFT EQUIPPED WITH A REIGL 1560II LIDAR SENSOR (SERIAL # S2224887) AND PHASE ONE CAMERA (SERIAL # MM010158). TOPOGRAPHIC MAPPING WAS PERFORMED TO PRODCUE 1"=50' SCALE PLANIMETRICS AND A DIGITAL TERRAIN MODEL (DTM) SUITABLE FOR 1' CONTOURS ALONG WITH 3–INCH PIXEL ORTHOPHOTOS. GROUND CONTROL VALUES CHECKED ACAINS THE LIDAR SURFACE RESULTED IN AN RMSEZ OF 0.088 FT. PHOTO TRIANGULATION RESULTED IN RMS CONTROL OF X: 0.009, Y: 0.012, Z: 0.011,XY: 0.010

" 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 PRACTICE MANUAL FOR SURVEYING IN SOUTH CAROLINA, AND MEETS OR EXCEEDS THE REQUIREMENTS AS SPECIFIED THEREIN."

Raymond B Dawber 9/2/2021

RAYMOND B. DAWBER
SOUTH CAROLINA PROFESSIONAL LAND SURVEYOR
LICENSE NUMBER NO. 24270

SOUTH CAROLINA PROFESSIONAL LAND SURVEYOR No. 24270 RAYMOND B. DAWBER

SOUTH CAROLINA McKIM & CREED INC. NO. 453 CERTIFICATE OF AUTHORIZATION

SCALE: 1" = 150'
0 75 150 300 450

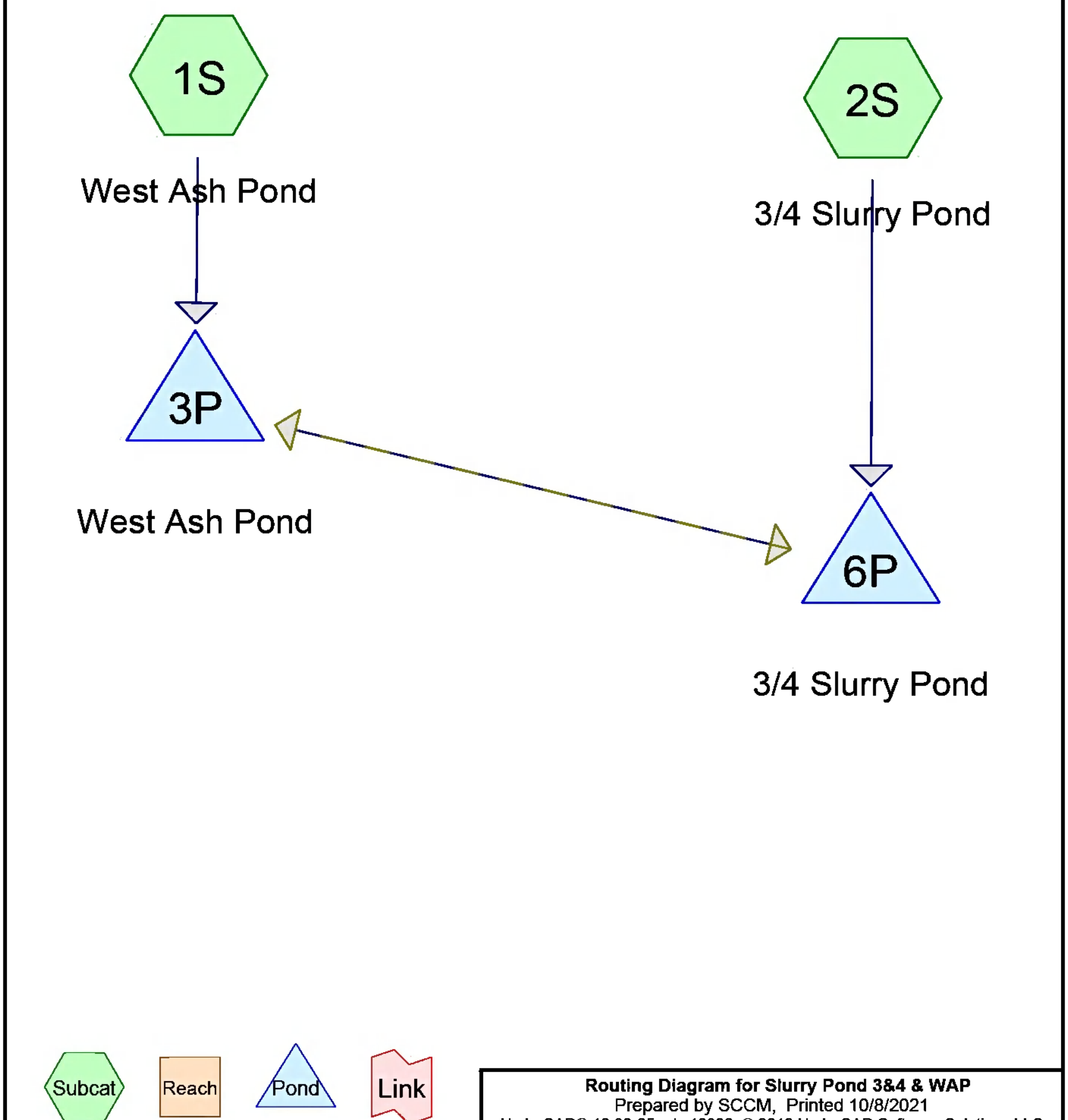
McKIM & CREED
454 DEANNA LANE SUITE A
CHARLESTON SC, 29492
TELEPHONE: (843) 459-7894
SOUTH CAROLINA FIRM COA NUMBER: 453

TOPOPGRAPHIC SURVEY
FOR
WINYAH GENERATING STATION
Slurry Ponds 3 & 4
SANTEE COOPER
LOCATION
07-24-2021

JOB NUMBER:	00633–0014
SCALE:	1" = 150'
CAD NUMBER:	CAD#
PLS:	RD
PARTY CHIEF:	JB
CAD TECH:	SV
FIELD BOOK/PAGE:	FB/PG
DRAWING NUMBER:	DWG

SHEET 1 OF 1

ATTACHMENT 2
HYDROLOGIC AND HYDRAULIC ANALYSIS RESULTS



Routing Diagram for Slurry Pond 3&4 & WAP
Prepared by SCCM, Printed 10/8/2021
HydroCAD® 10.00-25 s/n 10932 © 2019 HydroCAD Software Solutions LLC

Area Listing (selected nodes)

Area (acres)	CN	Description (subcatchment-numbers)
107.205	88	(2S)
64.385	98	Exposed Geomembrane (1S)
171.590	**92**	**TOTAL AREA**

Soil Listing (selected nodes)

Area (acres)	Soil Group	Subcatchment Numbers
0.000	HSG A	
0.000	HSG B	
0.000	HSG C	
0.000	HSG D	
171.590	Other	1S, 2S
171.590		**TOTAL AREA**

Ground Covers (selected 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		2S
0.000	0.000	0.000	0.000	64.385	64.385	Exposed Geomembrane	1S
0.000	**0.000**	**0.000**	**0.000**	**171.590**	**171.590**	**TOTAL AREA**	

Pipe Listing (selected nodes)

Line#	Node Number	In-Invert (feet)	Out-Invert (feet)	Length (feet)	Slope (ft/ft)	n	Diam/Width (inches)	Height (inches)	Inside-Fill (inches)
1	3P	24.96	23.87	99.1	0.0110	0.013	36.0	0.0	0.0
2	3P	24.90	23.64	98.7	0.0128	0.013	36.0	0.0	0.0
3	6P	23.87	24.96	99.1	-0.0110	0.013	36.0	0.0	0.0
4	6P	23.64	23.90	98.7	-0.0026	0.013	36.0	0.0	0.0

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

Subcatchment 1S: West Ash Pond Runoff Area=64.385 ac 100.00% Impervious Runoff Depth=50.76"
Flow Length=3,546' Slope=0.0025 '/' Tc=8.5 min CN=98 Runoff=1,203.47 cfs 272.327 af

Subcatchment 2S: 3/4 Slurry Pond Runoff Area=107.205 ac 0.00% Impervious Runoff Depth=49.40"
Flow Length=3,030' Tc=98.3 min CN=88 Runoff=987.21 cfs 441.321 af

Pond 3P: West Ash Pond Peak Elev=35.80' Storage=61.640 af Inflow=1,203.47 cfs 272.327 af
Primary=168.30 cfs 255.515 af Tertiary=220.33 cfs 25.039 af Outflow=380.58 cfs 280.554 af

Pond 6P: 3/4 Slurry Pond Peak Elev=33.06' Storage=479.354 af Inflow=1,327.53 cfs 721.876 af
Primary=160.23 cfs 608.983 af Tertiary=0.00 cfs 0.000 af Outflow=160.23 cfs 608.983 af

Total Runoff Area = 171.590 ac Runoff Volume = 713.648 af Average Runoff Depth = 49.91"
62.48% Pervious = 107.205 ac 37.52% Impervious = 64.385 ac

Summary for Subcatchment 1S: West Ash Pond

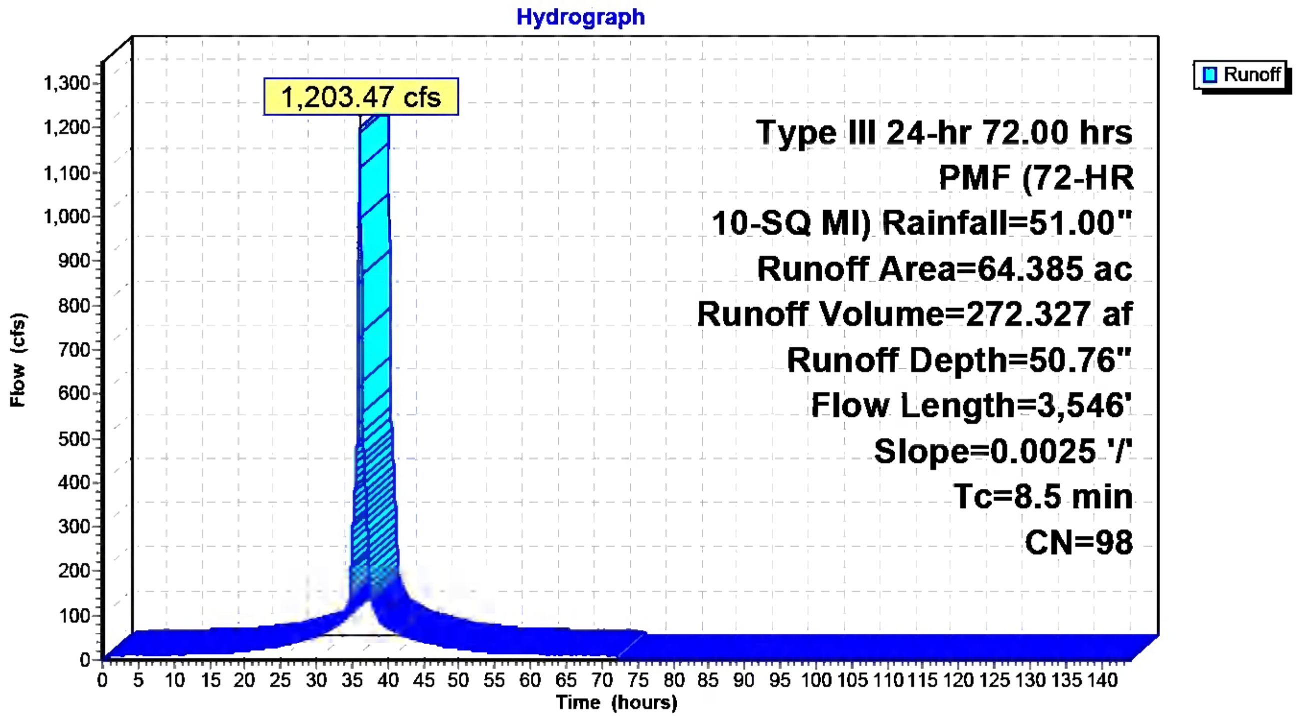
Runoff = 1,203.47 cfs @ 36.12 hrs, Volume= 272.327 af, Depth=50.76"

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 72.00 hrs PMF (72-HR, 10-SQ MI) Rainfall=51.00"

	Area (ac)	CN	Description	
ỹ	64.385	98	Ex osed Geomembrane	g
	64.385		100.00% Impervious Area	

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.6	302	0.0025	8.13	316.96	**Channel Flow, Channel A Flow** Area= 39.0 sf Perim= 23.0' r= 1.70' n= 0.013
2.1	1,022	0.0025	8.13	316.96	**Channel Flow, Channel B Flow** Area= 39.0 sf Perim= 23.0' r= 1.70' n= 0.013
5.8	2,222	0.0025	6.39	249.16	**Channel Flow, Channel C Flow** Area= 39.0 sf Perim= 33.0' r= 1.18' n= 0.013
8.5	3,546	Total			

Subcatchment 1S: West Ash Pond



Summary for Subcatchment 2S: 3/4 Slurry Pond

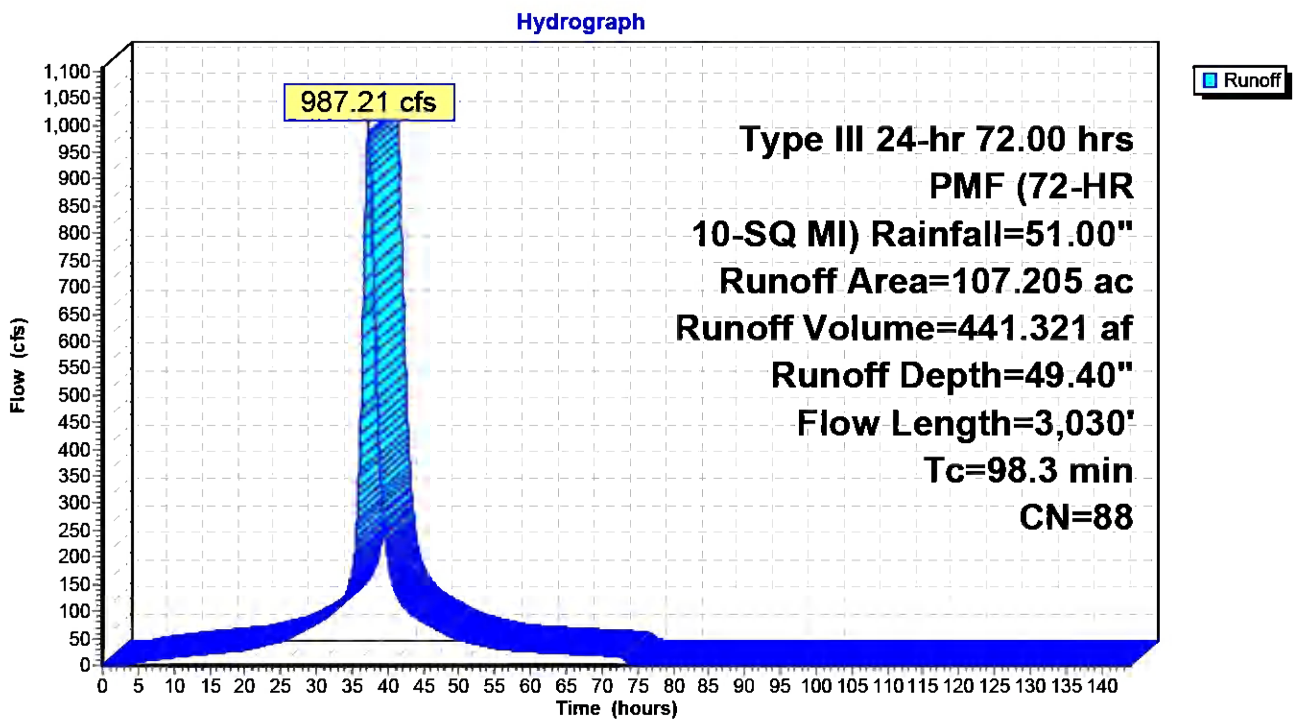
Runoff = 987.21 cfs @ 37.26 hrs, Volume= 441.321 af, Depth=49.40"

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 72.00 hrs PMF (72-HR, 10-SQ MI) Rainfall=51.00"

	Area (ac)	CN	Description
*	107.205	88	
	107.205		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
7.3	100	0.0005	0.23		**Sheet Flow, Sheet Flow** n= 0.020 P2= 4.38"
88.9	1,600	0.0009	0.30		**Shallow Concentrated Flow, Shallow Concentrated Flow** Nearly Bare & Untilled Kv= 10.0 fps
1.5	656	0.0025	7.11	739.18	**Channel Flow, Channel A Flow** Area= 104.0 sf Perim= 39.3' r= 2.65' n= 0.020
0.6	674	0.0169	18.48	1,921.88	**Channel Flow, Channel B Flow** Area= 104.0 sf Perim= 39.3' r= 2.65' n= 0.020
98.3	3,030	Total			

Subcatchment 2S: 3/4 Slurry Pond



Summary for Pond 3P: West Ash Pond

[44] Hint: Outlet device #1 is below defined storage
[44] Hint: Outlet device #2 is below defined storage
[86] Warning: Oscillations may require smaller dt (severity=283)

Inflow	=	1,203.47 cfs @	36.12 hrs,	Volume=	272.327 af
Outflow	=	380.58 cfs @	36.85 hrs,	Volume=	280.554 af, Atten= 68%, Lag= 43.6 min
Primary	=	168.30 cfs @	36.40 hrs,	Volume=	255.515 af
Tertiary	=	220.33 cfs @	36.89 hrs,	Volume=	25.039 af

Routing by Sim-Route method, Time Span= 0.00-600.00 hrs, dt= 0.050 hrs
Peak Elev= 35.80' @ 36.89 hrs Surf.Area= 25.815 ac Storage= 61.640 af

Plug-Flow detention time= (not calculated: outflow precedes inflow)
Center-of-Mass det. time= 115.5 min (2,292.5 - 2,177.0)

Volume	Invert	Avail.Storage	Storage Description
#1	26.00'	98.929 af	**Custom Stage Data (Prismatic)**Listed below (Recalc)

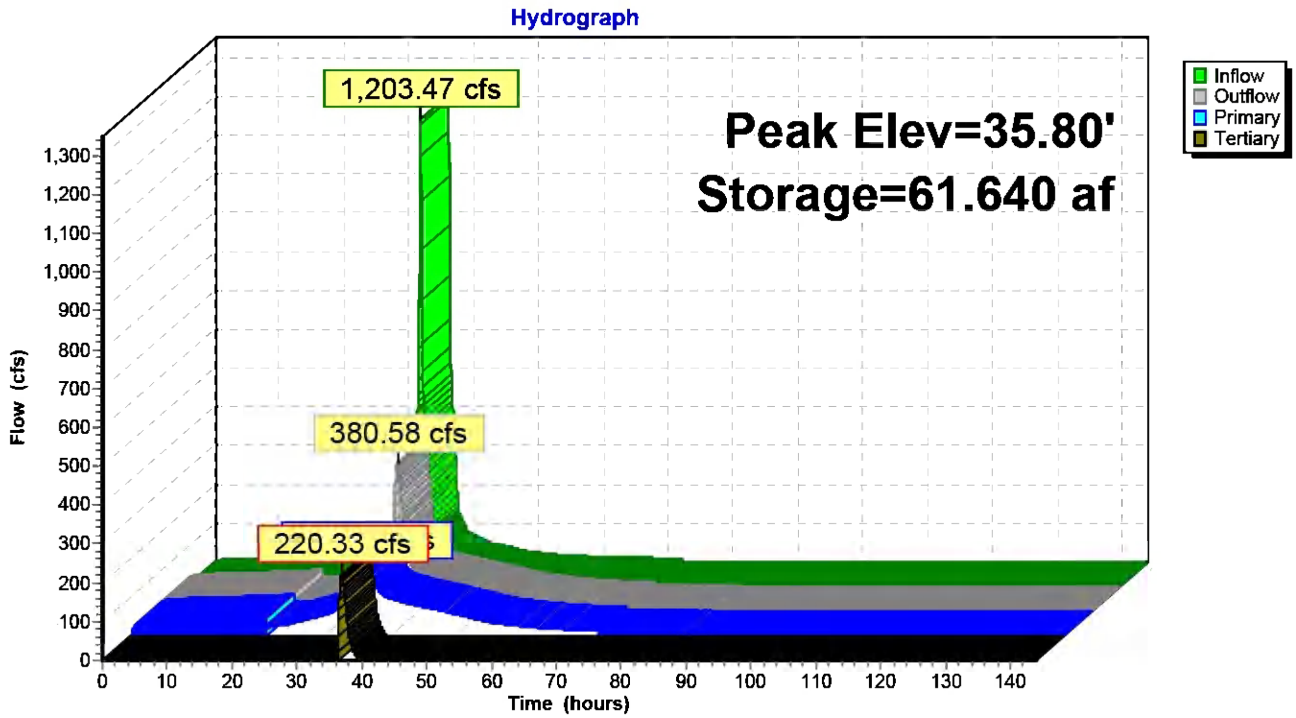
Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
26.00	0.004	0.000	0.000
27.00	0.182	0.093	0.093
28.00	0.481	0.331	0.425
29.00	0.932	0.706	1.131
30.00	1.599	1.265	2.397
31.00	2.978	2.288	4.685
32.00	5.395	4.186	8.872
33.00	8.851	7.123	15.995
34.00	13.446	11.149	27.143
35.00	19.354	16.400	43.543
36.00	27.418	23.386	66.929
37.00	36.582	32.000	98.929

Device	Routing	Invert	Outlet Devices
#1	Primary	24.96'	**36.0" Round Culvert 1** L= 99.1' CPP, mitered to conform to fill, Ke= 0.700 Inlet / Outlet Invert= 24.96' / 23.87' S= 0.0110 '/' Cc= 0.900 n= 0.013 Corrugated PE, smooth interior, Flow Area= 7.07 sf
#2	Primary	24.90'	**36.0" Round Culvert 2** L= 98.7' CPP, mitered to conform to fill, Ke= 0.700 Inlet / Outlet Invert= 24.90' / 23.64' S= 0.0128 '/' Cc= 0.900 n= 0.013 Corrugated PE, smooth interior, Flow Area= 7.07 sf
#3	Tertiary	35.25'	**200.0' long x 12.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.57 2.62 2.70 2.67 2.66 2.67 2.66 2.64
#4	Tertiary	35.75'	**30.0' long x 12.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.57 2.62 2.70 2.67 2.66 2.67 2.66 2.64

Primary OutFlow Max=167.42 cfs @ 36.40 hrs HW=35.58' TW=27.81' (Dynamic Tailwater)
1=Culvert 1 (Inlet Controls 83.71 cfs @ 11.84 fps)
2=Culvert 2 (Inlet Controls 83.71 cfs @ 11.84 fps)

Tertiary OutFlow Max=220.18 cfs @ 36.89 hrs HW=35.80' TW=28.86' (Dynamic Tailwater)
3=Broad-Crested Rectangular Weir (Weir Controls 219.29 cfs @ 1.99 fps)
4=Broad-Crested Rectangular Weir (Weir Controls 0.89 cfs @ 0.58 fps)

Pond 3P: West Ash Pond



Summary for Pond 6P: 3/4 Slurry Pond

[92] Warning: Device #4 is above defined storage

Inflow	=	1,327.53 cfs @ 37.17 hrs,	Volume=	721.876 af
Outflow	=	160.23 cfs @ 44.49 hrs,	Volume=	608.983 af, Atten= 88%, Lag= 439.6 min
Primary	=	160.23 cfs @ 44.49 hrs,	Volume=	608.983 af
Tertiary	=	0.00 cfs @ 0.00 hrs,	Volume=	0.000 af

Routing by Sim-Route method, Time Span= 0.00-600.00 hrs, dt= 0.050 hrs
Starting Elev= 18.60' Surf.Area= 9.203 ac Storage= 26.855 af
Peak Elev= 33.06' @ 44.49 hrs Surf.Area= 48.761 ac Storage= 479.354 af (452.499 af above start)

Plug-Flow detention time= 2,196.1 min calculated for 582.128 af (81% of inflow)
Center-of-Mass det. time= 1,849.6 min (4,146.6 - 2,297.0)

Volume	Invert	Avail.Storage	Storage Description
#1	11.00'	692.546 af	**Custom Stage Data (Prismatic)** Listed below

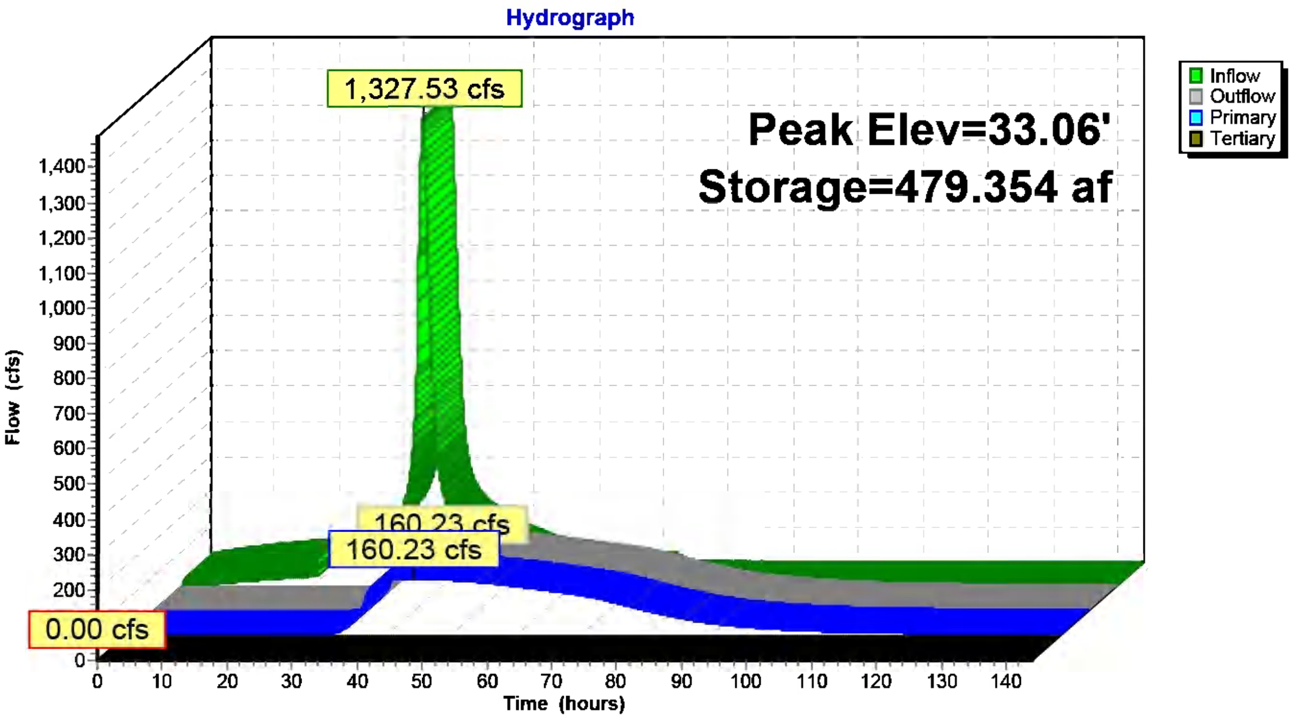
Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
11.00	0.216	0.000	0.000
12.00	0.486	0.351	0.351
14.00	1.904	2.390	2.741
16.00	4.777	6.681	9.422
18.00	5.756	10.533	19.955
20.00	17.245	23.001	42.956
22.00	24.986	42.231	85.187
24.00	29.380	54.366	139.553
26.00	31.772	61.152	200.705
28.00	35.815	67.587	268.292
30.00	39.208	75.023	343.315
32.00	45.474	84.682	427.997
34.00	51.692	97.166	525.163
35.67	148.767	167.383	692.546

Device	Routin	Invert	Outlet Devices
#1	Primary	24.96'	**36.0" Round Culvert 1** L= 99.1' CPP, mitered to conform to fill, Ke= 0.700 Inlet / Outlet Invert= 23.87' / 24.96' S= -0.0110 '/' Cc= 0.900 n= 0.013, Flow Area= 7.07 sf
#2	Primary	23.90'	**36.0" Round Culvert 2** L= 98.7' CPP, mitered to conform to fill, Ke= 0.700 Inlet / Outlet Invert= 23.64' / 23.90' S= -0.0026 '/' Cc= 0.900 n= 0.013, Flow Area= 7.07 sf
#3	Tertiary	35.25'	**200.0' long x 12.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.57 2.62 2.70 2.67 2.66 2.67 2.66 2.64
#4	Tertiary	35.67'	**30.0' long x 12.0' breadth Broad-Crested Rectangular Weir** Head (feet) 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 Coef. (English) 2.57 2.62 2.70 2.67 2.66 2.67 2.66 2.64

Primary OutFlow Max=160.23 cfs @ 44.49 hrs HW=33.06' (Free Discharge)
1=Culvert 1 (Inlet Controls 77.13 cfs @ 10.91 fps)
2=Culvert 2 (Inlet Controls 83.10 cfs @ 11.76 fps)

Tertiary OutFlow Max=0.00 cfs @ 0.00 hrs HW=18.60' TW=26.00' (Dynamic Tailwater)
3=Broad-Crested Rectangular Weir (Controls 0.00 cfs)
4=Broad-Crested Rectangular Weir (Controls 0.00 cfs)

Pond 6P: 3/4 Slurry Pond



Elevation in NAVD 88

ATTACHMENT 3

SEISMIC HAZARD EVALUATION AND SITE RESPONSE ANALYSIS

Written by: M. Limas/C. Carlson Date: 10/14/2021 Reviewed by: W. Shin/G. Rix Date: 10/14/2021

Client: Santee Cooper Project: Winyah Generating Station Project/ Proposal No.: GC8100 Task No: 03

SEISMIC HAZARD EVALUATION AND SITE RESPONSE ANALYSIS: SLURRY POND

PURPOSE

The purpose of this calculation package is to present the results of the seismic hazard evaluation and site response analyses performed for the Slurry Pond 3 & 4 (Slurry Pond) at Winyah Generating Station (WGS or Site). This calculation package is provided as Attachment 3 to *2021 Periodic Safety Factor Assessment* (2021 Safety Factor Assessment Report). Seismic hazard analysis for the Site includes the selection of an appropriate hazard level and associated hazard parameters. Based on the selected hazard level and associated hazard parameters, site response analyses were performed to evaluate the local site effects on the selected time history records propagated from the hypothetical firm ground outcrop to the ground surface of the Site. The objective of this site response analysis is to calculate accelerations and shear stresses within the critical representative soil profiles of the Slurry Pond perimeter dikes. Cyclic shear stresses will be used to evaluate liquefaction potential for dike fill and foundation soils and to calculate the seismic coefficient for seismic slope stability analyses presented in Attachments 4 and 5 to the 2021 Safety Factor Assessment Report, respectively.

SEISMIC HAZARD EVALUATION

Seismic hazard analysis for the Site includes the selection of: (i) appropriate hazard level; and (ii) associated hazard parameters. The appropriate hazard level is often expressed in probabilistic terms as a specific hazard level that has a certain probability of exceedance in a given time period. Selecting the hazard parameters includes developing an understanding of the seismic sources, ground motion attenuation, and site response. The goals of this section are to: (i) develop the target response spectrum, including the peak ground acceleration (PGA), at a hypothetical firm ground outcrop at WGS corresponding to the appropriate seismic hazard level; (ii) select the earthquake magnitude that contributes predominantly to the seismic hazard at WGS; and (iii) select a set of ground motion time histories that envelope the target spectrum, and are generally consistent with the source and path characteristics of ground motions at WGS.

Seismic Hazard Level

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). §257.63(a) of the CCR Rule states that:

"New CCR landfills, existing and new CCR surface impoundments, and all lateral expansions of CCR units must not be located in seismic impact zones, unless the owner or operator demonstrates by the dates specified in paragraph (c) of this section that all structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site."

§257.53 of the CCR Rule defines the maximum horizontal acceleration in lithified earth material as:

"... the maximum expected horizontal acceleration at the ground surface as depicted on a seismic hazard map, with a 98 percent or greater probability that the acceleration will not be exceeded in 50 years, or the maximum expected horizontal acceleration based on a site-specific seismic risk assessment."

A 98 percent or greater probability of not being exceeded in 50 years (or two percent probability of exceedance in 50 years) corresponds to a return period of approximately 2,500 years. The Preamble of the CCR Rule indicates that USEPA selected this return period by considering a typical operating life for CCR surface impoundments (i.e., 50 years) and its common use in seismic design criteria throughout engineering (e.g., American Society of Civil Engineers [ASCE] 7-16 [2016]). For the CCR surface impoundments at WGS, closure was initiated in 2021 and is expected to be complete in less than 15 years. Therefore, an earthquake return period of approximately 750 years was conservatively selected for the 2021 Safety Factor Assessment of the Slurry Pond (i.e., two percent probability of exceedance in 15 years) following the basis for selecting the return period of approximately 2,500 years for typical CCR surface impoundments. A 750-year return period is approximately equivalent to an annual frequency of exceedance of 1.33E-03.

Peak Ground Acceleration (PGA)

PGA values corresponding to different hazard levels and different site conditions, including firm ground outcrops, are published as seismic hazard maps or curves. The 2016 Safety Factor Assessment Report (Geosyntec, 2016) referenced seismic hazard maps presented in the South Carolina Department of Transportation (SCDOT) Geotechnical Design Manual (GDM) (SCDOT, 2010) for selection of a PGA to incorporate local site effects for the Charleston Seismic Zone researched by Chapman and Talwani (2006). The GDM was updated in 2019 (SCDOT, 2019) and does not present the seismic hazard maps referenced in the 2016 Safety Factor Assessment Report. Moreover, SCDOT is updating seismic hazard maps at the time of this seismic hazard evaluation.

As an alternative, United States Geological Survey (USGS) hazard curves for two percent probability of exceedance in 15 years (i.e., approximately 750-year return period event) at the BC boundary (i.e., boundary between National Earthquake Hazard Reduction Program [NEHRP] site classes B and C with a mean shear wave velocity of 2,500 ft/s) were used to estimate the PGA and spectral accelerations for a hypothetical firm ground outcrop, similar to "geologically realistic" site conditions, at the Site. The data available at the USGS website (Petersen et al., 2019) use pre-calculated hazard values at nearby grid locations and interpolate the hazard value for a given site location. As presented in Figure 1, the interpolated PGA from USGS Hazard Curves is 0.15g for the Site.

Earthquake Magnitude

In a probabilistic seismic hazard analysis, the PGA cannot be associated with a single earthquake event due to the hazard contribution from multiple possible events. An earthquake moment magnitude (M_w) value is required to conduct liquefaction potential analyses and select earthquake time histories. A process called deaggregation can be performed for sites that have multiple hazard sources using the most up-to-date USGS (2014) deaggregation tool.

Figure 2 presents the deaggregation for the PGA at the Site. A 7.3 moment magnitude earthquake event at a source-to-site distance of approximately 70 km is the modal event contributing to the hazard at the Site. Thus, a 7.3 moment magnitude was selected for liquefaction potential analyses and time history selection for WGS.

Target Acceleration Response Spectra

Using the USGS hazard curves, the uniform hazard spectrum (UHS) was developed for an approximately 750-year return period event at the BC boundary to represent the "geologically realistic" target acceleration response spectrum for WGS (Figure 3). The "geologically realistic" target acceleration response spectrum has a PGA (represented by a spectral period of 0.005 seconds) of 0.15g and a peak spectral acceleration of 0.40g at a spectral period of 0.075 seconds.

Time Histories

Time histories of ground motions are used as input for site response analysis and are selected such that their response spectra match or exceed the target spectrum. While use of recorded ground motion time histories from earthquakes with similar source characteristics is preferred, synthetic motions may be used if recordings are not available for a particular tectonic setting. Earthquake events with a moment magnitude, M_w, 7.0 or greater have not occurred in the stable continental tectonic environment of the Central and Eastern United States since the Charleston earthquake in

1886, so ground motion time history records matching the seismic source characteristics for the WGS are generally not available. Two synthetic acceleration time histories were selected from the six synthetic acceleration time histories developed for the Site using the USGS Interactive Deaggregation tool (USGS, 2002). These time histories are referred to herein as Winyah1 and Winyah2, and provide a reasonable match to the short-period portion of the "geologically realistic" target acceleration response spectrum. Three time histories, BOS-T1, DEL090, and YER360, were selected to provide a conservative envelope for the long-period portion of the "geologically realistic" target acceleration response spectrum. The three time histories were developed by McGuire et al. (2001) as part of a study for the Nuclear Regulatory Commission to provide time histories representative of expected earthquake events in the Central and Eastern United States. Also, one time history, RSN8529-HNE, from the Next Generation Attenuation – East (NGA East) database (Goulet et al., 2014), which provides a database of time histories recorded for earthquake events in the Central and Eastern United States, was selected to also provide a conservative envelope for the long-period portion of the "geologically realistic" target acceleration response spectrum for longer periods. As shown in Figure 4, this suite of six time histories provides a reasonably conservative envelope of the "geologically realistic" target spectrum for the Site over a broad range of periods. Time histories were scaled in the site response evaluation computer program to match the target PGA of 0.15g. These scaled acceleration time histories are presented in Appendix 1. Additional details of the time histories are presented in Table 1.

SITE RESPONSE ANALYSIS

Site response analyses were performed to evaluate the effect of local site conditions on the propagation of earthquake ground motions at the Site. The objective of the site response analysis for the 2021 Safety Factor Assessment Report is to calculate accelerations and shear stresses at the critical profiles observed in the 2016 Safety Factor Assessment (Geosyntec, 2016). Calculated shear stresses are used to evaluate the liquefaction potential analysis (Attachment 4 to the 2021 Safety Factor Assessment Report) and seismic stability analysis (Attachment 5 to the 2021 Safety Factor Assessment Report) for the Slurry Pond perimeter dikes.

Methodology for Site Response Analysis

Site response analyses presented herein were conducted using DEEPSOIL® (Hashash et al., 2020), a one-dimensional nonlinear site response analysis program. The program assumes that all the soil layers are perfectly horizontal (i.e., "layer cake") and that ground response is mainly caused by vertically-propagating, horizontally polarized shear waves. This assumption is valid for many geotechnical cases including the analyses of the Site. Under these assumptions, the subsurface stratigraphy is modeled as a one-dimensional column of soil layers for the analyses.

DEEPSOIL® employs a viscoelastic material model, described by its shear modulus (G), mass density (ρ) or unit weight (γ), and damping (D). Preliminary equivalent-linear site response analyses yielded relatively large maximum shear strains in some layers, which are greater than the shear strains for which equivalent-linear analyses are considered applicable (i.e., one to two percent) (Kaklamanos et al., 2013). Therefore, nonlinear site response analyses were performed.

Input Parameters for Site Response Analysis

Input Motions

As discussed in the Time Histories subsection, six acceleration time histories were selected and scaled to match the target PGA of 0.15g. These ground motions were applied as outcrop motions in DEEPSOIL® at the top of the half space.

Representative Soil Profile

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 2016 Safety Factor Assessment Report. Also, no additional geotechnical subsurface investigations have been conducted since 2016. Therefore, the subsurface stratigraphy developed in the 2016 Safety Factor Assessment is still valid and was also used for the 2021 Safety Factor Assessment. A detailed description of the subsurface stratigraphy is presented in the 2016 Safety Factor Assessment Report (Geosyntec, 2016). Information that is specific to the site response analysis is presented herein.

To develop representative soil profiles, the Slurry Pond perimeter dike was divided into three sections depending on the depth of the dike fill and the V_s profile of the subsurface as shown in Figure 5. The top of the perimeter dike is relatively flat, whereas the base of the dike toe undulates with free-field ground surface elevation. In general, the base of the northwestern dike is lower (i.e., the dike is taller) than the rest of the dike (Profile 3 in Figure 5). The remainder of the perimeter dike was divided into two sections based on the V_s profile. As such, three representative profiles to 100 ft below ground surface (bgs) were developed for the perimeter dike (Profiles 1, 2, and 3). The 2016 Safety Factor Assessment identified zones of subsurface materials with lower calculated factors of safety against triggering of liquefaction and sections with lower calculated factors of safety for slope stability along the east (Profile 1) and north (Profile 2) side of the Slurry Pond (Geosyntec, 2016). Therefore, site response analyses were only performed for Profiles 1 and

2 for the 2021 Safety Factor Assessment Report to provide an updated evaluation of the critical area of the Slurry Pond. The two profiles are shown in Figure 6.

A review of the topographic survey data from September 2021 and the topographic survey used in the 2016 Safety Factor Assessment indicated the surface of the residuals in the Slurry Pond is similar to that used for the 2016 Safety Factor Assessment. Santee Cooper provided available water level measurements from wells in the area of the Slurry Pond, located outside the downstream toe of the perimeter dike. The recorded water levels in these wells have generally been steady over the last five years. Based on the review of the topographic survey and limited water level measurements adjacent to the Slurry Pond perimeter dikes, the water level within the perimeter dike is expected to be similar to the water level used for the 2016 Safety Factor Assessment. The site response analyses for the Slurry Pond in the 2016 Safety Factor Assessment Report considered a water table 15 ft bgs. Therefore, site response analyses for the 2021 Safety Factor Assessment were performed with the water table modeled at 15 ft bgs.

Profiles 1 and 2 were extended to a depth of 500 ft bgs using information on deep V_s profiles derived from URS (2001) and S&ME (2001). At that depth, the deep V_s profiles indicate the presence of firm Coastal Plain sediments with V_s of approximately 2,300 ft/s, which is consistent with the definition of "geologically realistic" soil conditions and approximately represents the BC boundary. The site response analysis presented in this package thus considers the full depth of the soil columns (i.e., 500 ft bgs), but results are presented for the soil columns to a depth of 100 ft bgs to emphasize the near-surface response.

Dynamic Soil Properties

Shear Modulus Reduction and Damping Curves

The modified Kondner-Zelasko model implemented in DEEPSOIL® is described in Matasovic (1993). The shear modulus reduction and damping curves are required as input parameters for the constitutive soil model, and were developed with consideration of regional soil characteristics based on guidance presented in the SCDOT GDM (2019) and previous geotechnical reports of the Site. Adopting relationships proposed by Stokoe et al. (1995 and 1999), Andrus et al. (2003) developed regression equations for shear modulus reduction and damping curves suitable for South Carolina soils. The regression equations are presented in the SCDOT GDM (2019). These region-specific curves are a function of the plasticity index (PI) of the soil, effective mean stress, and geologic age and location of soil deposits. Geologic interpretation of the foundation soil at WGS by Paul C. Rizzo Associates (PCRA) (PCRA, 1999) and the SC Department of Natural Resources (DNR) (2012) indicates the native foundation soils above the Chicora and

Williamsburg Formation strata are Pleistocene deposits. The dike fill soils were considered to be a Holocene deposit, since the perimeter dikes were constructed of compacted earthen fill in 1979-1980. The shear modulus reduction and damping curves were calculated for the dike fill and foundation soils located above the Chicora and Williamsburg Formation strata. Soft rock curves (Silva et al., 1997) were selected for the Chicora and Williamsburg Formation strata to be consistent with the V_s-based classification indicating soft rock conditions. Pacific Engineering (S&ME, 2001) also used these soft rock shear modulus reduction and damping curves to perform the site response analysis of an ammonia tank building onsite. Figure 7 presents shear modulus reduction and damping curves used for these analyses. An example of the development of the dynamic curves and the references are provided in Appendix 2.

Representative Shear Wave Velocity Profile

Geosyntec developed representative V_s profiles of the dike fill and foundation soils using both direct measurements from Seismic Cone Penetration Tests (SCPTs) and estimates using Cone Penetration Tests (CPTs) and associated correlations. Upon evaluation of several correlations, the Mayne (2006) correlation was found to agree most closely with results of site-specific V_s measurements. This correlation is as follows:

$$V_s = 118.8 \log (f_s) + 18.5$$

where,

V_s = shear wave velocity (m/sec); and
f_s = sleeve friction from CPT (kPa).

Appendix 3 presents SCPT measurements, estimated values, and the selected V_s profiles. Figure 8 shows the shallow (depths less than 100 ft bgs) V_s profiles used for the site response analyses presented herein. As described previously, these profiles were extended to a greater depth to layers with V_s of approximately 2,300 ft/s to be consistent with the definition of "geologically realistic" soil conditions.

Unit Weight

Unit weights of the dike fill and foundation soils were selected predominantly based on laboratory measured values as presented in the 2016 Safety Factor Assessment Report (Geosyntec, 2016). The selected unit weights of the dike and foundation soils were 125 pcf. Unit weights of the Chicora and Williamsburg Formation soils were assumed to be 130 pcf and 105 pcf, respectively, based on Standard Penetration Test (SPT) N-values and material descriptions presented in the

PCRA (1999) report. Williamsburg Formation soils at depths greater than approximately 110 feet bgs were assumed to have unit weights of 125 pcf.

Site Response Analysis Results

Figures 9a and 9b show calculated maximum shear strain and shear stress profiles for Profiles 1 and 2, respectively. The maximum shear strains produced by two of the motions (BOS-T1 and YER360) are relatively large in the foundation soils, supporting the use of nonlinear site response analyses. Calculated acceleration profiles within the soil profiles are presented in Appendix 4. The envelopes of maximum shear strain and shear stress for the six motions for each profile are presented in Figure 10. The calculated envelopes of maximum shear stress (τ_{max}) values for different depths are presented in Table 2. These values were used to calculate cyclic stress ratios for the evaluation of liquefaction potential (Attachment 4 to the 2021 Safety Factor Assessment Report) and to calculate the seismic coefficient for seismic stability analyses (Attachment 5 to the 2021 Safety Factor Assessment Report).

CONCLUSIONS

- The design PGA was conservatively selected to be 0.15 g. This firm ground PGA corresponds to an event with a probability of exceedance of two percent in 15 years (i.e., event with a 750-year return period) and is representative of a motion expected for the "geologically realistic" site condition presented in the SCDOT GDM (2019).

- The design earthquake was assumed to have an M_w of 7.3 based on the deaggregation of the probabilistic seismic hazard analysis. This M_w was used for soil liquefaction analysis and time history selection.

- A target response spectrum for "geologically realistic" site conditions was developed using the USGS seismic hazard curves (Petersen et al., 2019) and is presented in Figure 2.

- Six time history recordings were used for the site response analyses. Two synthetic time histories were obtained using the USGS Interactive Deaggregation tool (USGS, 2002), three of the time histories were selected from the McGuire et al. (2001) database, and one of the time histories was selected from the NGA East database (Goulet et al., 2014). The time histories were scaled to match the design PGA of 0.15g for the site response analyses.

- Nonlinear site response analyses were conducted using DEEPSOIL® (Hashash et al., 2020). The two soil profiles identified in the 2016 Safety Factor Assessment were used for the site response analyses. The analyses used region-specific shear modulus reduction and damping curves. The shear wave velocity profiles were estimated from measured SCPT values and correlations between V_s and measured CPT sleeve frictions. The inputs used for each profile in DEEPSOIL® are shown in Appendix 5.

- The site response analysis results are presented in Figures 9a, 9b, and 10. The calculated maximum shear stresses are presented in Table 2 and are used for evaluation of soil liquefaction potential and calculation of the seismic coefficient for seismic stability analyses.

REFERENCES

American Society of Civil Engineers (ASCE), (2017), "Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16)", American Society of Civil Engineers, Reston, Virginia.

Andrus, R.D., Zhang, J., Ellis, B.S., and Juang, C.H., (2003), "Guide for Estimating the Dynamic Properties of South Carolina Soils for Ground Response Analysis", South Carolina Department of Transportation, SC-DOT Research Project No. 623, FHWA-SC-03-07.

Chapman, M.C. and Talwani, P., (2006), "Seismic Hazard Mapping for Bridge and Highway Design in South Carolina", South Carolina Department of Transportation, FHWA-SC-06-09.

Geosyntec Consultants, Inc. (2016), "2016 Surface Impoundment Periodic Safety Factor Assessment Report: Slurry Pond, Winyah Generating Station, Georgetown, South Carolina", submitted to Santee Cooper.

Goulet, C.A., Kishida, T., Ancheta, T.D., Cramer, C.H., Darragh, R.B., Silva, W.J., Hashash, Y.M.A., Harmon, J., Stewart, J.P., Wooddell, K.E., and Youngs, R.R., (2014), "PEER NGA-East Database", Pacific Earthquake Engineering Research Center, PEER 2014/17.

Hashash, Y.M.A., Musgrove, M.I., Harmon, J.A., Ilhan, O., Xing, G., Numanoglu, O., Groholski, D.R., Phillips, C.A., and Park, D. (201520), "DEEPSOIL 7.06.1, User Manual", Board of Trustees of University of Illinois at Urbana-Champaign, Urbana, Illinois.

Kaklamanos, J., Bradley, B.A., Thompson, E.M., and Baise, L.G. (2013). "Critical parameters affecting bias and variability in site response analyses using KiK-net downhole array data." Bulletin of the Seismological Society of America, Vol. 103, No. 3, pp. 1733-1749.

Matasovic, N. (1993), "Seismic Response of Composite Horizontally-Layered Soil Deposits", Ph.D. Dissertation, University of California, Los Angeles, California.

Mayne, P.W., (2006), "The 2nd James K. Mitchell Lecture: Undisturbed Sand Strength from Seismic Cone Tests", Geomechanics and Geoengineering, Vol. 1, No. 4, 2006, pp.239–247.

McGuire, R.K., Silva, W.J., and Constantino, C.J., (2001), "Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines", United States Nuclear Regulatory Commission, NUREG/CR-6728.

Paul C. Rizzo Associates, (1999), "Geotechnical/Hydrogeologic Investigation Winyah Generating Station", Georgetown, South Carolina, submitted to Santee Cooper.

Petersen, M.D., Shumway, A.M., Powers, P.M., Mueller, C.S., Moschetti, M.P., Frankel, A.D., Rezaeian, S., McNamara, D.E., Luco, N., Boyd, O.S., Rukstales, K.S., Jaiswal, K.S., Thompson, E.M., Hoover, S.M., Clayton, B.S., Field, E.H., and Zeng, Y. (2019), "The 2018 update of the US national seismic hazard model: Overview of model and implications", Earthquake Spectra, Vol. 36, No. 1, 2019, pp. 5-41.

S&ME, Inc., (2001), "Report of Geotechnical Exploration Winyah Generating Station Units 1&2 Ammonium Sulfate FGD System", Georgetown, South Carolina, submitted to Marsulex Environmental Technologies.

Silva, W.J., N. Abrahamson, G. Toro, and C Costantino (1997), "Description and validation of the stochastic ground motion model", Submitted to Brookhaven National Laboratory, Associated Universities, Inc. Upton, New York.

South Carolina Dept. of Transportation (SCDOT) (2010), Geotechnical Design Manual, available: http://www.scdot.org/doing/structural_Geotechnical.aspx

South Carolina Dept. of Transportation (SCDOT) (2019), Geotechnical Design Manual Version 2.0, available: https://www.scdot.org/business/pdf/geotech/SCDOT-Geotechnical-Design-Manual.pdf

South Carolina Department of Natural Resources: Geologic Survey, (2012). "Geologic Map of the Georgetown South Quadrangle, Georgetown County, South Carolina", 2012.

Stokoe, K. H., II, Hwang, S. K., Darendeli, M. B., and Lee, N. J. (1995), "Correlation Study of Nonlinear Dynamic Soils Properties", final report to Westinghouse Savannah River Company, The University of Texas at Austin, Austin, TX.

Stokoe, K. H., II, Darendeli, M. B., Andrus, R. D., and Brown, L. T. (1999), "Dynamic Soil Properties: Laboratory, Field and Correlation Studies", Proceedings, 2nd International Conference on Earthquake Geotechnical Engineering, Vol. 3, Lisbon, Portugal, 811-845.

URS Corporation, Durham Technologies, Inc., Image Cat, Inc., Pacific Engineering and Analysis, S&ME, Inc. (2001), "Comprehensive Seismic Risk and Vulnerability Study for the State of South Carolina", prepared for South Carolina Emergency Preparedness Division, 51-D0111027.00, Final Report, 10 September 2001.

USGS, (2002), "2002 Interactive Deaggregation", 2002. https://geohazards.usgs.gov/deaggint/2002/index.php

U.S. Geological Survey (USGS), (2014), "Dynamic: Conterminous U.S. 2014 (v4.2.0) Interactive Deaggregations", 2014. https://earthquake.usgs.gov/hazards/interactive/

Tables

Table 1. Summary of Hazard Parameters of the Time Histories Selected for Site Response Analysis

Name	**Site Class**	**M_w**	**R (km)**	**PGA (g)**	**T_p (s)**
BOS-T1	-	7.40	26.1	0.14	0.36
DEL090	C	6.70	59.3	0.27	0.22
RSN8529-HNE	C	5.74	124.1	0.09	0.26
Winyah1	A	7.04	30.2	0.56	0.08
Winyah2	A	7.04	30.2	0.56	0.10
YER360	C	7.30	24.9	0.22	0.22

Note:

1. All accelerations are scaled within DEEPSOIL® to match the target PGA of 0.15g.

Table 2. Calculated Maximum Shear Stress Envelopes

Profile 1		Profile 2	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	38	2.5	35
7.5	94	7.5	93
12.5	151	12.5	138
17.5	206	17.5	187
22.5	258	22.5	235
27.5	299	27.5	272
32.0	327	32.0	293
36.0	346	36.0	301
40.0	353	40.0	305
44.0	353	44.0	304
48.0	363	48.0	313
52.5	440	52.5	389
60.0	551	60.0	487
70.0	612	70.0	582
80.0	728	80.0	674
90.0	860	90.0	815
100.0	989	100.0	919

Figures

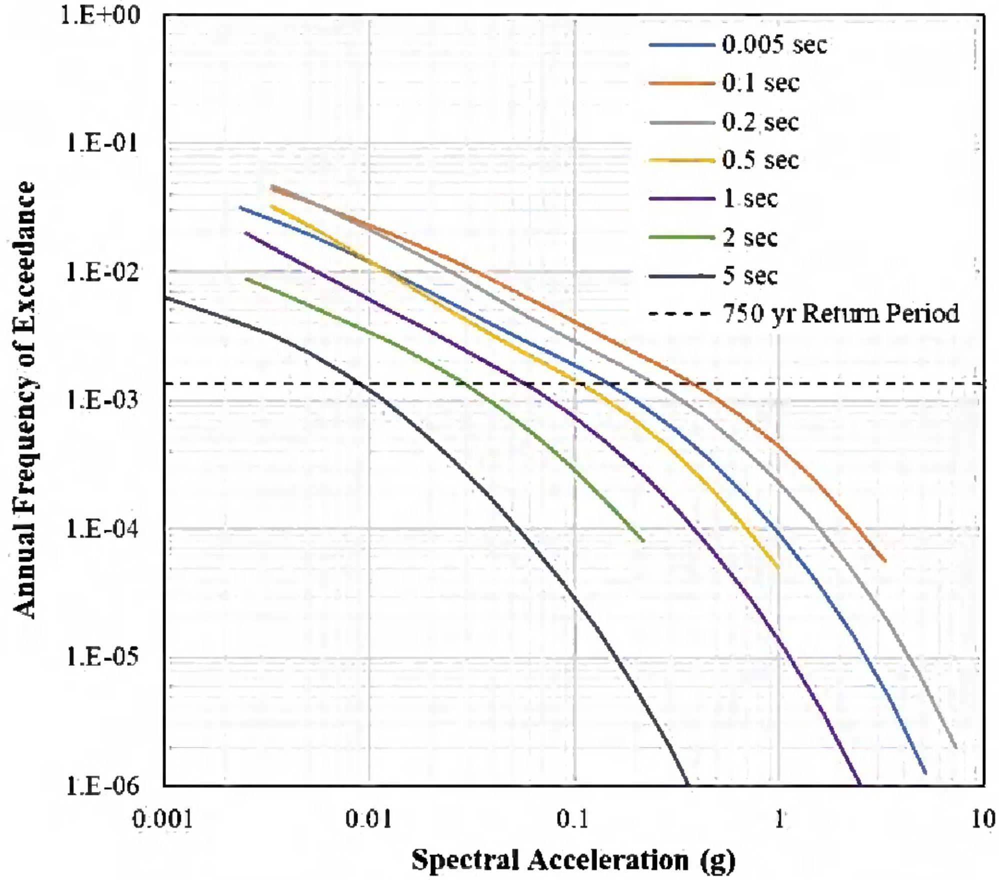
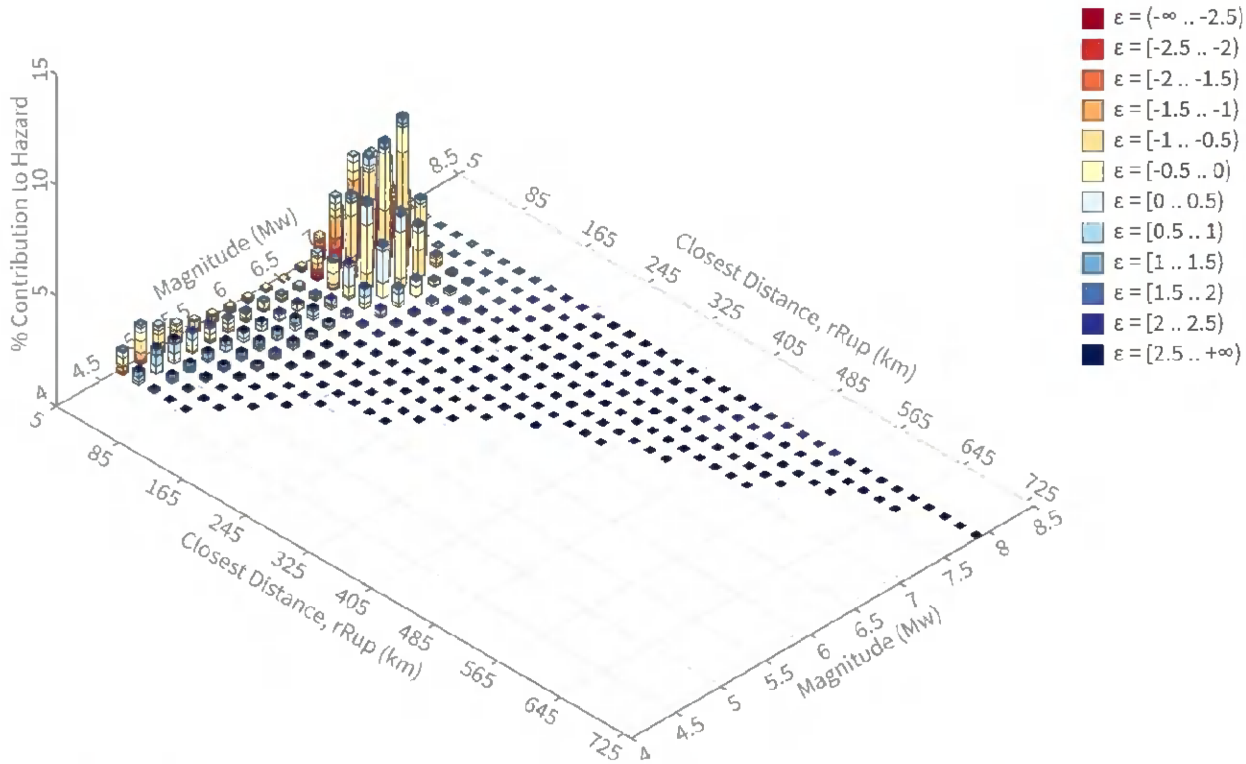


Figure 1. USGS Hazard Curves for Geologically Realistic Conditions (BC Boundary) at the Site (Petersen et al., 2019)

Notes:

1. The different hazard curves represent different spectral periods. USGS provides hazard curves for 22 spectral periods; however, hazard curves are only shown for 7 spectral periods in the figure for clarity.
2. The dashed line represents a hazard with a 750-year return period (i.e., approximately annual frequency of exceedance of 1.33E-03). The intersections of the dashed lines and curves for the different spectral periods were used to develop the UHS for the Site shown in Figure 3.
3. PGA is approximated by a spectral period of 0.005 seconds. As illustrated, PGA for WGS was selected as 0.15g.



Deaggregation targets

Return period: 750 yrs
Exceedance rate: 0.0013333333 yr^{-1}
PGA ground motion: 0.12257412 g

Recovered targets

Return period: 748.46717 yrs
Exceedance rate: 0.0013360639 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.93 %

Mean (over all sources)

m: 6.76
r: 52.82 km
ε_0: -0.76 σ

Mode (largest m-r bin)

m: 7.3
r: 68.52 km
ε_0: -0.68 σ
Contribution: 6.82 %

Mode (largest m-r-ε_0 bin)

m: 7.3
r: 70.18 km
ε_0: -0.76 σ
Contribution: 3.14 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Figure 2. Deaggregation of Two Percent Probability of Exceedance in 15 Years at the BC Boundary of the Site

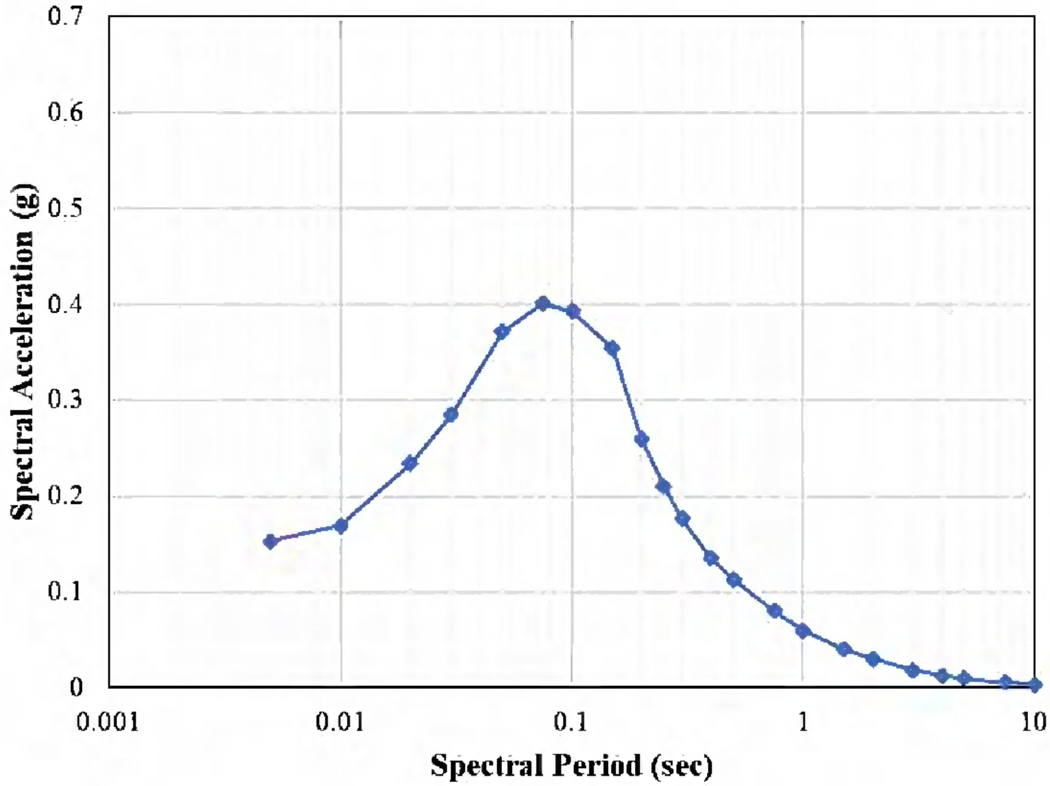


Figure 3. “Geologically Realistic” Target Response Spectrum for WGS

Notes:

1. Target response spectrum shown for “geologically realistic” was developed using USGS seismic hazard curves (see Figure 1).

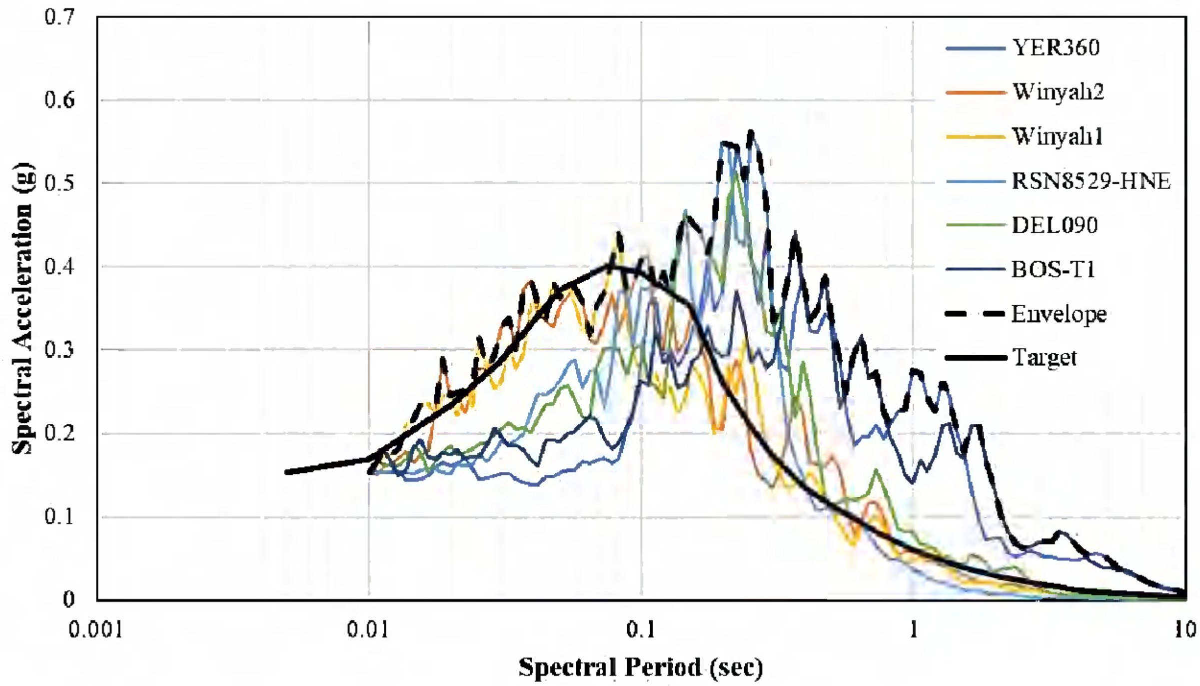


Figure 4. Response Spectra of Scaled Time Histories Selected for Seismic Evaluations

Note:

1. Time histories were scaled to match the target PGA = 0.15g (represented by a period of 0.005 seconds).

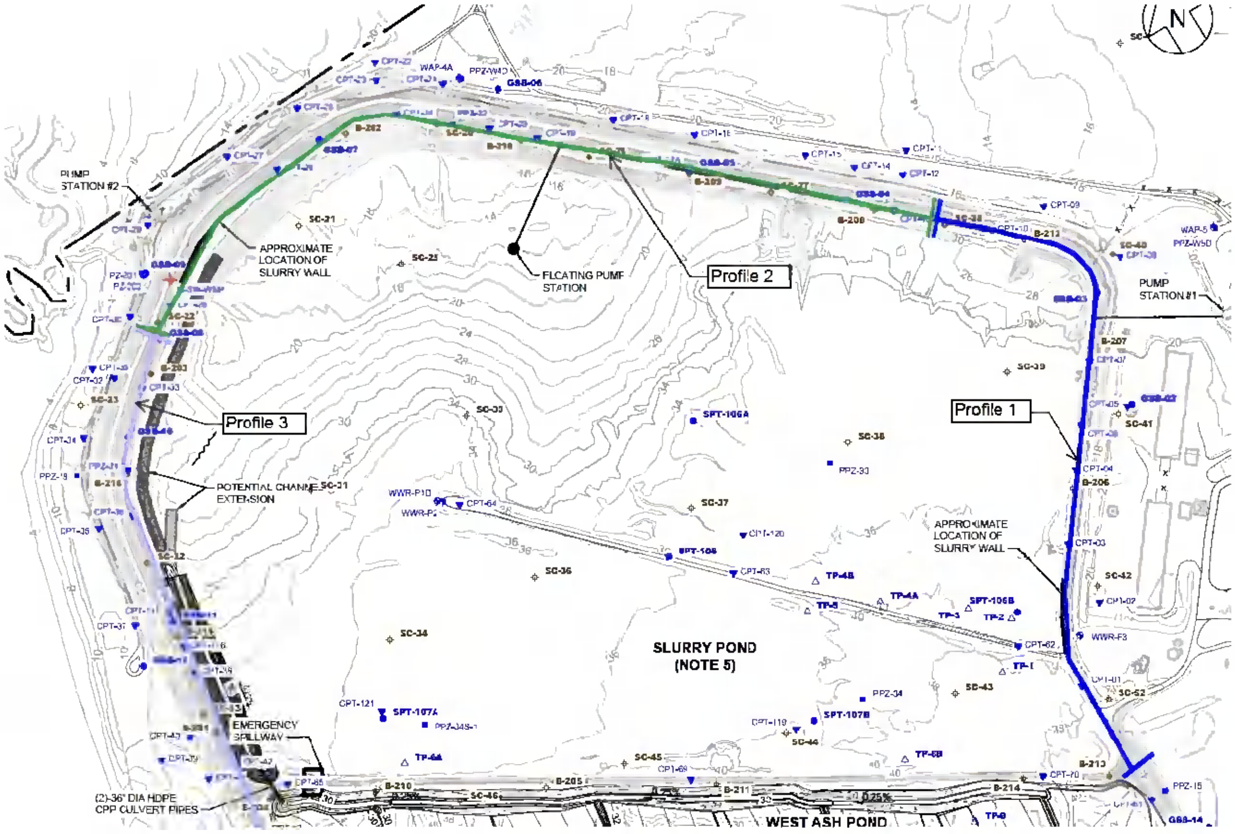


Figure 5. Locations of Representative Soil Profiles

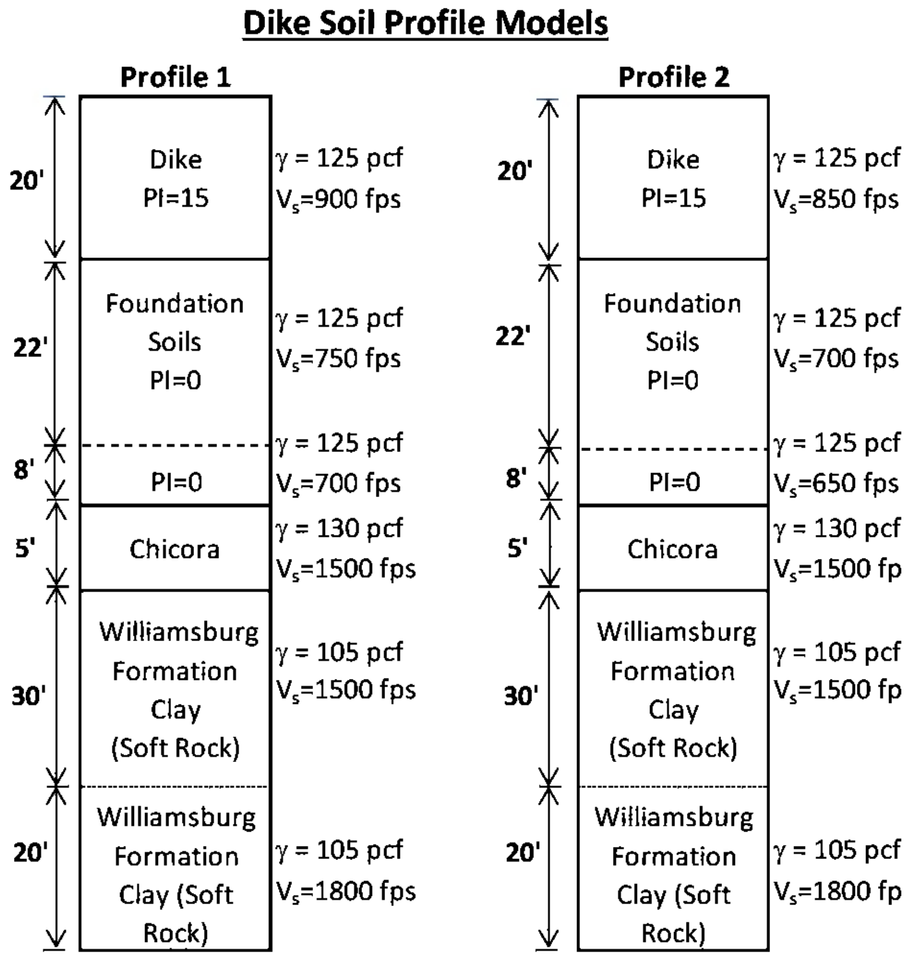


Figure 6. DEEPSOIL® Soil Profile Models for the Two Representative Dike Profiles

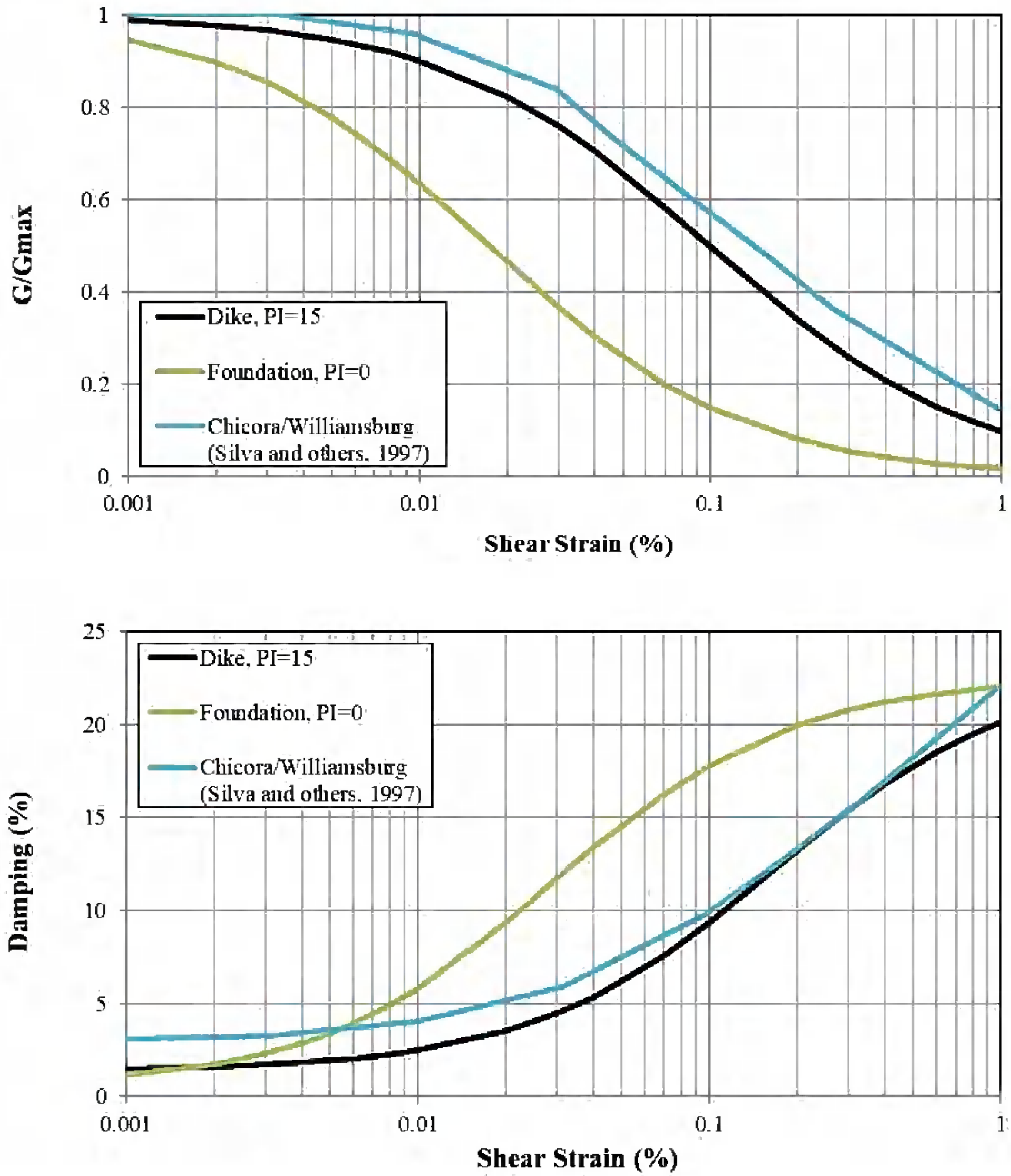


Figure 7. Shear Modulus Reduction (G/G_{max}) and Damping Curves for Soils Used in Site Response Analyses

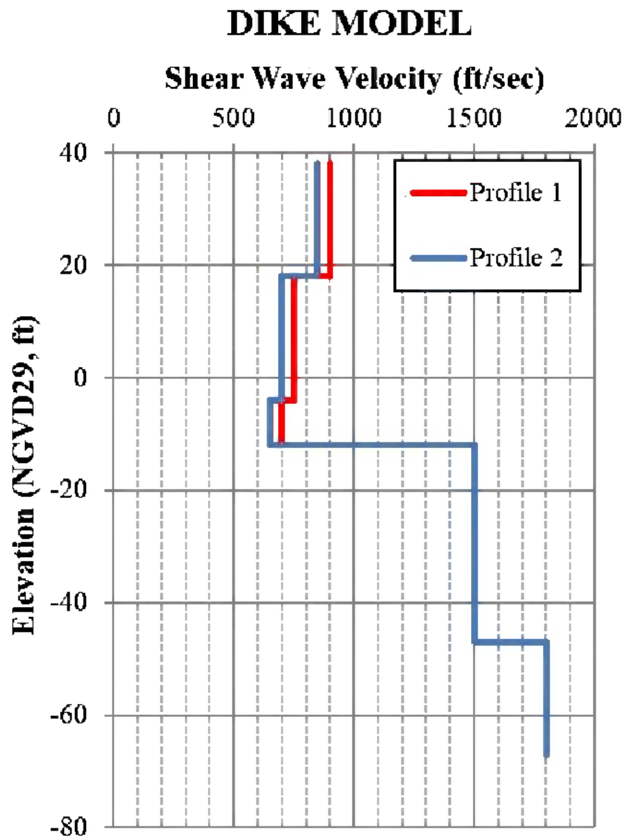


Figure 8. Selected Shear Wave Velocity (V_s) Profiles for Site Response Analyses

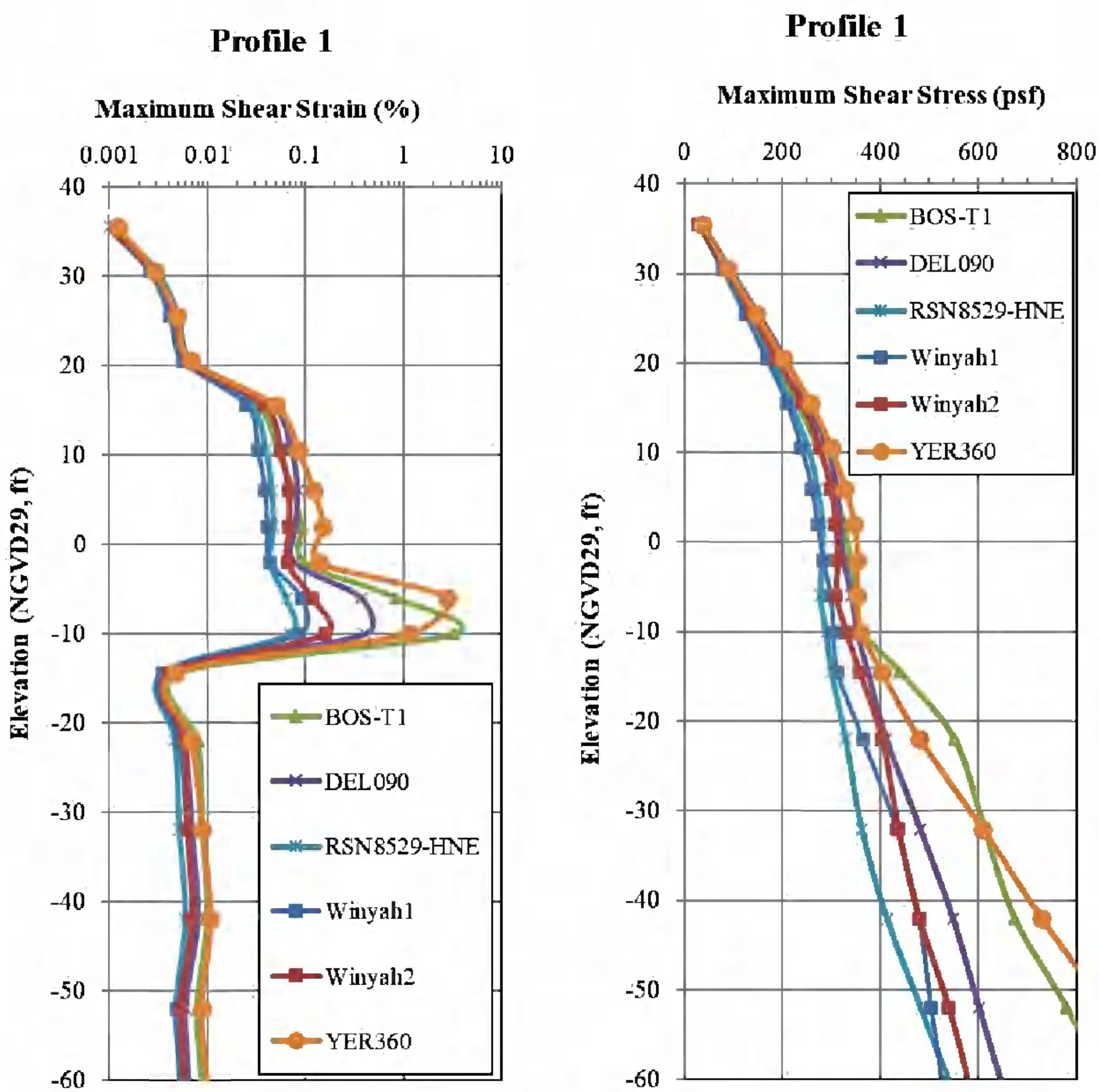


Figure 9a. Site Response Analysis Results for Profile 1

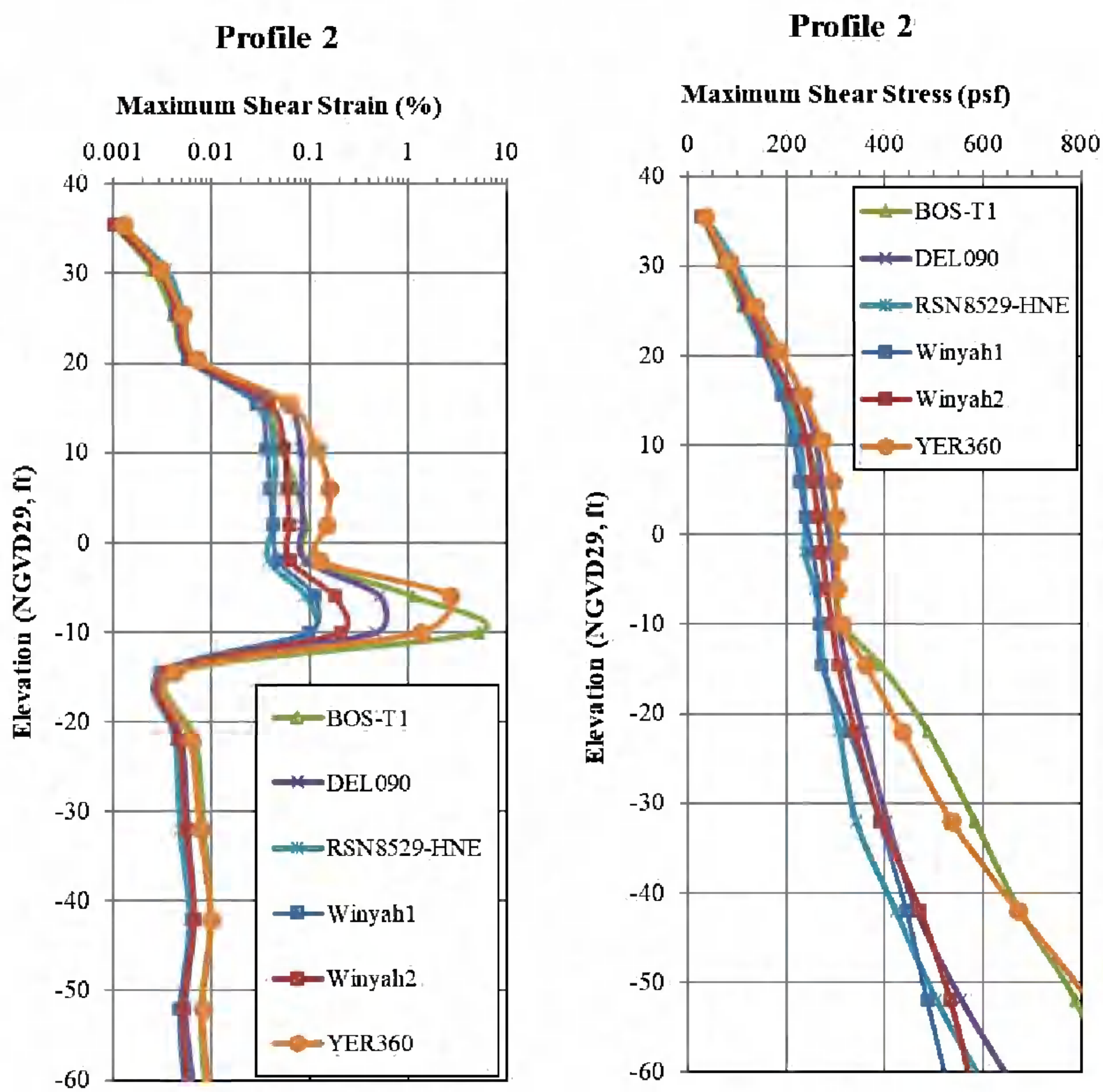


Figure 9b. Site Response Analysis Results for Profile 2

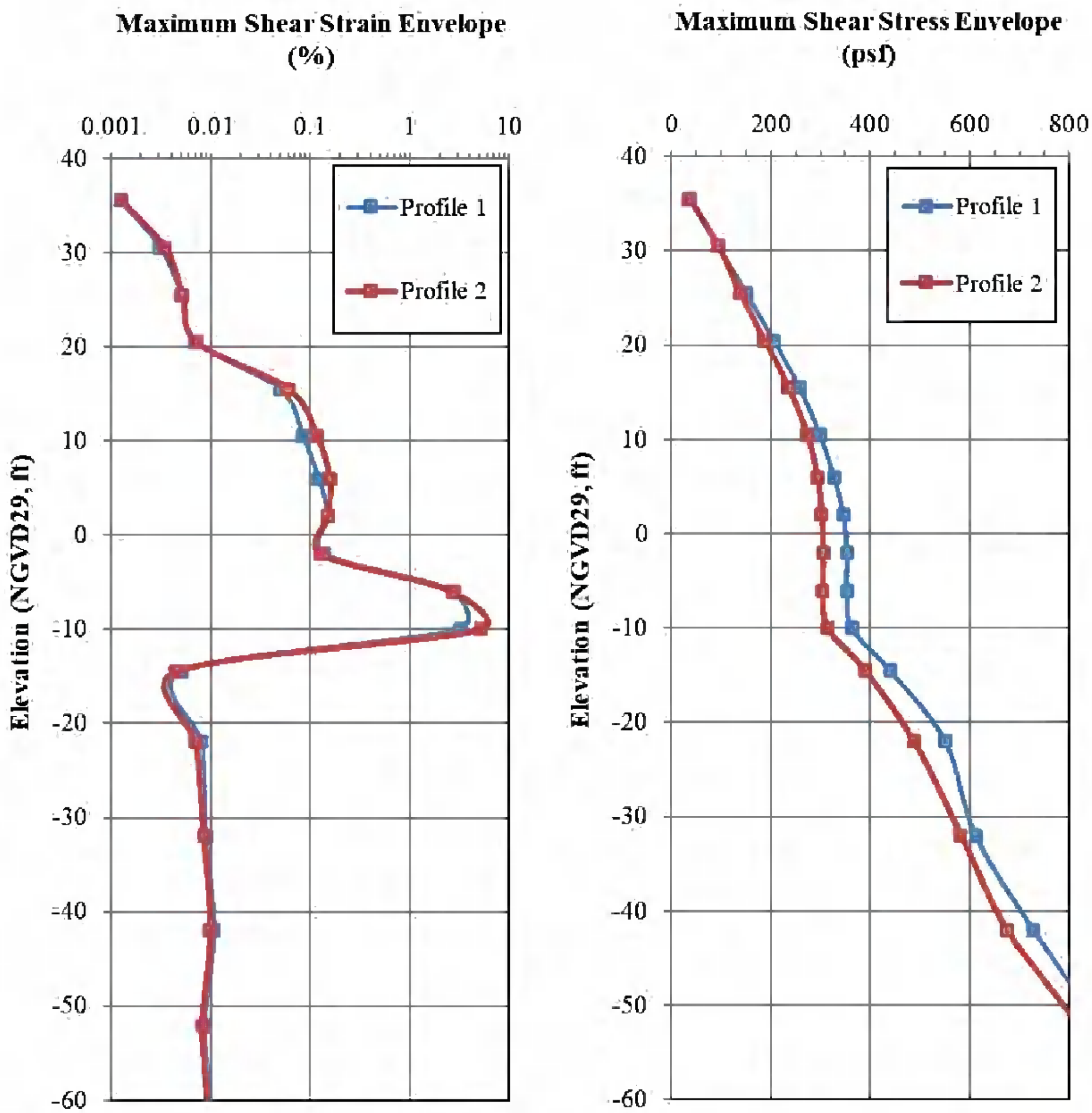


Figure 10. Maximum Shear Strain and Shear Stress Envelopes for Each Profile

Appendix 1

Selected Time Histories

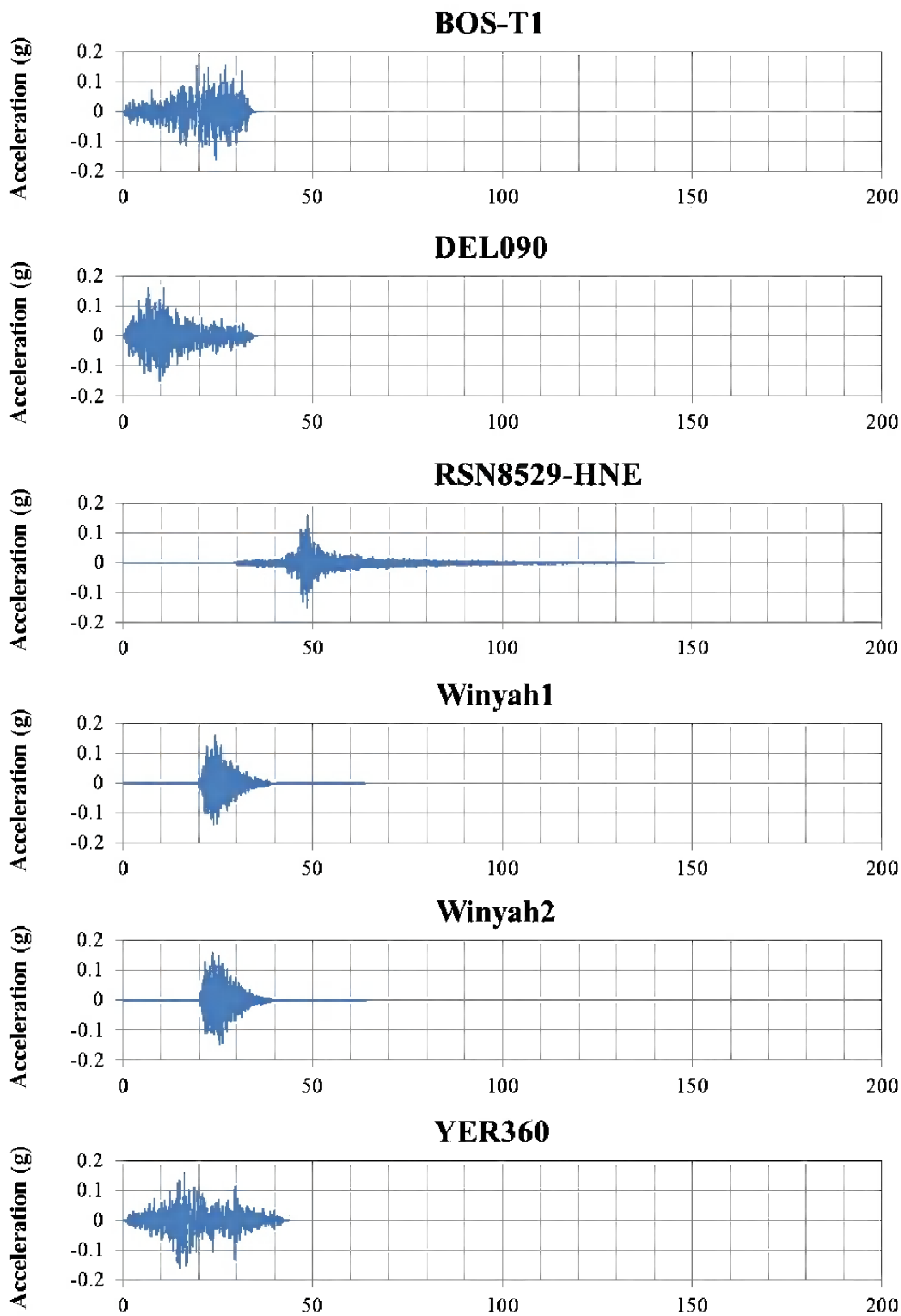


Figure 1-1. Acceleration Time Histories of Selected Earthquake Motions Scaled to a PGA of 0.15g.

Appendix 2

Shear Modulus Reduction and Damping Curve Selection

As indicated in the package, Geosyntec developed region-specific shear modulus reduction and damping curves based on the procedures presented in SCDOT GDM (2010). Figures 2-1 and 2-5 show the procedures. An example calculation following these procedures is presented as follows.

Shear Modulus Reduction Curve for the foundation soil in Profile 2

(see Figure 2-1 for description on each step; see Figure 2-2 for the profile)

Step 1 – age of the soil layer: Pleistocene deposit.

Step 2 – soil type: sandy soils with PI=0; groundwater table @ 15 ft bgs.

Step 3 – calculate σ_m'@ mid-depth of the layer (35 ft bgs)

$\sigma_v' = \gamma H - \gamma_w H_w = 125\times35 - 62.4\times20 = 3127$ psf

$\sigma_m' = \sigma_v' (1+2K'_o)/3 = 3127\times(1+2\times0.47)/3 = 2022.3$ psf

$(K_o' = 1 - \sin\phi' = 1 - \sin(32) = 0.47)$

Step 4 – σ_m' for the upper and lower native soils are within ±50% σ_m' value calculated above. The modulus reduction curve developed here can be used for the entire upper and lower native soils in Profile 2.

Step 5 – select the parameters α, γ_{r1}, k from Figure 2-4.

$\gamma_{r1} = 0.018\%$, $\alpha = 1.00$, $k = 0.454$

Step 6 – compute the reference strain using SCDOT GDM Equation 7-135 (see Figure 2-3 for the equation).

$\gamma_r = \gamma_{r1}\ (\sigma_m'/P_a)^k = 0.018\times(2022.3/2089)^{0.454} = 0.0177\%$

Step 7 – compute shear modulus reduction curve using SCDOT GDM Equation 7-134 (see Figure 2-3 for the equation)

$$\frac{G}{G_{max}} = \frac{1}{1+(\frac{\gamma}{\gamma_r})^{\alpha}}$$

If $\gamma = 0.001\%$, $G/G_{max} = 1/[1+(0.001/0.0178)] = 0.947$

If $\gamma = 0.01\%$, $G/G_{max} = 1/[1+(0.01/0.0178)] = 0.639$

If $\gamma = 0.1\%$, $G/G_{max} = 1/[1+(0.1/0.0178)] = 0.151$

Damping Curve for the upper native soil in Profile 2

(see Figure 2-5 for description on each step; see Figure 2-2 for the profile)

Steps 1 through 4 are the same as those for modulus reduction curve development.

Step 5 – select small-strain material damping @ $\sigma_m' = 1$ atm, D_{min1} from Figure 2-6.

$D_{min1} = 0.59\%$

Step 6 – compute the small strain material damping, D_{min}, using SCDOT GDM Equation 7-137 (see Figure 2-7 for the equation).

$D_{min} = D_{min1}\ (\sigma_m'/P_a)^{-0.5k} = 0.59\times(2022.1/2089)^{-0.5\times0.454} = 0.594\%$

Step 7-9 – instead of taking Steps 7 through 9, use SCDOT GDM Equation 7-138 to compute damping ratio curve (D).

$D = 12.2\ (G/G_{max})^2 - 34.2\ (G/G_{max}) + 22.0 + D_{min}$

If $\gamma = 0.001\%$, $D = 12.2\times(0.947)^2 - 34.2\times(0.947) + 22.0 + 0.594 = 1.15\%$

If $\gamma = 0.01\%$, $D = 12.2\times(0.640)^2 - 34.2\times(0.640) + 22.0 + 0.594 = 5.70\%$

If $\gamma = 0.1\%$, $D = 12.2\times(0.151)^2 - 34.2\times(0.151) + 22.0 + 0.594 = 17.71\%$

Shear Modulus Reduction and Damping Curves for Chicora / Williamsburg Formation

Figure 2-8 presents shear modulus reduction and damping curves used for Pacific Engineering's site response analyses of the Ammonia tank building located at the WGS.

Table 7-30, Procedure for Computing G/G_{max}

Step	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Develop soil profiles based on geologic units, soil types, average PI, and soil density. Subdivide major geologic units to reflect significant changes in PI and soil density. Identify design ground water table based on seasonal fluctuations and artesian pressures.
3	Calculate the average σ'_m and determine the corresponding ±50% range of σ'_m for each major geologic unit using Equation 7-136.
4	Calculate σ'_m for each layer within each major geologic unit. If the values for σ'_m of each layer are within a geologic unit's ±50% range of σ'_m (Step 3) then assign the average σ'_m for the major geologic unit (Step 3) to all layers within it. If the σ'_m of each layer within a geologic unit is not within the ±50% range of σ'_m for the major geologic unit, then the geologic unit needs to be "subdivided" and more than one average σ'_m needs to be used, provided the σ'_m remain within the ±50% range of σ'_m for the "subdivided" geologic unit.
5	Select the appropriate values for each layer of cyclic reference strain, γ_{cr1}, at 1 tsf (1 atm), curvature coefficient, α, and k exponent from Table 7-29. These values may be selected by rounding to the nearest PI value in the table or by interpolating between listed PI values in the table.
6	Compute the cyclic reference strain, γ_{cr}, based on Equation 7-135 for each geologic unit (or "subdivided" geologic unit) that has a corresponding average σ'_m.
7	Compute the design shear modulus reduction curves (G/G_{max}) for each layer by substituting cyclic reference strain, γ_{cr}, and curvature coefficient, α, for each layer using Equation 7-134. Tabulate values of normalized shear modulus, G/G_{max} with corresponding cyclic shear strain, γ_c, for use in a site-specific response analysis.

Figure 2-1. Procedure for Development of Region-specific Modulus Reduction Curve (SCDOT, 2019)

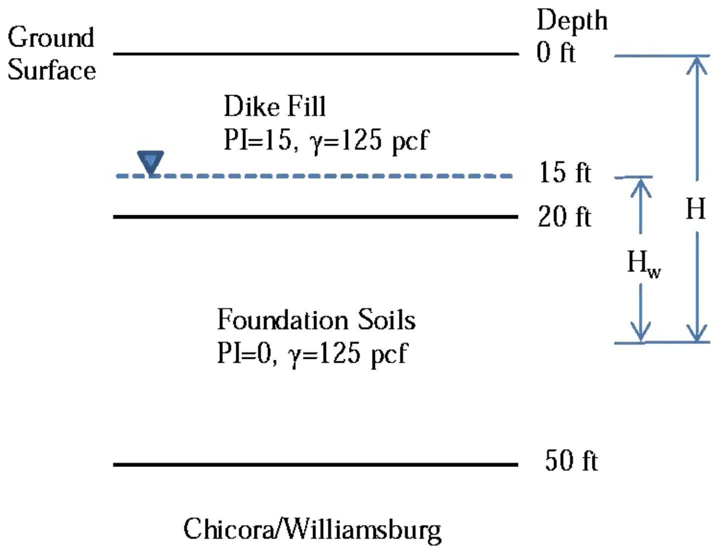


Figure 2-2. Profile 2 for the Example Calculations

$$G/G_{max} = \frac{1}{1+\left(\frac{\gamma_c}{\gamma_{cr}}\right)^{\alpha}}$$ Equation 7-134

Where,

α = Curvature coefficient

γ_c = Cyclic shear strain

γ_{cr} = Cyclic reference shear strain

$$\gamma_{cr} = \gamma_{cr1} * \left(\frac{\sigma'_m}{P_a}\right)^{k}$$ Equation 7-135

$$\sigma'_m = \sigma'_v * \left(\frac{1+2*K_o}{3}\right)$$ Equation 7-136

Where,

σ'_v = Vertical effective pressure, kPa

K_o = At-rest earth pressure coefficient

Figure 2-3. Equations Needed for Shear Modulus Reduction Curve Development (SCDOT, 2019)

Table 7-29, Recommended Values γ_{cr1}, α, and k for SC Soils (Andrus, et al. (2003))

Geologic Age and Location of Deposits [1]	Variable	Soil Plasticity Index, PI (%) 0	15	30	50	100	150
Holocene	γ_{cr1} (%)	0.073	0.114	0.156	0.211	0.350	0.488
	α	0.95	0.96	0.97	0.98	1.01	1.04 [2]
	k	0.385	0.202	0.106	0.045	0.005	0.001 [2]
Pleistocene (Wando)	γ_{cr1} (%)	0.018	0.032	0.047	0.067	0.117	0.166
	α	1.00	1.02	1.04	1.06	1.13	1.19
	k	0.454	0.402	0.355	0.301	0.199	0.132
Tertiary Ashley Formation (Cooper Marl)	γ_{cr1} (%)	—	—	0.030 [2]	0.049	0.096 [2]	—
	α	—	—	1.10 [2]	1.15	1.28	—
	k	—	—	0.497 [2]	0.455	0.362 [2]	—
Tertiary (Stiff Upland Soils)	γ_{cr1} (%)	—	—	0.023	0.041 [2]	—	—
	α	—	—	1.00	1.00 [2]	—	—
	k	—	—	0.102	0.045 [2]	—	—
Tertiary (All soils at SRS except Stiff Upland Soils)	γ_{cr1} (%)	0.038	0.058	0.079	0.106	0.174 [2]	—
	α	1.00	1.00	1.00	1.00	1.00 [2]	—
	k	0.277	0.240	0.208	0.172	0.106 [2]	—
Tertiary (Tobacco Road, Snapp)	γ_{cr1} (%)	0.029	0.056	0.082	0.117	0.205 [1]	—
	α	1.00	1.00	1.00	1.00	1.00 [1]	—
	k	0.220	0.185	0.156	0.124	0.070 [1]	—
Tertiary (Soft Upland Soils, Dry Branch, Santee, Warley Hill, Congaree)	γ_{cr1} (%)	0.047	0.059	0.071	0.086	0.125 [1]	—
	α	1.00	1.00	1.00	1.00	1.00 [1]	—
	k	0.313	0.299	0.285	0.268	0.229 [1]	—
Residual Soil and Saprolite	γ_{cr1} (%)	0.040	0.066	0.093 [1]	0.129 [1]	—	—
	α	0.72	0.80	0.89	1.01 [1]	—	—
	k	0.202	0.141	0.099	0.061 [2]	—	—

[1] SRS = Savannah River Site
[2] Tentative Values – Andrus et al. (2003)

Figure 2-4. Recommended Parameters for South Carolina Soils (SCDOT, 2019)

Table 7-32, Procedure for Computing Damping Ratio

Step	Procedure Description
1	Perform a geotechnical subsurface exploration and identify subsurface soil geologic units, approximate age, and formation.
2	Develop soil profiles based on geologic units, soil types, average PI, and soil density. Subdivide major geologic units to reflect significant changes in PI and soil density. Identify design ground water table based on seasonal fluctuations and artesian pressures.
3	Calculate the average σ'_m and determine the corresponding ±50% range of σ'_m for each major geologic unit using Equation 7-136.
4	Calculate σ'_m for each layer within each major geologic unit. If the values for σ'_m of each layer are within a geologic unit's ±50% range of σ'_m (Step 3) then assign the average σ'_m for the major geologic unit (Step 3) to all layers within it. If the σ'_m of each layer within a geologic unit is not within the ±50% range of σ'_m for the major geologic unit, then the geologic unit needs to be "subdivided" and more than one average σ'_m needs to be used, provided the σ'_m remain within the ±50% range of σ'_m for the "subdivided" geologic unit.
5	Select appropriate small-strain material Damping @ σ'_m = 1 atm, λ_{min1}, from Table 7-31 for each layer within a geologic unit.
6	Compute the small-strain material Damping, λ_{min}, for each layer within a geologic unit using Equation 7-137.
7	Select the appropriate values for each layer of cyclic reference strain, γ_{cr1}, @ σ'_m = 1atm , curvature coefficient, α, and k exponent from Table 7-29. These values may be selected by rounding to the nearest PI value in the table or by interpolating between listed PI values in the table.
8	Compute the cyclic reference strain, γ_{cr}, based on Equation 7-135 for each geologic unit that has a corresponding average σ'_m.
9	Compute the design equivalent viscous damping ratio curves (λ) for each layer by substituting cyclic reference strain, γ_{cr}, and curvature coefficient, α, and small-strain material Damping, λ_{min}, for each layer using Equation 7-139. Tabulate values of Soil Damping Ratio, λ, with corresponding cyclic shear strain, γ_c, for use in a site-specific site response analysis.

Figure 2-5. Procedure for Development of Region-Specific Damping Curve (SCDOT, 2019)

Table 7-31, Recommended Value λ_{min1} (%) for SC Soils (Andrus, et al. (2003))

Geologic Age and Location of Deposits	Soil Plasticity Index, PI (%)					
	0	15	30	50	100	150
Holocene	1.09	1.29	1.50	1.78	2.48	3.18 [1]
Pleistocene (Wando)	0.59	0.66	0.73	0.83	1.08	1.32
Tertiary Ashley Formation (Cooper Marl)	—	—	1.14 [1]	1.52 [1]	2.49 [1]	—
Tertiary (Stiff Upland Soils)	—	—	0.98	1.42 [1]	—	—
Tertiary (All soils at SRS except Stiff Upland Soils)	0.68	0.94	1.19	1.53	2.37 [1]	—
Tertiary (Tobacco Road, Snapp)	0.68	0.94	1.19	1.53	2.37 [1]	—
Tertiary (Soft Upland Soils, Dry Branch, Santee, Warley Hill, Congaree)	0.68	0.94	1.19	1.53	2.37 [1]	—
Residual Soil and Saprolite	0.56 [1]	0.85 [1]	1.14 [1]	1.52 [1]	—	—

[1] Tentative Values – Andrus, et al. (2003)

Figure 2-6. Recommended D_{min1} for South Carolina Soils (Table 12-17 of SCDOT, 2019)

$$\lambda_{min} = \lambda_{min1} * \left(\frac{\sigma'_m}{P_a}\right)^{-0.5*k}$$ **Equation 7-137**

Where λ_{min1} is the small-strain damping at σ'_m of 1 tsf (1 atm). The mean confining pressure,

Equation 7-137 represents a best-fit equation (UTA Correlation) of the observed relationship of $(\lambda - \lambda_{min})$ vs. (G/G_{max}) indicated below:

Equation 7-138

$$\lambda - \lambda_{min} = 12.2 * \left(\frac{G}{G_{max}}\right)^2 - 34.2 * \left(\frac{G}{G_{max}}\right) + 22.0$$

If we substitute Equation 7-134 into Equation 7-138 and solve for the damping ratio, λ, the Equivalent Viscous Damping Ratio curves can be generated using the following equation.

Equation 7-139

$$\lambda = \lambda_{min} + 12.2 * \left[\frac{1}{1 + \left(\frac{\gamma_c}{\gamma_{cr}}\right)^{a}}\right]^2 - 34.2 * \left[\frac{1}{1 + \left(\frac{\gamma_c}{\gamma_{cr}}\right)^{a}}\right] + 22.0$$

Where values of reference strain, γ_{cr}, are computed using Equation 7-135.

Figure 2-7. Equations Needed for Damping Curve Development (SCDOT, 2019)

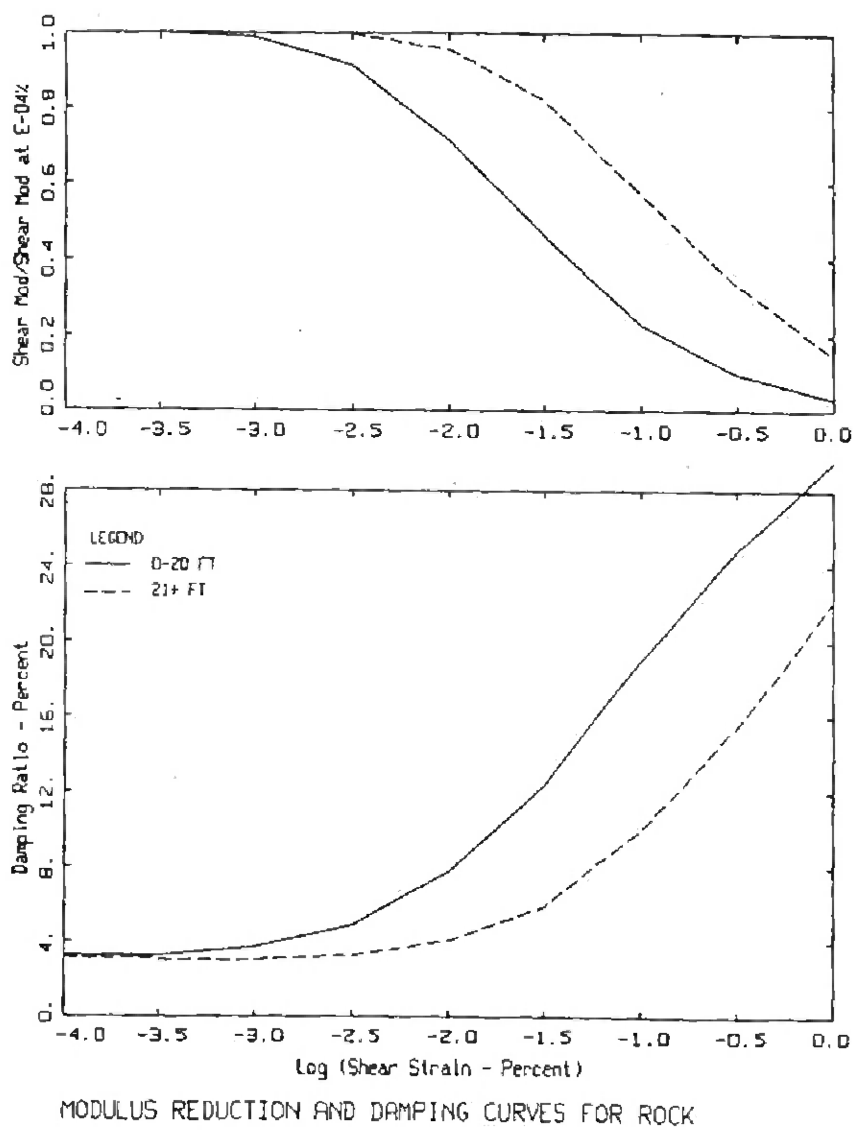


Figure 5b. Generic G/Gmax and hysteretic damping curves for soft rock (Silva et al., 1997).

Figure 2-8. Shear Modulus Reduction and Damping Curves for Chicora/Williamsburg Formation (S&ME, 2001)

Appendix 3

Shear Wave Velocity Profile Selection

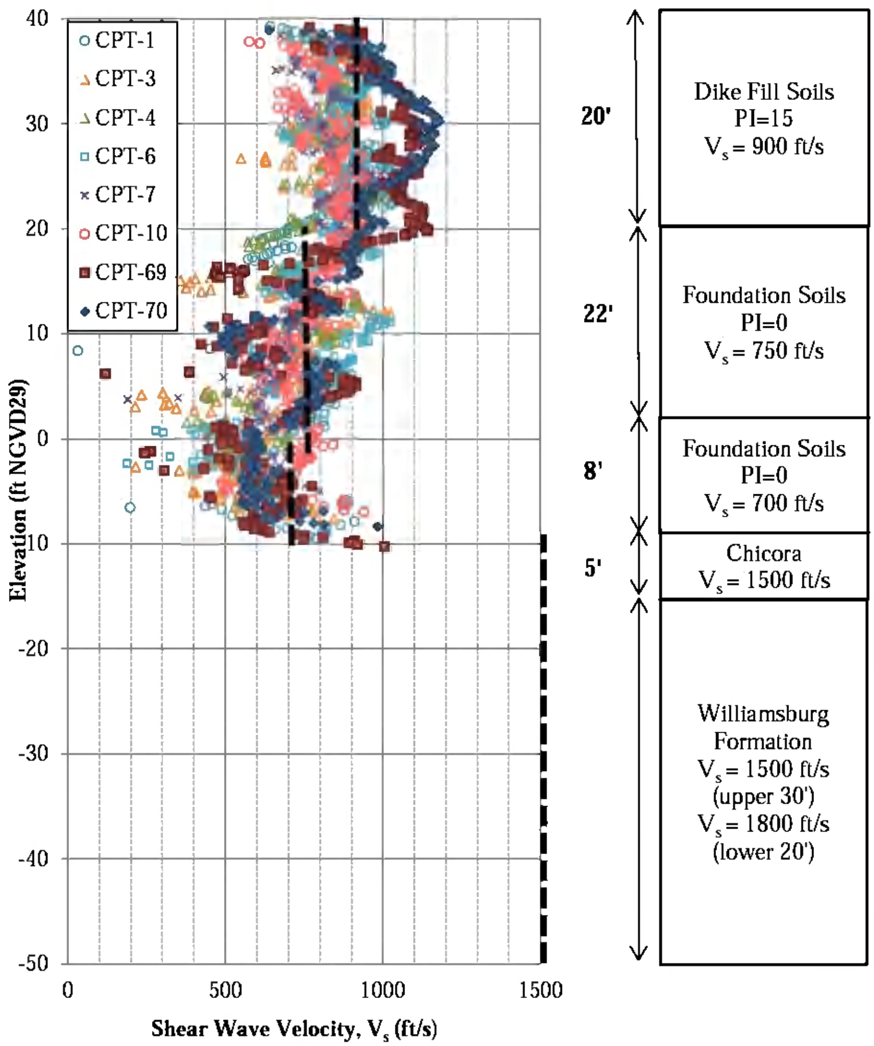


Figure 3-1a. Selected V_s Profile for the Section 1 Dike Model (Profile 1)

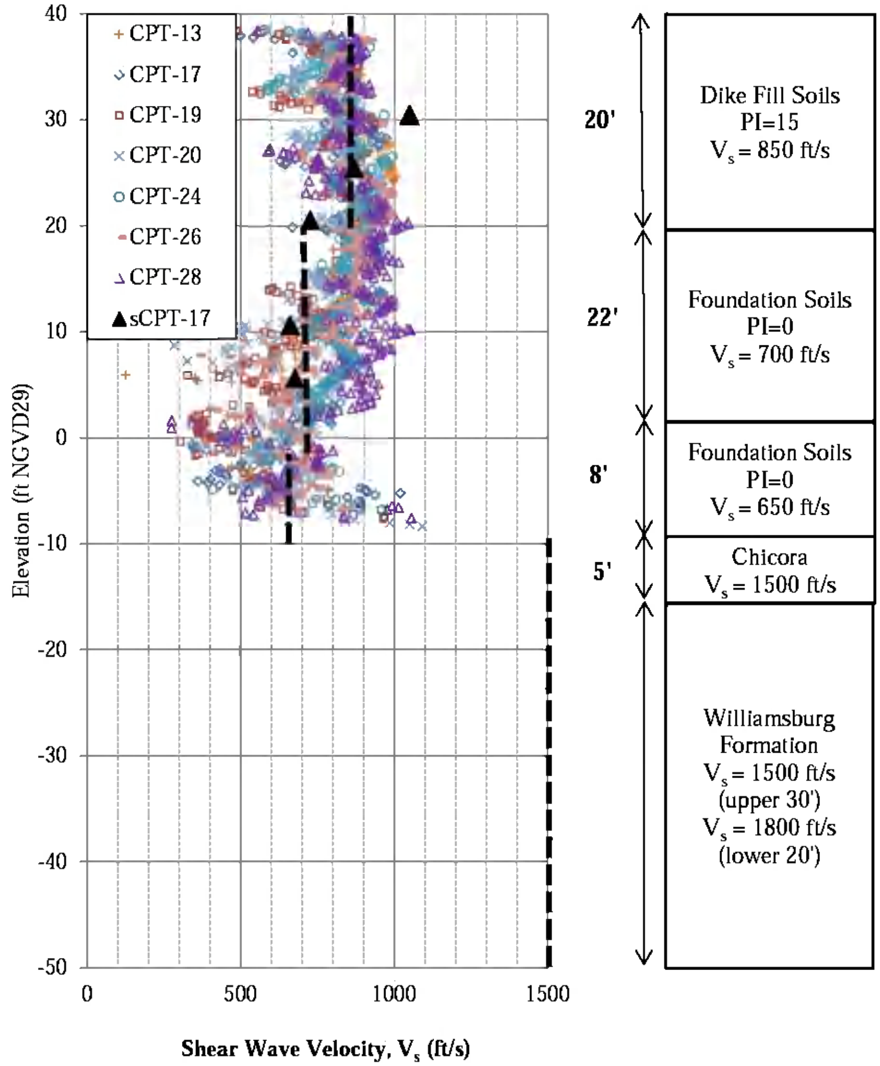


Figure 3-1b. Selected V_s Profile for the Section 2 Dike Model (Profile 2)

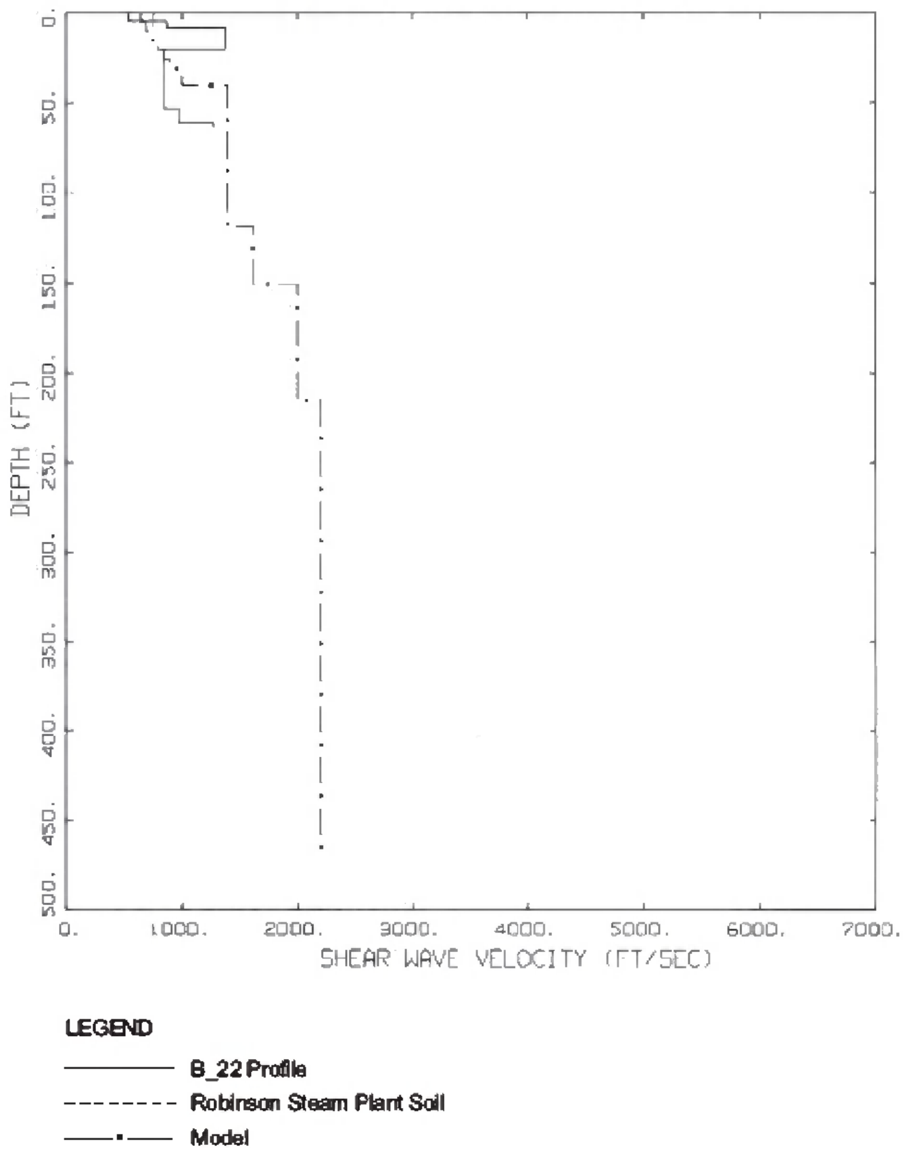


Figure 3-9. Base case shear-wave velocity profile for the Myrtle Beach site response category along with available profiles.

Figure 3-2. Reference V_s Profile for Chicora/Williamsburg Formation (URS, 2001)

Appendix 4

Calculated Acceleration Profiles

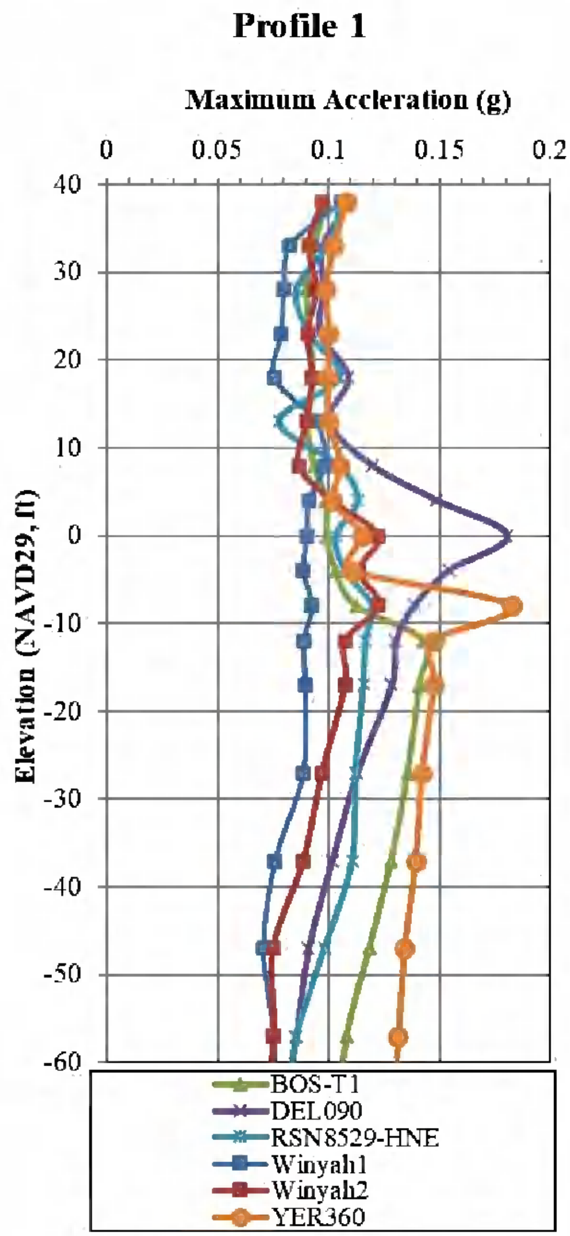


Figure 4-1. Calculated Maximum Acceleration for Profile 1

Note:

1. The input motions were applied as an outcrop motion with a PGA of 0.15g.

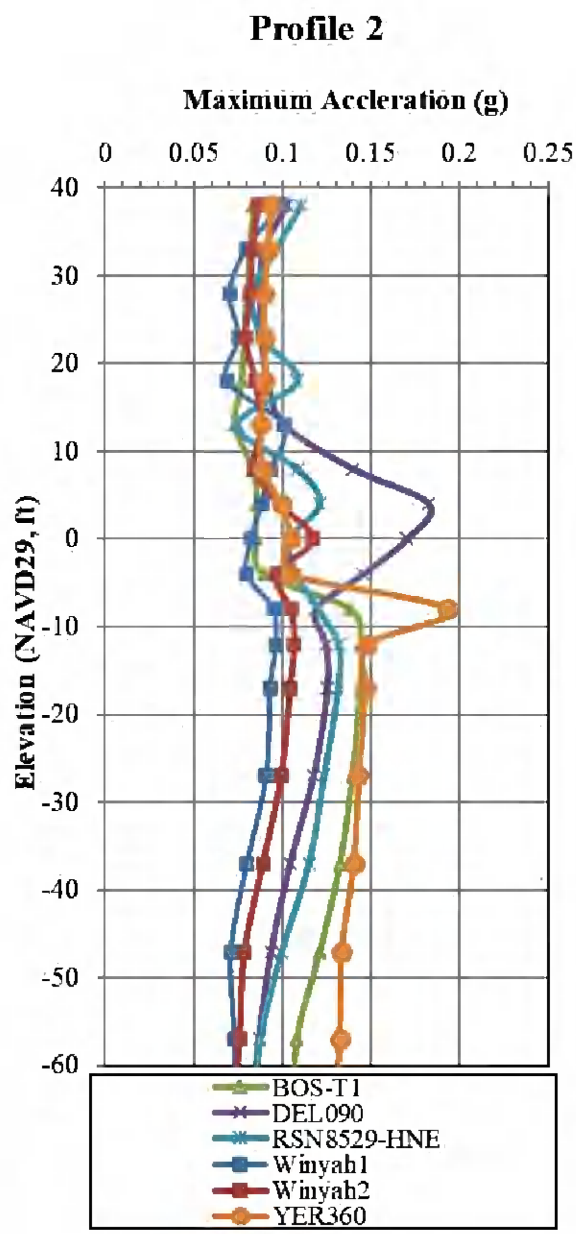


Figure 4-2. Calculated Maximum Acceleration for Profile 2

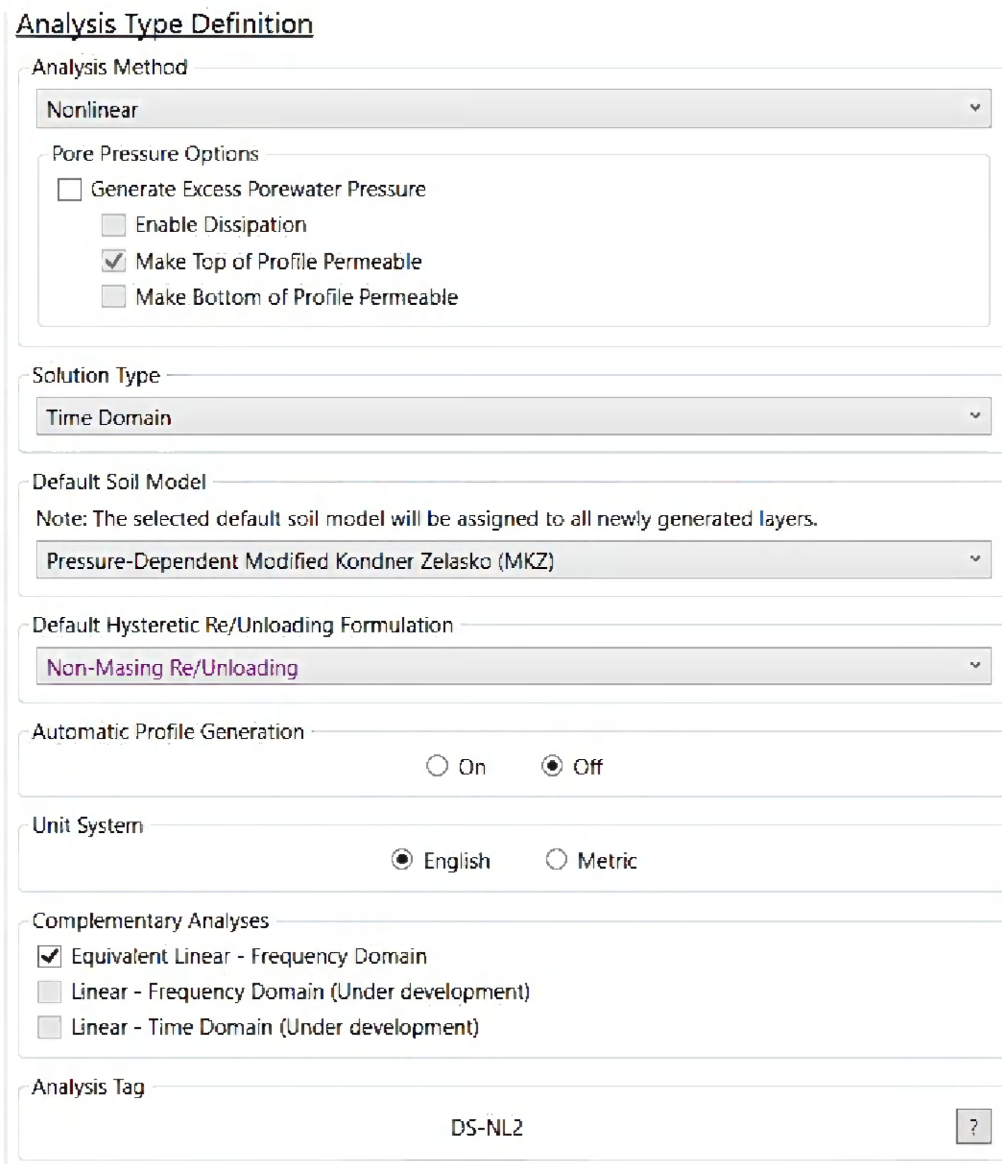
Note:

1. The input motions were applied as an outcrop motion with a PGA of 0.15g

.

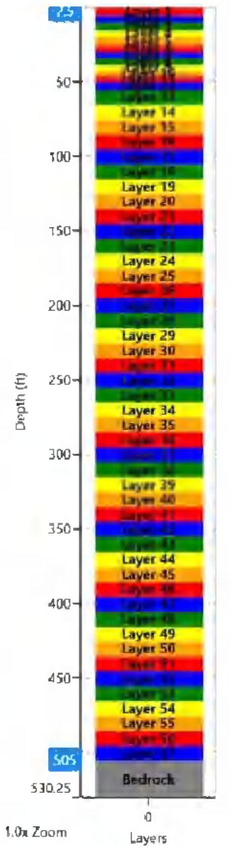
Appendix 5

DEEPSOIL® Input


Analysis Type Definition
Analysis Method
Nonlinear
Pore Pressure Options
Generate Excess Porewater Pressure
Enable Dissipation
Make Top of Profile Permeable
Make Bottom of Profile Permeable
Solution Type
Time Domain
Default Soil Model
Note: The selected default soil model will be assigned to all newly generated layers.
Pressure-Dependent Modified Kondner Zelasko (MKZ)
Default Hysteretic Re/Unloading Formulation
Non-Masing Re/Unloading
Automatic Profile Generation
On
Off
Unit System
English
Metric
Complementary Analyses
Equivalent Linear - Frequency Domain
Linear - Frequency Domain (Under development)
Linear - Time Domain (Under development)
Analysis Tag
DS-NL2
?

Soil Profile Definition

Soil Profile Plot



Soil Profile Metrics

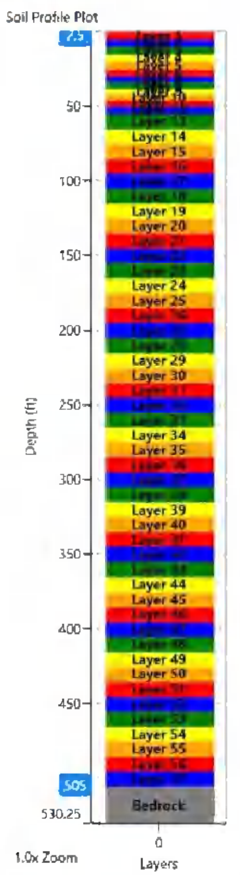
Total Profile Depth: 505
Profile Natural Frequency (Hz): 0.8573
Profile Natural Period (sec): 1.166

Layer Properties | Advanced Table View

Layer Number	Layer Name	Thickness (ft)	Unit Weight (pcf)	Shear Wave Velocity (ft/s)	Shear Strength (psf)	Soil Model	Dmin (%)	Ref. Strain (%)	Reference Stress (MPa)	β	s	b	d
1	Dike	5	125	900		MKZ	1.62242631908	0.1104	0.18	1.515	0.96	0	0
2	Dike	5	125	900		MKZ	1.45250494134	0.1392	0.18	1.53	0.96	0	0
3	Dike	5	125	900		MKZ	1.37964656664	0.156	0.18	1.545	0.96	0	0
4	Dike	5	125	900		MKZ	1.34373080332	0.1628	0.18	1.53	0.96	0	0
5	Foundation Soil	5	125	750		MKZ	0.61868612256	0.024	0.18	1.545	1.005	0	0
6	Foundation Soil	5	125	750		MKZ	0.60175464979	0.0254	0.18	1.545	1.005	0	0
7	Foundation Soil	4	125	750		MKZ	0.58850946052	0.0248	0.18	1.44	1.005	0	0
8	Foundation Soil	4	125	750		MKZ	0.57793595127	0.0268	0.18	1.5	1.005	0	0
9	Foundation Soil	4	125	750		MKZ	0.56831113023	0.028	0.18	1.515	1.005	0	0
10	Foundation Soil	4	125	700		MKZ	0.55954549127	0.0292	0.18	1.53	1.005	0	0
11	Foundation Soil	4	125	700		MKZ	0.55145287925	0.0292	0.18	1.485	1.005	0	0
12	Chicora	5	130	1500		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
13	Williamsburg Fc	10	105	1500		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
14	Williamsburg Fc	10	105	1500		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
15	Williamsburg Fc	10	105	1500		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
16	Williamsburg Fc	10	105	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
17	Williamsburg Fc	10	105	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
18	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
19	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
20	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
21	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
22	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
23	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
24	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
25	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
26	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
27	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
28	Williamsburg Fc	10	125	2000		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
29	Williamsburg Fc	10	125	2000		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0

☑ Water table at top of layer: 4 | Add Layer(s) | Remove Layer(s)

Soil Profile Definition



Soil Profile Metrics

Total Profile Depth 505

Profile Natural Frequency (Hz): 0.8573

Profile Natural Period (sec): 1.166

Layer Properties | Advanced Table View

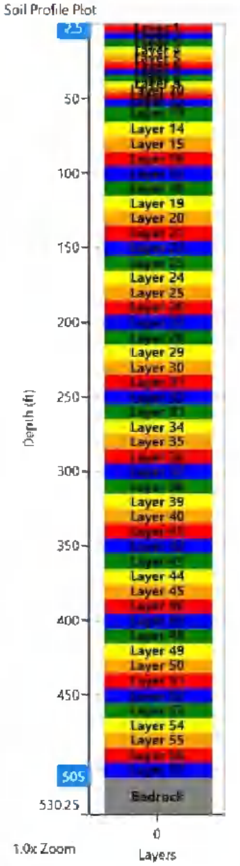
Soil Profile Plot

c	b	d	θ1	θ2	θ3	θ4	θ5	A	γ1	Reduction Factor Formulation	P1	P2	P3
96	0	0								MRDF-UIUC	0.632	0.236	3.25
96	0	0								MRDF-UIUC	0.63	0.232	3.25
96	0	0								MRDF-UIUC	0.63	0.232	3.25
96	0	0								MRDF-UIUC	0.626	0.226	3.25
005	0	0								MRDF-UIUC	0.618	0.26	3.25
005	0	0								MRDF-UIUC	0.618	0.26	3.25
005	0	0								MRDF-UIUC	0.618	0.26	3.25
005	0	0								MRDF-UIUC	0.618	0.26	3.25
005	0	0								MRDF-UIUC	0.618	0.26	3.25
005	0	0								MRDF-UIUC	0.618	0.26	3.25
005	0	0								MRDF-UIUC	0.618	0.26	3.25
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2

☑ Water table at top of layer: 4 | Add Layer(s) | Remove Layer(s)

Soil Profile Definition

Soil Profile Plot



Soil Profile Metrics

Total Profile Depth 505
Profile Natural Frequency (Hz): 0.8573
Profile Natural Period (sec): 1.166

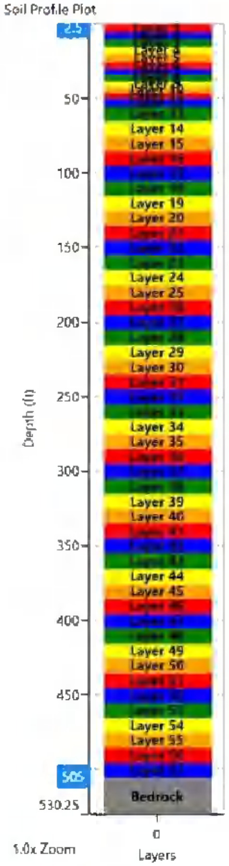
Layer Properties | Advanced Table View

Layer Number	Layer Name	Thickness (ft)	Unit Weight (pcf)	Shear Wave Velocity (ft/s)	Shear Strength (psf)	Soil Model	Dmin (%)	Ref. Strain (%)	Reference Stress (MPa)	β	s	b	d
29	Williamsburg Fc	10	125	2000		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
30	Williamsburg Fc	10	125	2000		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
31	Williamsburg Fc	10	125	2000		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
32	Williamsburg Fc	10	125	2000		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
33	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
34	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
35	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
36	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
37	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
38	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
39	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
40	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
41	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
42	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
43	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
44	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
45	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
46	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
47	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
48	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
49	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
50	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
51	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
52	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
53	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
54	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
55	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
56	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0
57	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96	0	0

☑ Water table at top of layer: 4 | Add Layer(s) | Remove Layer(s)

Soil Profile Definition

Soil Profile Plot



1.0x Zoom

Soil Profile Plot

Layer Properties | Advanced Table View

s	b	d	Θ1	Θ2	Θ3	Θ4	Θ5	A	γ1	Reduction Factor Formulation	P1	P2	P3
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2
96	0	0								MRDF-UIUC	0.676	0.254	2

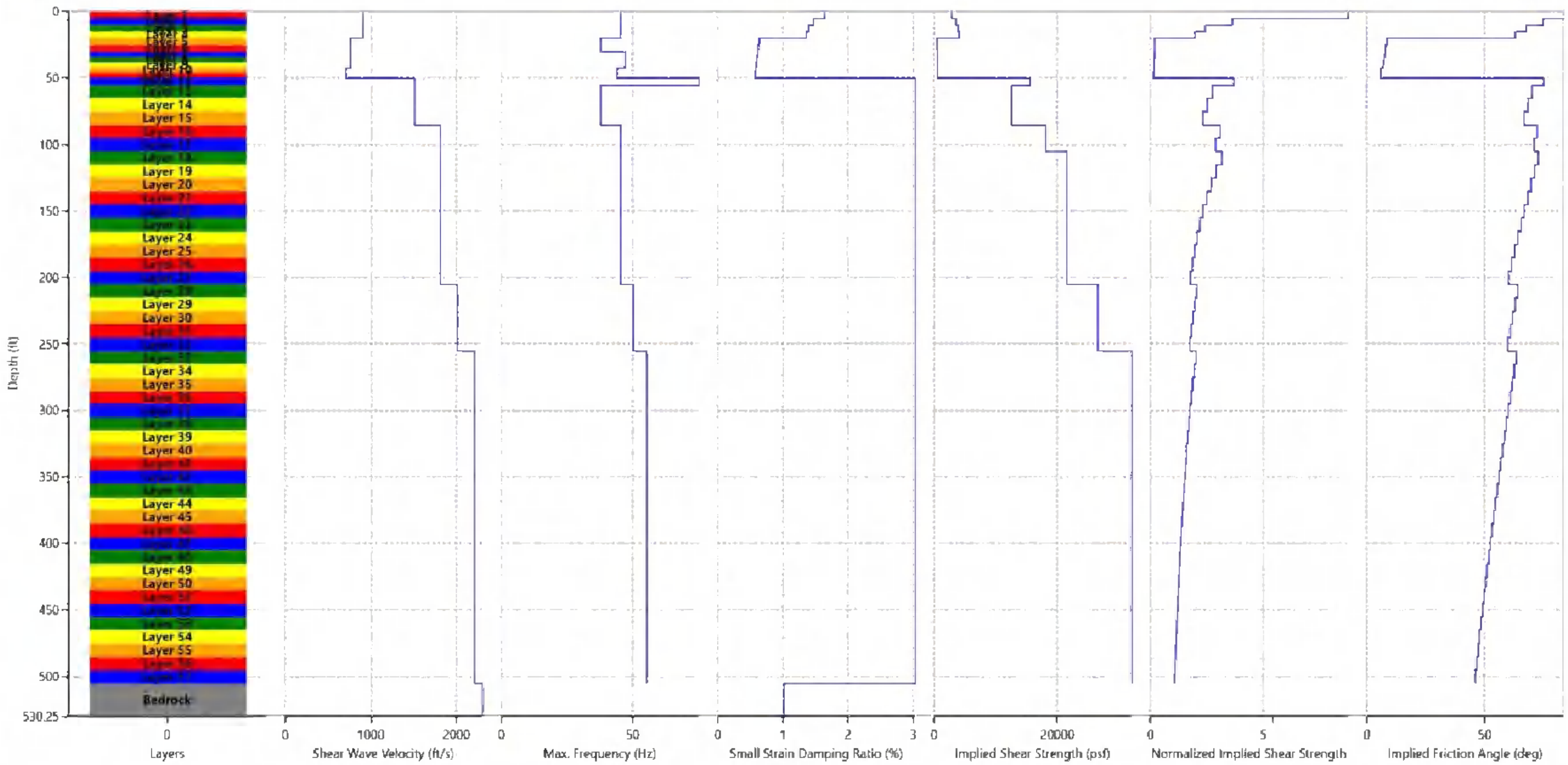
Soil Profile Metrics

Total Profile Depth	505
Profile Natural Frequency (Hz):	0.8573
Profile Natural Period (sec):	1.166

☑ Water table at top of layer: 4 | Add Layer(s) | Remove Layer(s)

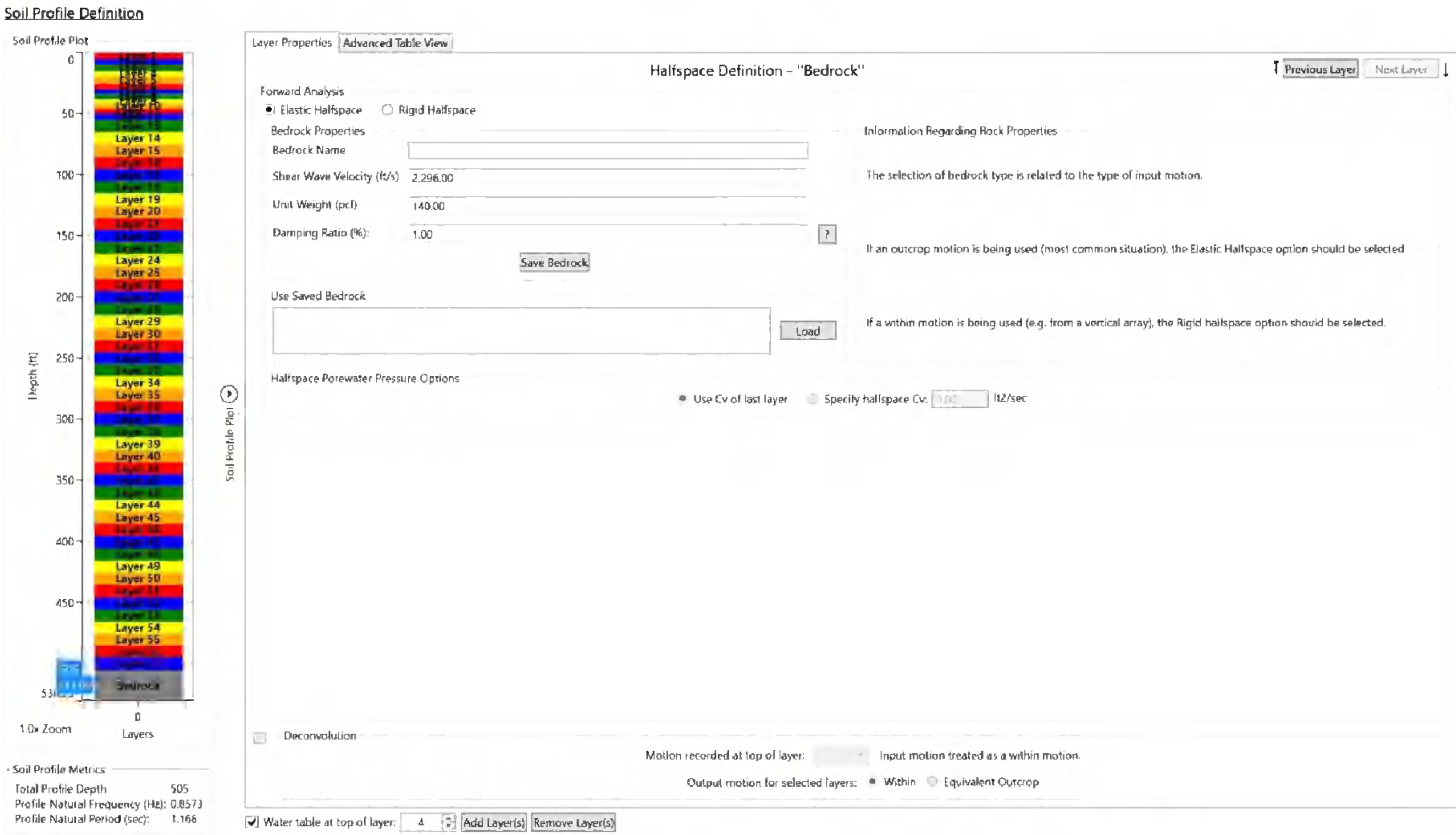
Soil Profile Definition

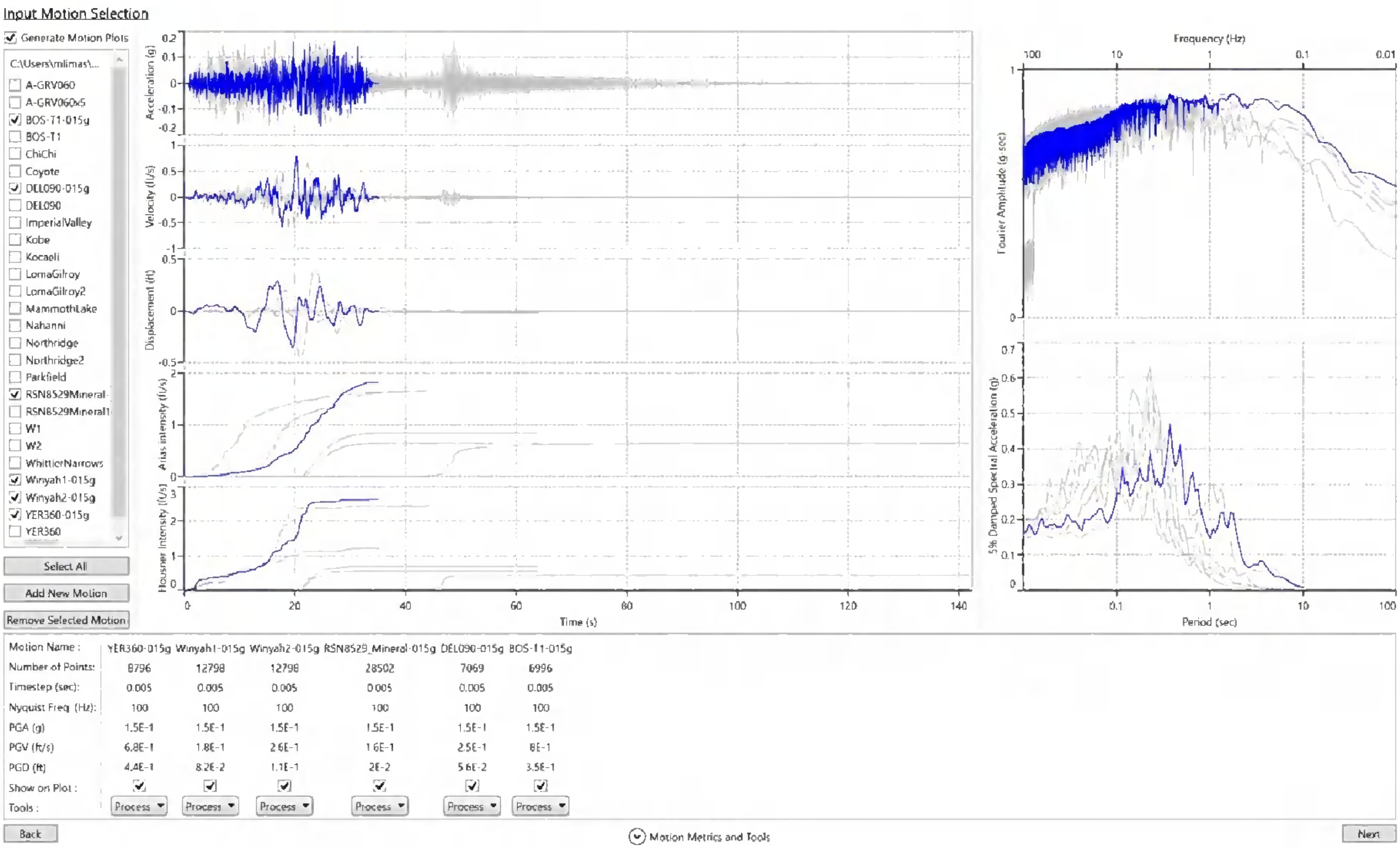
Soil Profile Plot

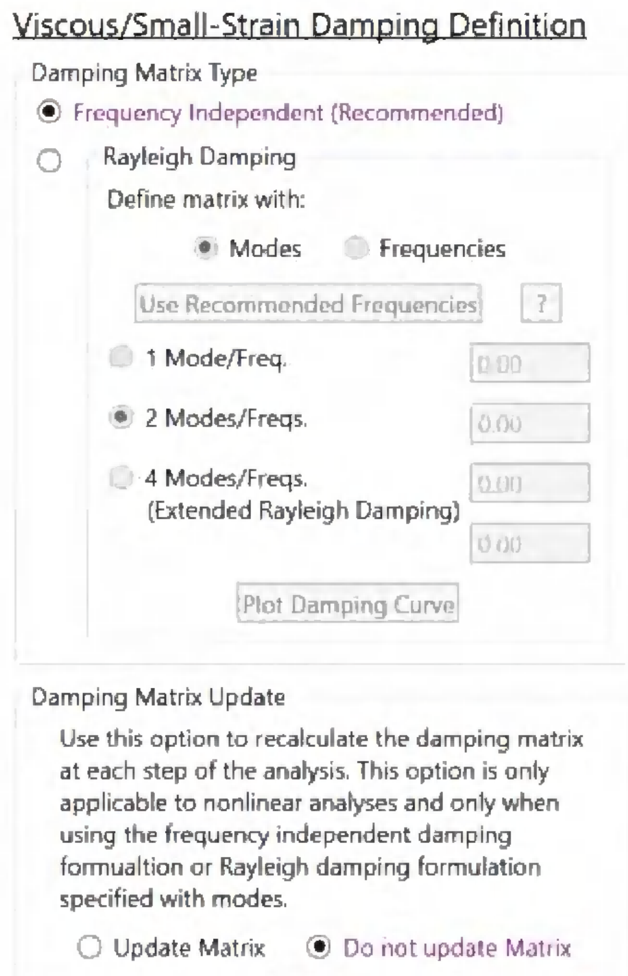


Soil Profile Metrics

Total Profile Depth	505
Profile Natural Frequency (Hz):	0.8573
Profile Natural Period (sec):	1.166


Soil Profile Definition
Soil Profile Plot
Layer Properties
Advanced Table View
Halfspace Definition - "Bedrock"
Previous Layer
Next Layer
Forward Analysis
Elastic Halfspace
Rigid Halfspace
Bedrock Properties
Bedrock Name
Shear Wave Velocity (ft/s)
2,296.00
Unit Weight (pcf)
140.00
Damping Ratio (%):
1.00
Save Bedrock
Use Saved Bedrock
Load
Information Regarding Rock Properties
The selection of bedrock type is related to the type of input motion.
If an outcrop motion is being used (most common situation), the Elastic Halfspace option should be selected
If a within motion is being used (e.g. from a vertical array), the Rigid halfspace option should be selected.
Halfspace Porewater Pressure Options
Use Cv of last layer
Specify halfspace Cv:
ft2/sec
Deconvolution
Motion recorded at top of layer:
Input motion treated as a within motion.
Output motion for selected layers:
Within
Equivalent Outcrop
Depth (ft)
Layers
1.0x Zoom
Soil Profile Plot
Bedrock
Soil Profile Metrics
Total Profile Depth 505
Profile Natural Frequency (Hz): 0.8573
Profile Natural Period (sec): 1.166
Water table at top of layer:
4
Add Layer(s)
Remove Layer(s)


Input Motion Selection
Generate Motion Plots
C:\Users\mlimas\...
A-GRV060
A-GRV060x5
BOS-T1-015g
BOS-T1
ChiChi
Coyote
DEL090-015g
DEL090
ImperialValley
Kobe
Kocaeli
LomaGilroy
LomaGilroy2
MammothLake
Nahanni
Northridge
Northridge2
Parkfield
RSN8529Mineral-
RSN8529Mineral1
W1
W2
WhittierNarrows
Winyah1-015g
Winyah2-015g
YER360-015g
YER360
Select All
Add New Motion
Remove Selected Motion
Acceleration (g)
Velocity (ft/s)
Displacement (ft)
Arias Intensity (ft/s)
Housner Intensity (ft/s)
Time (s)
Frequency (Hz)
Fourier Amplitude (g-sec)
5% Damped Spectral Acceleration (g)
Period (sec)
Motion Name: YER360-015g Winyah1-015g Winyah2-015g RSN8529_Mineral-015g DEL090-015g BOS-T1-015g
Number of Points: 8796 12798 12798 28502 7069 6996
Timestep (sec): 0.005 0.005 0.005 0.005 0.005 0.005
Nyquist Freq. (Hz): 100 100 100 100 100 100
PGA (g) 1.5E-1 1.5E-1 1.5E-1 1.5E-1 1.5E-1 1.5E-1
PGV (ft/s) 6.8E-1 1.8E-1 2.6E-1 1.6E-1 2.5E-1 8E-1
PGD (ft) 4.4E-1 8.2E-2 1.1E-1 2E-2 5.6E-2 3.5E-1
Show on Plot:
Tools: Process Process Process Process Process Process
Back
Motion Metrics and Tools
Next


Viscous/Small-Strain Damping Definition
Damping Matrix Type
Frequency Independent (Recommended)
Rayleigh Damping
Define matrix with:
Modes
Frequencies
Use Recommended Frequencies
?
1 Mode/Freq.
0.00
2 Modes/Freqs.
0.00
4 Modes/Freqs.
(Extended Rayleigh Damping)
0.00
0.00
Plot Damping Curve
Damping Matrix Update
Use this option to recalculate the damping matrix at each step of the analysis. This option is only applicable to nonlinear analyses and only when using the frequency independent damping formualtion or Rayleigh damping formulation specified with modes.
Update Matrix
Do not update Matrix

<u>Analysis Control Definition</u>

Frequency Domain

Number of iterations: 15

Effective Shear Strain Definition

$$SSR = \frac{M-1}{10}$$

Effective Shear Strain Ratio (SSR): 0.65

Complex Shear Modulus Formulation

- (●) Frequency Independent (Recommended)

$$G^* = G(1+2i\xi)$$

- ( ) Frequency Dependent (Use with Caution)

$$G^* = G(1-2\xi^2+2i\xi\sqrt{1-\xi^2})$$

- ( ) Simplified

$$G^* = G(1-\xi^2+2i\xi)$$

Time Domain

Step Control

(●) Flexible ( ) Fixed

Maximum Strain Increment (%): 0.005

Number of Sub-increments:

Integration Scheme

- (●) Implicit: Newmark Beta Method (β=0.25, γ=0.5)
- ( ) Explicit: Heun's Method (P(EC)^nE)

Time-history Interpolation Method

- (●) Linear in time domain
- ( ) Zero-padded in frequency-domain

Output Settings

Layers

- ( ) Surface Only
- ( ) All Layers
- ( ) At Specific Depth
- (●) At Specific Layers

Profile 1

Layer #	Layer Name	Want Output
1	Dike	☑
2	Dike	☐
3	Dike	☐
4	Dike	☐
5	Foundation Soils	☑
6	Foundation Soils	☐
7	Foundation Soils	☐
8	Foundation Soils	☐
9	Foundation Soils	☐

Displacement Animation

☐ Output displacement animation. (Warning: Generating the displacement animation will slow down the speed of analysis!)

Bedrock

☐ Output deconvolution result at top of rock

ATTACHMENT 4
LIQUEFACTION POTENTIAL ANALYSIS

Written by:	C. Carlson	Date:	10/14/2021	Reviewed by:	O. Kutlu/W. Shin	Date:	10/14/2021
Client:	Santee Cooper	Project:	Winyah Generating Station	Project/ Proposal No.:	GC8100	Task No.:	03

LIQUEFACTION POTENTIAL ANALYSIS: SLURRY POND

INTRODUCTION

This liquefaction potential analysis calculation package (Liquefaction Package) was prepared to present the evaluation for soil liquefaction potential of the perimeter dike soils forming the Slurry Pond 3&4 (Slurry Pond) at Winyah Generating Station (WGS or Site). This calculation package is Attachment 4 to *2021 Periodic Safety Factor Assessment* (2021 Safety Factor Assessment Report) prepared by Geosyntec Consultants, Inc. (Geosyntec) to demonstrate compliance with the United States Environmental Protection Agency's (USEPA) Coal Combustion Residuals (CCR) Rule with respect to the periodic safety factor assessment criteria presented in 40 Code of Federal Regulations (CFR) 257.73(e). Ground motions and resulting shear stresses for the design seismic event are presented in Attachment 3 *Seismic Hazard Evaluation and Site Response Analysis: Slurry Pond* (Site Response Package) to the 2021 Safety Factor Assessment Report. The liquefaction potential of soils was evaluated using the results from soil borings and cone penetration test (CPT) soundings advanced through the Slurry Pond perimeter dike during Geosyntec's 2013 geotechnical subsurface investigations and a historical investigation performed in 1999 (PCRA, 1999). Details of these investigations are discussed in the 2016 Safety Factor Assessment Report (Geosyntec, 2016). The remainder of this Liquefaction Package presents: (i) methodology; (ii) analysis cases; (iii) input parameters; (iv) results; (v) conclusions; and (vi) references.

METHODOLOGY

Current state-of-practice procedures for evaluating the liquefaction potential of a soil were developed based on case histories of occurrences and non-occurrences of liquefaction due to past earthquakes. Occurrences (or non-occurrences) of liquefaction were determined by presence (or absence) of surface manifestations of liquefaction such as sand boils, ground cracking, slope movements, and/or flow failures. Surface manifestations were generally present if large excess pore pressures are generated during seismic loading and "liquefaction" is triggered. Therefore, if soils at a particular site are not expected to undergo triggering of liquefaction based on the state-of-practice or regulatory guidance, additional analyses, such as post-liquefaction slope stability or lateral spreading estimations, are not necessary for the anticipated seismic ground motions.

It was assumed that soils classified as Organic Peat, Silt, and Clay, or a combination of these materials, are typically not susceptible to liquefaction. Additionally, soils that exhibit "clay-like" behavior according to data collected during CPT soundings were also screened as not susceptible to liquefaction. "Clay-like" behavior was defined as a soil with a Soil Behavior Index (I_c) greater than 2.60. The interpretation of CPT soundings and the computation of I_c are discussed in the 2016 Safety Factor Assessment Report (Geosyntec, 2016) and reiterated below. If a zone of soil screened as not susceptible to liquefaction by the above criteria, the soil zone was assigned a factor of safety (FS) against liquefaction triggering of 2.0. The criteria recommended by Bray and Sancio (2006) were applied to evaluate the susceptibility of fine-grained soils to cyclic softening. Most of the tested samples were found to be "Not Susceptible" to cyclic softening by these criteria.

The liquefaction analysis described below was performed based on the simplified procedure recommended by Seed and Idriss (1971) and later updated by Boulanger and Idriss (2014), unless otherwise indicated. Analyses were performed on both the CPT soundings and SPT borings. The methodology to compute the liquefaction potential of soils and the factor of safety against liquefaction are described below.

Cyclic Stress Ratio

The Cyclic Stress Ratio (CSR) is a measure of the shear stresses developed during an earthquake and is normalized with effective overburden stress. The CSR for a depth interval is calculated as follows:

$$\mathrm{CSR}_{M,\sigma'_{vo}} = 0.65\,\frac{\tau_{max}}{\sigma'_{vo}} \tag{1}$$

where:

$CSR_{M,\sigma'vo}$ = cyclic stress ratio due to an earthquake with a magnitude, M, for an effective vertical stress, σ'_{vo}, at the depth interval (dimensionless);

τ_{max} = maximum shear stress developed at the depth interval during the seismic loading (psf); and

σ'_{vo} = effective vertical stress at the depth interval (psf).

The CSR represents the loading or demand on a soil unit during an earthquake.

Corrected Normalized CPT Sounding Interpretation

To evaluate the resistance or capacity of the soil against liquefaction, soil data must be interpreted from each boring or CPT sounding. A discussion of the interpretation of the CPT data is provided in the 2016 Safety Factor Assessment Report (Geosyntec, 2016). Equations used in the interpretation are reiterated below.

The normalized cone tip resistance ratio, Q, and normalized friction ratio, F, were calculated by:

$$Q = \left(\frac{q_c - \sigma_{vo}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vo}}\right)^n \tag{2}$$

and,

$$F = \left(\frac{f_s}{q_c - \sigma_{vo}}\right) \times 100\% \tag{3}$$

where:

q_c	= measured tip resistance (tsf);
σ_{vo}	= total vertical stress (tsf);
σ'_{vo}	= effective vertical stress (tsf);
P_a	= atmospheric pressure (P_a = 1.058 tsf);
n	= varies from 0.5 for sands to 1.0 for clays; and
f_s	= measured sleeve friction (tsf).

It is noted that the tip resistance (q_c) measured in the field must be adjusted for pore pressure effects on the cone tip if the data collection software does not automatically account for the area ratio of the cone. This correction is discussed within the 2016 Safety Factor Assessment Report (Geosyntec, 2016).

The soil behavior type index, I_c, as derived by Robertson and Wride (1998) is calculated by:

$$I_c = \left[\left(3.47 - \log(Q)\right)^2 + \left(\log(F) + 1.22\right)^2\right]^{0.5} \tag{4}$$

The I_c is used to compute the soil behavior type (SBT) index which may be used to infer the type of soil that is present at the depth interval.

To compute the resistance of a soil interval against liquefaction, the overburden-corrected tip resistance, q_{c1}, must be computed for the depth interval. q_{c1} can be computed as follows:

$$q_{c1} = C_N q_c \tag{5}$$

where:

C_N	= overburden correction factor = $(P_a/\sigma'_{vo})^{1.338-0.249(q_{c1Ncs})^{0.264}}$;
q_{c1N}	= normalized tip resistance q_{c1}/P_a (dimensionless); and
q_{c1Ncs}	= equivalent clean sand corrected tip resistance defined in the Cyclic Resistance Ratio (CRR) section.

The computation of C_N was limited to a maximum value of 1.7 and is applicable for values of q_{c1Ncs} between 21 and 254. As evident in the equations above and below, the computation of q_{c1}, q_{c1N}, and q_{c1Ncs} is an iterative procedure, which was performed using an algorithm developed within the MathCAD® computation software.

Corrected Normalized SPT Blow Count

Interpretation of soil test borings and SPT blow counts is discussed within the 2016 Safety Factor Assessment Report (Geosyntec, 2016) but is briefly reiterated below. The corrected normalized SPT blow count, $(N_1)_{60}$, which is applied in computing resistance of a soil against liquefaction, was calculated by the following equation:

$$(N_1)_{60} = N_{meas} C_E C_B C_S C_R C_N \tag{6}$$

where:

N_{meas} = measured SPT blow count (blows/ft);
C_E = correction factor for energy ratio;
C_B = correction factor for borehole diameter;
C_R = correction factor for rod length;
C_S = correction factor for sampler; and
C_N = correction factor for overburden pressure.

The correction factor for the applied energy (C_E) is dependent on the type and calibration of the hammer system attached to the drill rig. The correction factor (C_E) converts the measured N-value to a standard value, which assumes a 60 percent efficiency of the hammer system. This correction factor was computed as follows:

$$C_E = \frac{ER}{60} \tag{7}$$

where:

ER = energy ratio of the SPT hammer system.

Energy ratios selected for these analyses are discussed later within this Liquefaction Package. The correction factors above (excluding C_N) are given in Table 1. C_N was calculated for equivalent clean sand corrected SPT blow counts, $(N_1)_{60cs}$, (defined in the Cyclic Resistance Ratio (CRR) section) values less than 46 blows per foot, as follows:

$$C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^{\left(0.784 - 0.0768\sqrt{(N_1)_{60cs}}\right)} \tag{8}$$

where:

P_a = atmospheric pressure (2,117 psf); and
σ'_{vo} = effective vertical stress (psf).

The computation of C_N was limited to a maximum value of 1.7. As evident in the equations above and below, the computation of $(N_1)_{60}$ and $(N_1)_{60cs}$ is an iterative procedure, which was performed using an algorithm developed within the MathCAD® computation software.

Cyclic Resistance Ratio (CRR)

The Cyclic Resistance Ratio (CRR) is a measure of a soil's resistance to triggering of liquefaction. The CRR was computed from CPT sounding data based on the corrected tip resistance of clean sand for an earthquake of magnitude = 7.5 and an overburden pressure of one atmosphere, as follows:

$$CRR_{M=7.5,\sigma'_{vo}=1\,\text{atm}} = \exp\left(\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000}\right)^2 - \left(\frac{q_{c1Ncs}}{140}\right)^3 + \left(\frac{q_{c1Ncs}}{137}\right)^4 - 2.8\right) \quad (9)$$

Equation 9 is considered valid for the equivalent clean sand corrected tip resistance (q_{c1Ncs}) with values less than 211. For clean sands, q_{c1Ncs}, is equivalent to q_{c1N}, but for soils with some percentage of fines, $q_{c1Ncs} = q_{c1N} + \Delta q_{c1N}$, where the correction factor, Δq_{c1N}, is given by:

$$\Delta q_{c1N} = \left(11.9 + \frac{q_{c1N}}{14.6}\right) \times \exp\left(1.63 - \frac{9.7}{FC+2} - \left(\frac{15.7}{FC+2}\right)^2\right) \quad (10)$$

where:

FC = percent of fines (by mass).

Using corrected SPT N-values, the CRR was computed similarly for an earthquake of magnitude, M = 7.5, and an overburden pressure of one atmosphere, using corrected SPT N-values, as follows:

$$CRR_{M=7.5,\,\sigma'_{vo}=1\,\text{atm}} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right) \quad (11)$$

For clean sands, the equivalent clean sand value of the SPT penetration resistance $(N_1)_{60cs}$, is equivalent to $(N_1)_{60}$, but for soils with some percentage of fines, $(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$, where the correction factor, $\Delta(N_1)_{60}$, is given by:

$$\Delta(N_1)_{60} = \exp\left(1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right) \quad (12)$$

The selected fines content (FC) values used in these computations are discussed later within this calculation package. It is noted that $\Delta(N_1)_{60}$ is limited to a maximum value of 5.5.

Overburden Correction Factor

The overburden correction factor, K_σ, was introduced by Seed (1983) to adjust the CRR to a reference value of effective overburden stress because the CRR of sands is dependent on the effective overburden stress. The recommended relationship for K_σ is given by:

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{vo}}{P_a}\right) \leq 1.1 \tag{13}$$

where:

$$C_\sigma = 1 \,/\, (37.3 - 8.27(q_{c1Ncs})^{0.264}) \quad \leq 0.3 \text{ for CPT soundings.} \tag{14}$$

and,

$$C_\sigma = 1 \,/\, (18.9 - 2.55((N_1)_{60cs})^{0.5}) \quad \leq 0.3 \text{ for SPT borings.} \tag{15}$$

Furthermore, Equations 14 and 15 are applicable for q_{c1Ncs} and $(N_1)_{60cs}$ values less than 211 and 37 blows per foot, respectively. The overburden correction factor is used in liquefaction potential computations to adjust the CRR to a common effective overburden stress as shown by the following equation:

$$CRR_{\sigma'_{vo}} = K_\sigma \times CRR_{\sigma'_{vo}=1\,\text{atm}} \tag{16}$$

Magnitude Scaling Factor (MSF)

The magnitude scaling factor (MSF) is applied to adjust the CRR to the design earthquake magnitude, M. For cohesionless soils, the MSF is calculated using the equation below:

$$MSF = 6.9 \times \exp\left(\frac{-M}{4}\right) - 0.058, \text{ and MSF} \leq 1.8 \tag{17}$$

The MSF was calculated as 1.05 for a magnitude 7.3 earthquake, which was selected based on the deaggregation of the probabilistic seismic hazard as described in the Site Response Package.

The CRR for a magnitude M earthquake is calculated as follows:

$$CRR_M = MSF \times CRR_{M=7.5} \tag{18}$$

Age Correction Factor (K_{DR})

Correlations associated with liquefaction potential analysis were developed based on case histories of the presence or absence of liquefaction in relatively young soil deposits (i.e., Holocene age). As described in the South Carolina Department of Transportation (SCDOT) Geotechnical Design Manual

(GDM) (2019), the CRR may be adjusted to account for diagenesis and other age-related effects in older soils that have not previously experienced liquefaction. Equation 13-30 of the SCDOT GDM computes the Age Correction Factor (K_{DR}) based on its age (t in years) as:

$$K_{DR} = 0.13 \log_{10}(t) + 0.83 \tag{19a}$$

The K_{DR} in Equation 19a is limited to a maximum value of 2.09. Meanwhile, Andrus et al. (2008) presents a similar equation for the K_{DR} as:

$$K_{DR} = 0.19 \log_{10}(t) + 0.68 \tag{19b}$$

It is noted that "t" is considered based on the "geotechnical age" instead of the "geologic age". Geologic age is the time since initial soil deposition; whereas geotechnical age is the time since the last significant liquefaction event resulting in re-sedimentation of the soil fabric.

The CRR for sand strata was adjusted by the age correction factor to account for this aging effect, and is computed as follows.

$$CRR_{M,K} = K_{DR} \times CRR_{M} \tag{20}$$

Factor of Safety

The factors of safety against triggering of liquefaction (FS_{liq}) for both SPT and CPT analyses were computed by:

$$FS_{liq} = \frac{CRR_{M=7.5,\sigma'_{vo},Kdr}}{CSR_{M,\sigma'_{vo}}} \tag{21}$$

where:

$CRR_{M,\sigma'vo,Kdr}$ = cyclic resistance ratio adjusted for earthquake magnitude, overburden, and aging ($CRR_{M=7.5,\sigma'_{vo}=1\,atm} \times K_\sigma \times MSF \times K_{DR}$); and

$CSR_{M,\sigma'vo}$ = cyclic stress ratio for the corresponding design earthquake magnitude and overburden stress at the depth interval.

ANALYSIS CASES

As noted previously, liquefaction potential computations were conducted on soil data collected in soil borings and CPT soundings overseen by Geosyntec in 2013 and on historical borings performed by Paul C. Rizzo and Associates (PCRA) in 1999. PCRA performed an investigation in 1999 to evaluate phreatic conditions through the Slurry Pond perimeter dikes. PCRA advanced some borings without sampling and without SPTs to the top of the Chicora stratum. PCRA boring logs from 1999 where SPTs were collected were utilized as part of this Liquefaction Package. Santee Cooper personnel indicated that no additional geotechnical subsurface investigations have been conducted since 2016. Computations were limited to soil borings and CPT soundings located through the dike centerline into the dike fills and foundation soils immediately underlying the perimeter dikes.

Three representative soil profiles of shear wave velocity (V_s) were developed from the dike fill soils to the Chicora stratum as presented in the 2016 Safety Factor Assessment Report (Geosyntec, 2016). These profiles were developed from direct measurements of V_s and by means of a correlation with CPT sounding data. As discussed in the 2016 Safety Factor Assessment Report, these representative V_s profiles were supplemented with historical data to extend the V_s profile into the underlying Chicora and Williamsburg Formation Clay strata for the site response analyses of the Slurry Pond.

No zones expected to undergo triggering of liquefaction within the Slurry Pond were identified in the 2016 Safety Factor Assessment. However, there were some thin layers of soil deposits with calculated FS against triggering of liquefaction approaching 1.0 observed within the eastern and northern portions of the Slurry Pond perimeter dikes, represented by Profiles 1 and 2, respectively, in the Site Response Package, in the 2016 Safety Factor Assessment. Additionally, lower calculated FS for slope stability of cross sections through the eastern and northern portions of the Slurry Pond perimeter dikes were observed in the 2016 Safety Factor Assessment. Therefore, site response analyses were only performed for Profiles 1 and 2 for the 2021 Safety Factor Assessment Report to provide an updated evaluation of liquefaction potential of the subsurface materials in the critical areas of the Slurry Pond. Only the investigations along Profile 1 (CPT-01 through CPT-10, GSB-03, GSB-04, B-206, B-212, and B-214) and Profile 2 (CPT-13 through CPT-28, GSB-05, GSB-07, GSB-08, B-202, and B-209) are considered in the liquefaction potential evaluation presented in this calculation package, as shown on Figure 1.

For Profiles 1 and 2, site response analyses, described within the Site Response Package, were performed using six ground motions selected for the Site. A profile of the maximum shear stress (τ_{max}) was computed for each ground motion and the maximum value at each depth was calculated to create a single profile of τ_{max} for each of Profiles 1 and 2. The τ_{max} profiles were used to compute the CSR at every depth for each boring or CPT sounding. The maximum shear stress at each computed depth for Profiles 1 and 2 are provided in Table 2. The τ_{max} for depths between the intervals listed within Table 2 were linearly interpolated.

INPUT PARAMETERS

The following section describes the selection of the input parameters applied for the liquefaction potential analysis.

Calibration of Historical Borings

Liquefaction potential of dike fill soils was evaluated based on data provided by historical boring logs, in addition to borings and CPT soundings overseen by Geosyntec (2013). As stated previously, correlations developed to predict soil resistance to liquefaction are based on empirical observations using a standard procedure or method during drilling activities. The historical geotechnical reports did not explicitly provide details of the methodology during the geotechnical investigation. However, Geosyntec intentionally located a soil test boring and CPT sounding directly adjacent to a historical PCRA boring during the 2013 investigation to "calibrate" the historical boring to Geosyntec's soil borings, which were conducted in accordance with current industry standards. Based on a comparison of data, Geosyntec assumed for the purposes of this analyses that historical PCRA borings were advanced using mud rotary wash methods with a bit that deflects the drilling fluid and creates a borehole approximately four inches in diameter. Furthermore, calibration of historical borings demonstrated that an Energy Ratio (ER) between 60 percent and 70 percent for the hammer system during SPT testing corresponds well with Geosyntec data at the same location, as shown on Figure 2. Thus, an ER of 70 percent was used for the liquefaction potential analysis of PCRA borings (denoted by "B-2XX") for the Slurry Pond. The subcontractor during Geosyntec's 2013 investigation, Soil Consultants, Inc. (SCI), reported that the automatic hammer on the utilized drilling rig had an ER of 88 percent, which was independently evaluated within six months of the investigation.

Total Unit Weight

The total unit weight (γ_T) was used to calculate the total and effective stresses for the soil column for each boring and sounding analyzed. For the purposes of this analysis, CPT intervals were assigned a unit weight based on the ranges presented for soils in the region provided within the SCDOT GDM (SCDOT, 2019). The assigned unit weight is dependent on the measured soil behavior index (I_c) as follows:

- Clays and clayey sand mixtures ($I_c > 2.95$): 100 pcf;
- Silt to silty sand mixtures ($2.60 < I_c \leq 2.95$): 100 pcf;
- Silty sands to sand mixtures ($2.05 < I_c \leq 2.60$): 110 pcf;
- Sands ($1.31 < I_c \leq 2.05$): 120 pcf; and
- Gravelly sands to sands ($I_c \leq 1.31$): 125 pcf.

SPT intervals were assigned total unit weight values based on visual and laboratory observations on the soil type as follows:

- Clays and Silts: 100 pcf;
- Loose Sands ($N \leq 10$ blows/foot): 105 pcf;
- Medium Dense Sands (10 blows/foot $< N \leq 30$ blows/foot): 115 pcf

- Dense Sands (N ≥ 30 blows/foot): 120 pcf
- Chicora: 130 pcf
- Williamsburg Formation Clay: 105 pcf

Age Correction Factor

The susceptibility of soil deposits to liquefaction was summarized by type of deposit and geologic age by Youd and Perkins (1978) (Table 3). Youd and Perkins (1978) observed that younger soils (Holocene age) generally are the most susceptible to liquefaction. In the South Carolina (SC) region, the influence of soil age was investigated locally by Arango et al. (2009) and Andrus et al. (2008) based on cyclic strength testing of high-quality samples of sand and in-situ testing on paleoliquefaction sites, respectively. Each researcher compared observations and results in each study with the case-history-based chart for liquefaction triggering developed by Idriss and Boulanger (2008). Andrus (2008) developed a correlation (Equation 19b) relating soil age to a correction factor for CRR. Additionally, Leon et al. (2005) investigated a site nearby to WGS (Sampit, SC) and identified soil ages for sands encountered between 546 to 450,000 years old. Age correction factors (K_{DR}) were computed based on Equations 19a and 19b for the range of soil ages observed in the region presented by Leon et al. (2005) and are provided in Table 4. A K_{DR} was selected from Table 4 and applied to soils in the vicinity of the Slurry Pond perimeter dikes that were evaluated to be of geologic and geotechnical ages older than Holocene age (i.e., foundation soils).

As shown in Figure 3, soils immediately surrounding the Slurry Pond perimeter dikes were determined by the SC Department of Natural Resources (2012) to be of Pleistocene age. It was assumed that these soils are located beneath the recompacted dike fill soils, which are considered to be of Holocene age due to the relatively "recent" construction. Based on the range of soil ages presented in Table 4, an age correction factor of 1.2 was selected for Pleistocene-aged, foundation soils at WGS. An age correction factor of 1.0 was applied for dike fill soils, as these structures are approximately 40 years old. As noted previously, "geologic" age differs from "geotechnical" age. Geologic age refers to the overall age of the soil since deposition. Geotechnical age refers to the age of the soil since the last instance of liquefaction. The geotechnical age was considered in the selection of K_{DR}. The bottom of dike fill soils at each investigation point was estimated based on the surface elevation of historical borings conducted prior to dike construction by S&ME (1978). Foundation soils were assumed to begin at the surface elevation of the nearest historical boring adjacent to the investigation point. Information for the investigation points considered in this calculation package are summarized in Table 5.

Fines Content

As shown in Equations 9 through 12, the CRR is influenced by the fines content (percent particles by mass passing a No. 200 sieve). An increase in fines content of the soil results in larger CRR. The 2016 Safety Factor Assessment Report (Geosyntec, 2016) showed the fines content of dike fill and foundation materials is variable across the Site. Physical samples are not collected during CPT soundings and many historical borings did not extensively test soil index properties, so representative fines content profiles were developed based on index testing data presented within the 2016 Safety Factor Assessment Report and applied to CPT soundings when a soil boring was not located in the vicinity of the sounding. Index testing for soil borings, when available, were utilized for each individual SPT N-value. The representative fines content profiles are shown in Figures 4a and 4b and summarized in Table 6. The extent of the representative fines content profiles (Profiles A through C) is delineated in Figure 5. The assignment of the representative fines content profiles for the investigation points is summarized within Table 5.

Phreatic Surface

The phreatic surface through the perimeter dikes to the downstream toe of the dike at the time of the 2016 Safety Factor Assessment (Geosyntec, 2016) was developed predominantly based on water levels collected from temporary piezometers installed through the Slurry Pond perimeter dikes in 2014, CPT porewater pressure signatures, and CPT porewater pressure dissipation tests.

To support the 2021 Safety Factor Assessment, Santee Cooper provided available water level measurements from wells in the Slurry Pond area, located outside the downstream toe of the pond perimeter dike. The recorded water levels in these wells have generally been steady over the last five years. The free water within the Slurry Pond has been maintained at an operating pool elevation of 19.6 ft NGVD29. The phreatic surface within the perimeter dikes may be lowered due to water management operations since the 2016 Safety Factor Assessment. However, given the limited water level measurements along the Slurry Pond perimeter dikes, the water level within the perimeter dike is conservatively assumed to be similar to the water level used for the 2016 Safety Factor Assessment. As detailed in the Site Response Package, site response analyses were performed with the water table modeled at 15 ft below ground surface to be consistent with the modeled water table depth for the 2016 Safety Factor Assessment and a maximum shear stress profile was calculated for this water table elevation.

For the liquefaction potential evaluation presented in this calculation package, the phreatic surface assumptions through the Slurry Pond perimeter dikes at the time of the boring (TOB) were used to estimate CRR profiles based on the measurements at the time of the investigation. CSR profiles were estimated for the time at which the earthquake event occurs conservatively using the phreatic surface assumed for the 2016 Safety Factor Assessment. The elevations of the phreatic surface through the Slurry Pond perimeter dikes at TOB and at the time of analysis (TOA) for this calculation package are summarized in Table 5.

RESULTS

The methodology discussed previously was applied within a MathCAD® algorithm similar to the spreadsheets presented in Idriss and Boulanger (2008). Computations were performed on soil borings and CPT soundings (including the historical borings) located at the dike centerline. The factor of safety against liquefaction (FS_{liq}) was computed at every depth interval where data was collected for soil test borings (in 2-ft or 5-ft intervals) and CPT soundings (in 0.16-ft intervals). The computed FS_{liq} for the soil borings and CPT soundings within Profiles 1 and 2 are shown in Figures 6 through 16. Figure 6 shows CPT-01 and CPT-03, which are located in the southeast corner of the Slurry Pond immediately north of the divider dike. Subsequent figures depict calculation results for soil borings and CPT soundings positioned progressively in a counter-clockwise direction around the surface impoundment. As shown in the figures, the minimum computed FS_{liq} is greater than 1.1 and is found in foundation soil layers with thicknesses less than 2 ft. No zones of soils expected to undergo triggering of liquefaction (FS_{liq} below 1.0) under the design earthquake were indicated by the results. Example calculations are provided within Appendix 1.

CONCLUSIONS

Based on the liquefaction potential computations presented within this calculation package, the calculated FS_{liq} values are greater than 1.1. Therefore, the dike fill soils (i.e., native soils recompacted to form impounding perimeter dikes) and foundation soils beneath the perimeter dikes of the Slurry Pond are not expected to undergo triggering of liquefaction under the design earthquake. Given zones expected to undergo triggering of liquefaction were not identified for borings and CPT soundings advanced through the Slurry Pond perimeter dikes, post-liquefaction stability and displacement analyses are not warranted for the Slurry Pond perimeter dikes.

REFERENCES

Andrus, R. Gassman, S. L., Talwani, P., Hasek, M., Camp, W. Hayati, H., and Boller, R. (2008), "Characterization of Liquefaction Resistance of Aged Soils: Summary of Selected First Year Findings", *Proceedings of 2008 NSF Engineering Research and Innovation Conference. Knoxville, Tennessee,* NSF Grant # CMS-0556006.

Arango, I., Lewis, M. R., and McHood, M. D. (2009), "Site Characterization Philosophy and Liquefaction Evaluation of Aged Sands", *Bechtel Technology Journal*, Vol. 2, No. 1.

Boulanger, R. W. and Idriss, I. M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures", *Report No. UCD/CGM-14/01*, Center for Geotechnical Monitoring, University of California, Davis, CA.

Bray, J.D. and Sancio, R.B. (2006) "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils". *Journal of Geotechnical and Geoenvironmental Engineering*, 132 (9), 1165-1177.

Geosyntec Consultants, Inc. (2016), "2016 Surface Impoundment Periodic Safety Factor Assessment Report: Slurry Pond, Winyah Generating Station, Georgetown, South Carolina", submitted to Santee Cooper.

Idriss, I. M. and Boulanger, R. W. (2008), "Soil Liquefaction During Earthquakes", *Earthquake Engineering Research Institute*, EERI Publication MNO-12.

Leon, E., Gassman, S. L., and Talwani, P. (2005), "Effect of Soil Aging on Assessing Magnitudes and Accelerations of Prehistoric Earthquakes", *Earthquake Spectra*, Vol. 21, No. 3 pg. 737-759.

Robertson, P.K. and Wride, C.E. (1998), "Evaluating cyclic liquefaction potential using the cone penetration test, *Canadian Geotechnical Journal*, Volume 35, No. 3, pp. 442-59.

Seed, H.B. (1983), "Earthquake Resistant Design of Earth Dams", in *Proceedings, Symposium of Seismic Design of Embankments and Caverns, Pennsylvania*, ASCE, NY, pp. 41-64.

Seed, H.B, and Idriss, I.M. (1971), "Simplified Procedure for Evaluation Soil Liquefaction Potential", *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 107, NO. SM9.

South Carolina Department of Transportation (2019), "Geotechnical Design Manual Version 2.0", available: https://www.scdot.org/business/pdf/geotech/SCDOT-Geotechnical-Design-Manual.pdf

South Carolina Department of Natural Resources: Geologic Survey (2012), "Geologic Map of the Georgetown South Quadrangle, Georgetown County, South Carolina".

Youd, T. L. and Perkins, M. (1978), "Mapping liquefaction-induced ground failure potential", *J. Geotechnical Eng. Div.*, ASCE 104(GT4), 433-46.

Tables

Table 1. Correction Factors for Interpretation of SPT for Liquefaction Potential Analysis (Idriss and Boulanger, 2008)

Factor	Description
Energy ratio	Energy measurements are required to determine the delivered energy ratios or to calibrate the specific equipment being used. The correction factor is then computed as $$C_E = \frac{ER_m}{60}$$ where ER_m is the measured energy ratio as a percentage of the theoretical maximum. Empirical estimates of C_E (for rod lengths of 10 m or more) involve considerable uncertainty, as reflected by the following ranges: Doughnut hammer $C_E = 0.5$–1.0; Safety hammer $C_E = 0.7$–1.2; Automatic triphammer $C_E = 0.8$–1.3 (Seed et al. 1984, Skempton 1986, NCEER 1997)
Borehole diameter	Borehole diameter of 65–115 mm $C_B = 1.0$; Borehole diameter of 150 mm $C_B = 1.05$; Borehole diameter of 200 mm $C_B = 1.15$ (Skempton 1986)
Rod length	Where the ER_m is based on rod lengths of 10 m or more, the ER delivered with shorter rod lengths may be smaller. Recommended values from Youd et al. (2001) are as follows: Rod length < 3 m $C_R = 0.75$; Rod length 3–4 m $C_R = 0.80$; Rod length 4–6 m $C_R = 0.85$; Rod length 6–10 m $C_R = 0.95$; Rod length 10–30 m $C_R = 1.00$
Sampler	Standard split spoon without room for liners (the inside diameter is a constant $1^3/_8$ in.). $C_S = 1.0$. Split-spoon sampler with room for liners but with the liners absent (this increases the inside diameter to $1^1/_2$ in. behind the driving shoe): $C_S = 1.1$ for $(N_1)_{60} \leq 10$; $C_S = 1 + \frac{(N_1)_{60}}{100}$ for $10 \leq (N_1)_{60} \leq 30$; $C_S = 1.3$ for $(N_1)_{60} \geq 30$ (from Seed et al. 1984, equation by Seed et al. 2001)

Table 2. Calculated Maximum Shear Stress for the Dike Centerline

Profile 1		Profile 2	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	38	2.5	35
7.5	94	7.5	93
12.5	151	12.5	138
17.5	206	17.5	187
22.5	258	22.5	235
27.5	299	27.5	272
32	327	32	293
36	346	36	301
40	353	40	305
44	353	44	304
48	363	48	313
52.5	440	52.5	389
60	551	60	487
70	612	70	582
80	728	80	674
90	860	90	815
100	989	100	919

Notes:

1. Profiles were developed in the Site Response Package.
2. For calculation points located in between the depth intervals listed above, the average τ_{max} was linearly interpolated for liquefaction potential computations.

Table 3. Susceptibility of Soil Deposits to Liquefaction during Strong Seismic Shaking (Youd and Perkins, 1978)

Type of deposit	Distribution of cohesionless sediments in deposit	Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction			
		< 500 years	Holocene	Pleistocene	Pre-Pleistocene
Continental					
River channel	Locally variable	Very high	High	Low	Very low
Floodplain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plains	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	—	Low	Very low	Very low
Delta and fan delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebkha	Locally variable	High	Moderate	Low	Very low
Coastal zone					
Delta	Widespread	Very high	High	Low	Very low
Estuarine	Locally variable	High	Moderate	Low	Very low
Beach—high wave energy	Widespread	Moderate	Low	Very low	Very low
Beach—low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Foreshore	Locally variable	High	Moderate	Low	Very low
Artificial fill					
Uncompacted fill	Variable	Very high	—	—	—
Compacted fill	Variable	Low	—	—	—

Table 4. Age Correction Factor (K_{DR}) based on Soil Age

Soil Age, t (years)	K_{DR} [1]	K_{DR} [2]
126	1.10	1.08
546	1.19	1.20
5,038	1.31	1.38
10,000	1.35	1.44
450,000	1.56	1.75

Notes:

1. K_{DR} computed by SCDOT GDM (SCDOT, 2019), as provided in Equation 19a.
2. K_{DR} computed by Andrus et al (2008) as provided in Equation 19b.

Table 5. Summary of Soil Borings and Soundings Analyzed for Liquefaction Potential

Boring ID	Northing	Easting	Elevation at TOB	Dike Base Elevation	Dike Base Basis	GWT El. at TOB	GWT Depth at TOB	GWT El. at TOA	GWT Depth at TOA	τ_{max} Profile	FC Basis
-	ft	ft	ft NGVD29	ft NGVD29	-	ft NGVD29	ft	ft NGVD29	ft	-	-
CPT-01	548805.925	2500016.426	39.38	25.20	SC-52	29.38	10.00	29.38	10.00	Profile 1	Profile A
CPT-03	549179.558	2500199.574	38.33	19.00	SC-42	26.33	12.00	26.33	12.00	Profile 1	Profile A
CPT-04	549351.863	2500330.724	38.62	19.00	SC-42	22.62	16.00	22.62	16.00	Profile 1	Profile A
CPT-06	549456.322	2500411.877	38.32	17.50	SC-41	22.32	16.00	22.32	16.00	Profile 1	Profile A
CPT-07	549602.193	2500529.564	38.51	17.50	SC-41	22.51	16.00	22.51	16.00	Profile 1	Profile A
CPT-10	550077.297	2500483.518	37.98	15.30	SC-28	21.98	16.00	21.98	16.00	Profile 1	Profile B
CPT-13	550273.629	2500271.050	38.22	15.30	SC-28	22.22	16.00	21.22	17.00	Profile 2	Profile B
CPT-17	550690.756	2499821.833	38.07	16.40	SC-27	16.00	22.07	22.07	16.00	Profile 2	GSB-5
CPT-19	550993.800	2499493.874	38.51	17.10	SC-26	22.51	16.00	16.00	22.51	Profile 2	Profile B
CPT-20	551089.929	2499388.484	38.57	17.50	SC-20	22.57	16.00	16.00	22.57	Profile 2	Profile C
CPT-24	551266.120	2499180.501	38.61	17.50	SC-20	16.00	22.61	16.00	22.61	Profile 2	Profile C
CPT-26	551311.434	2498800.750	38.16	15.80	SC-21	22.16	16.00	16.00	22.16	Profile 2	Profile C
CPT-28	551131.581	2498327.600	38.52	9.00	SC-22	16.00	22.52	16.00	22.52	Profile 2	Profile C
GSB-03	549760.850	2500650.369	38.39	18.20	SC-40	28.39	10.00	28.40	9.99	Profile 1	GSB-03
GSB-04	550370.128	2500167.341	38.66	15.30	SC-28	28.66	10.00	28.40	10.26	Profile 1	GSB-04
GSB-05	550684.521	2499829.144	38.05	16.40	SC-27	22.05	16.00	16.00	22.05	Profile 2	GSB-05
GSB-07	551318.990	2498950.556	38.16	15.80	SC-21	22.16	16.00	16.00	22.16	Profile 2	GSB-07
GSB-08	551066.479	2498252.288	39.19	9.00	SC-22	23.19	16.00	16.00	23.19	Profile 2	GSB-08
B-202	2499022.861	551293.4313	38.2	15.80	SC-21	22.20	16.00	16.00	22.20	Profile 2	GSB-7
B-206	2500292.656	549310.6617	38.9	19.00	SC-42	26.90	12.00	26.90	12.00	Profile 1	B-206 / Profile A
B-209	2499818.024	550674.9716	38.8	16.40	SC-27	22.80	16.00	16.00	22.80	Profile 2	GSB-05
B-212	2500549.491	550003.0901	38.6	18.20	SC-40	22.60	16.00	22.60	16.00	Profile 1	B-212 / Profile B
B-214	2499731.727	548665.4911	39.1	25.20	SC-52	29.10	10.00	29.10	10.00	Profile 1	B-214 / Profile A

Notes:

1. ft NGVD29 - feet National Geodetic Vertical Datum of 1929; TOB - Time of Boring; TOA -Time of Analysis; GWT - Groundwater Table; FC - Fines Content.
2. GSB-series and CPTs were advanced by Geosyntec in spring 2013; and B-series borings were advanced by PCRA in 1999.
3. Dike base elevation was estimated based on the elevation of the nearest historical boring (prior to construction) performed by S&ME in 1978.

Table 6. Summary of Fines Content Profiles

Profile A		Profile B		Profile C	
Elevation (ft)	percent	Elevation (ft)	percent	Elevation (ft)	percent
38	35	38	40	38	40
18	35	15	40	18	40
18	20	15	22	18	10
4	20	10	22	2	10
4	25	10	10	2	16
-15	25	0	10	-15	16
-	-	0	20	-	-
-	-	-15	20	-	-

Notes:

1. Fines content profiles were developed from Geosyntec and historical fines content laboratory testing data. Results were spatially grouped together by area and plotted with respect to elevations to evaluate trends in the subsurface, because physical samples were not available directly adjacent to each CPT sounding. These profiles were applied to the CPT soundings.
2. The extent of the fines content profiles along the Slurry Pond perimeter dikes is shown on Figure 5.
3. Elevations refer to ft NGVD29.

Figures

N

LEGEND

W-SW-APB	EXISTING STAFF GAUGE
CPT-144	GEOSYNTEC CONE PENETRATION TEST
PPZ-5	PIEZOMETER
SPT-111	GEOSYNTEC SOIL BORING
B-1	HISTORICAL BORING
WAP-9	GROUNDWATER MONITORING WELL
TP-1	TEST PIT
	POND BOUNDARY
	PROPERTY BOUNDARY
	EXISTING GAS LINE
	EXISTING RAILROAD

NOTES:
1. Aerial imagery was obtaied from ESRI online database.
2. The position of underground utilities shown should be considered approximate.

600 300 0 600 Feet

SLURRY POND
WINYAH GENERATING STATION

GEORGETOWN, SOUTH CAROLINA

Geosyntec consultants

Charlotte, NC | October 2021

Figure 1

Y:\S\Santee Cooper\Santee Cooper - Winyah\2021 - 5 Yr CCR Rule Requirements\7 - Safety Factor Assessment\GIS\MXD\September 2021\FigureX_Slurry Pond_No X-Sections.mxd; Abby Wesley; 10/12/2021

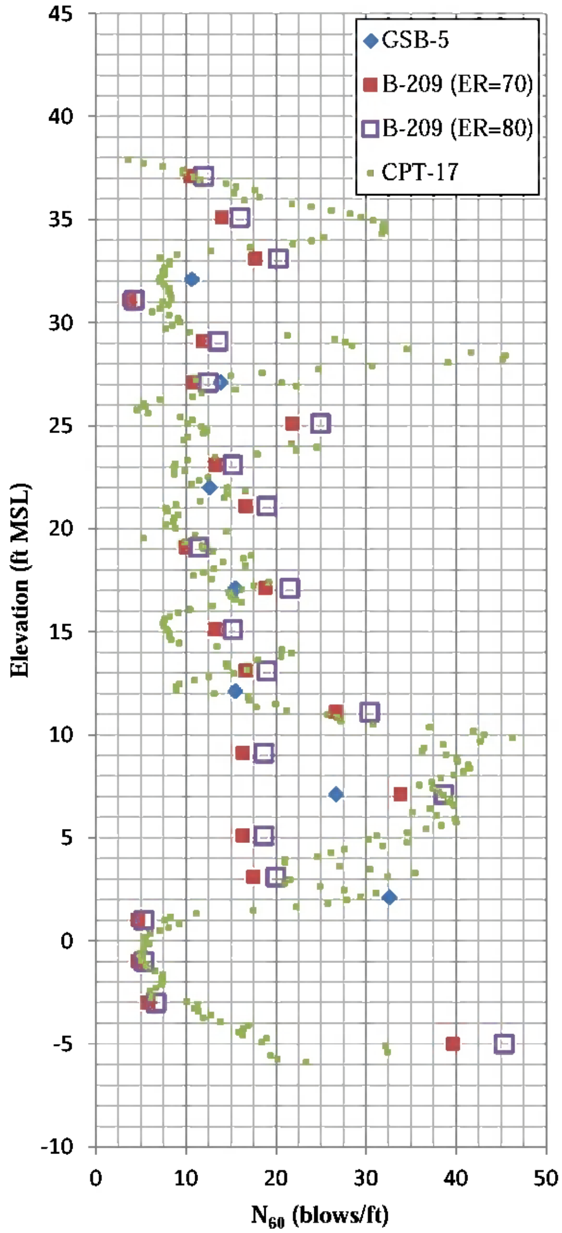
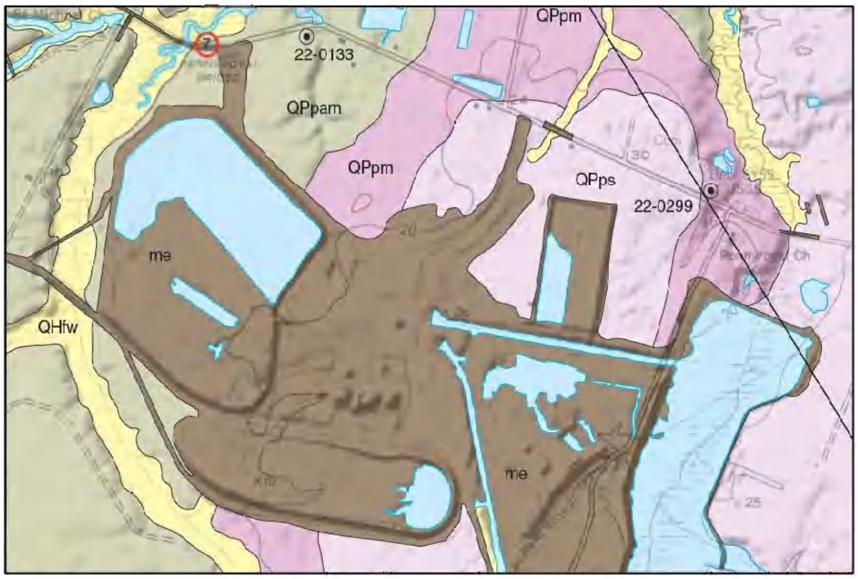
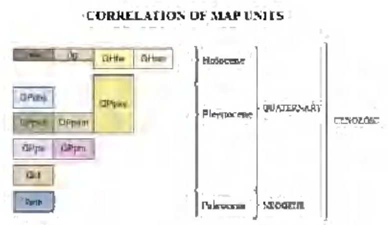


Figure 2. Geosyntec - PCRA Calibration Boring

Note:

1. A computed N_{60} profile is provided for CPT-17, which is directly adjacent to GSB-5 and B-209.





QPpam **Estuarine deposits or marine deposits or both (Pleistocene)** – In its lower part, quartz and phosphatic sand, medium bluish-gray (5B 5/1), poorly sorted, subrounded to very angular, fine to very coarse grained with trace amounts of very fine grained heavy minerals. Lower part 10 to 20 feet thick. In upper part, muddy sand to sandy mud, clay, silt, silty sand, clayey sand, phosphatic sand and quartz sand and shells, medium light-gray (N6) to medium bluish-gray (5B 5/1). Some zones contain broken and intact *Oliva, Polinices, Terebra, Mercenaria and Dosinia*. Upper part 1 to 10 feet thick.

Figure 3. Geologic Map of Areas Surrounding the Slurry Pond
(Map taken from SC Department of Natural Resources: Geological Survey, 2012)

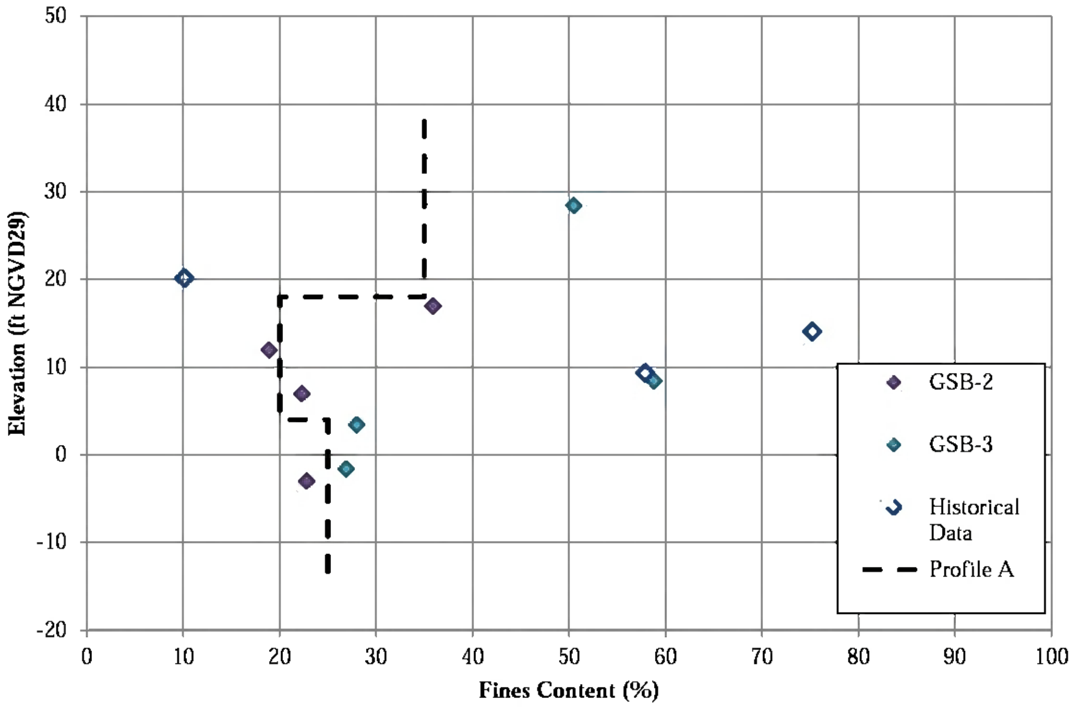


Figure 4a. Representative Fines Content Profile A

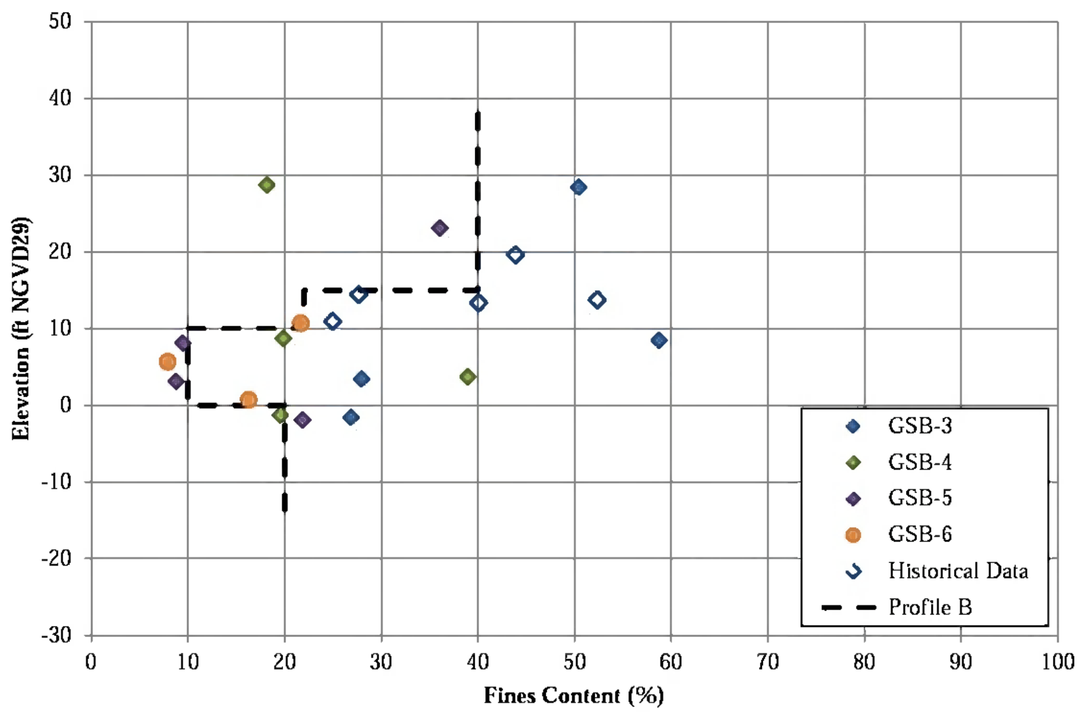


Figure 4b. Representative Fines Content Profile B

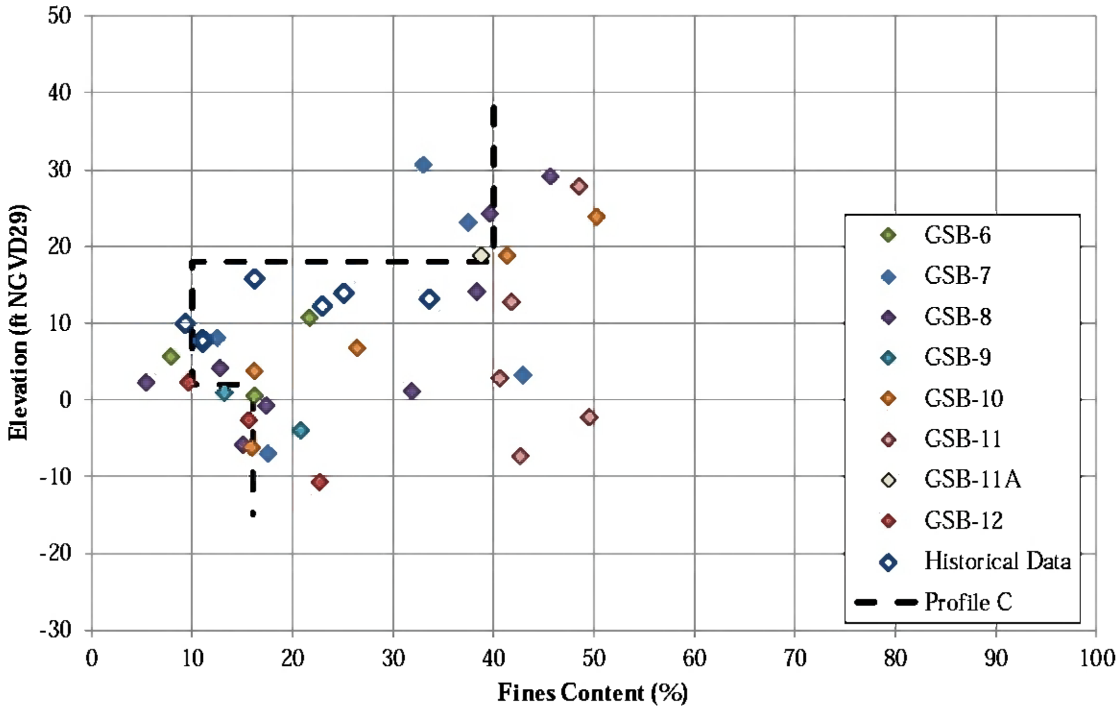


Figure 4c. Representative Fines Content Profile C

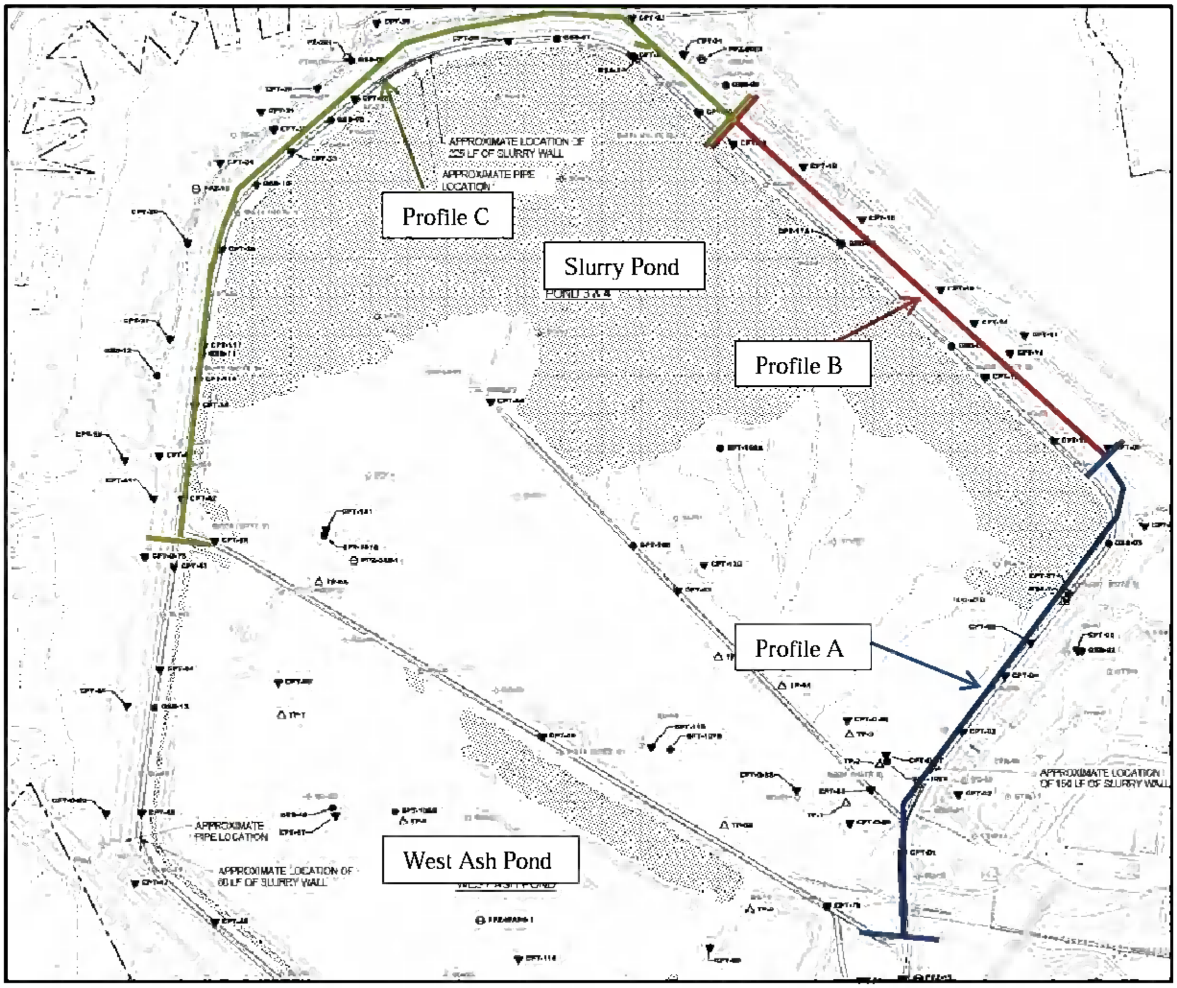


Figure 5. Location and Extent of Fines Content Profiles along the Slurry Pond Perimeter Dikes

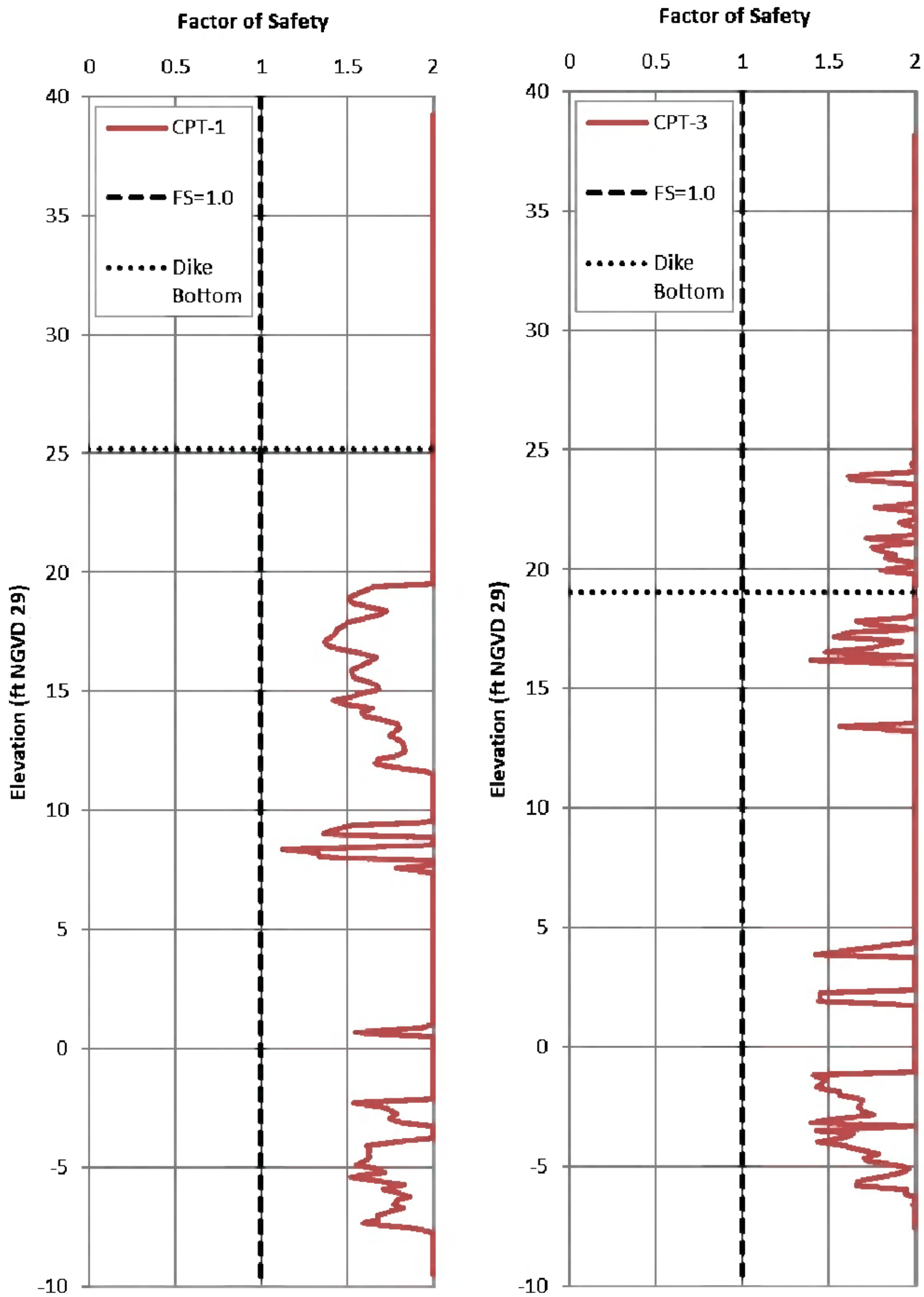


Figure 6. Liquefaction Results for Dike and Foundation Soils for CPT-1 and CPT-3

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings as provided in Table 5.

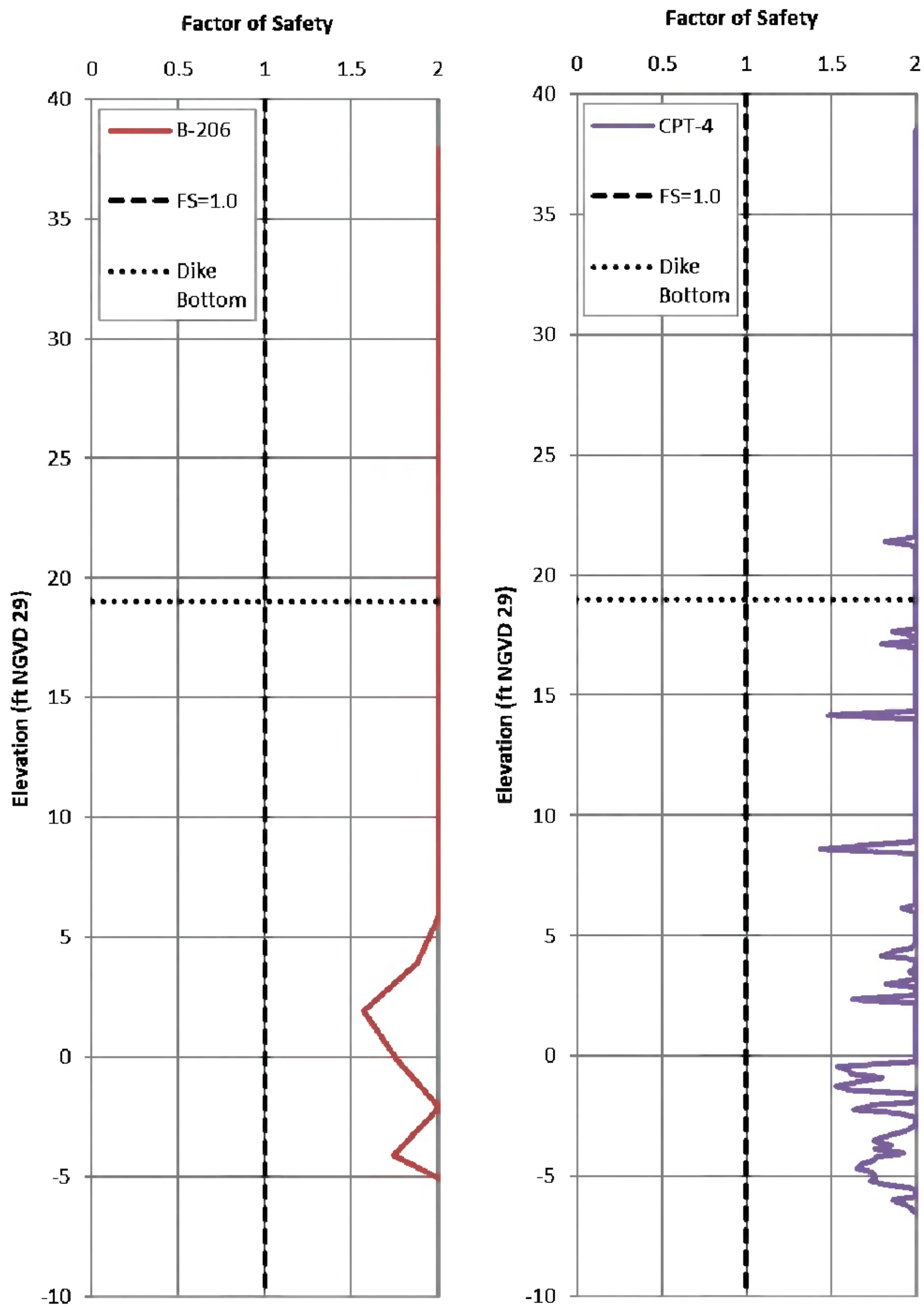


Figure 7. Liquefaction Results for Dike and Foundation Soils for B-206 and CPT-4

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings as provided in Table 5.

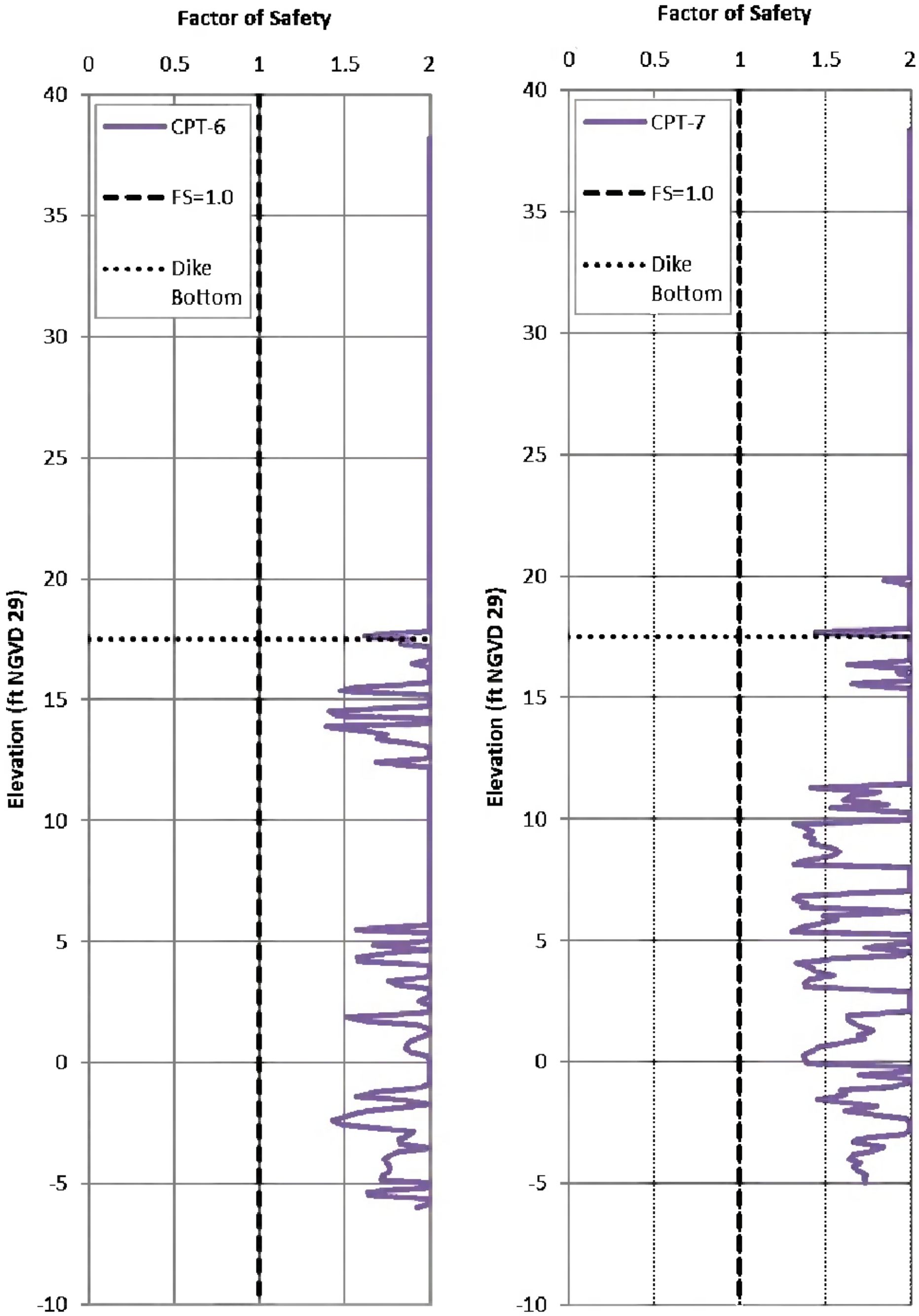


Figure 8. Liquefaction Results for Dike and Foundation Soils for CPT-6 and CPT-7

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings as provided in Table 5.

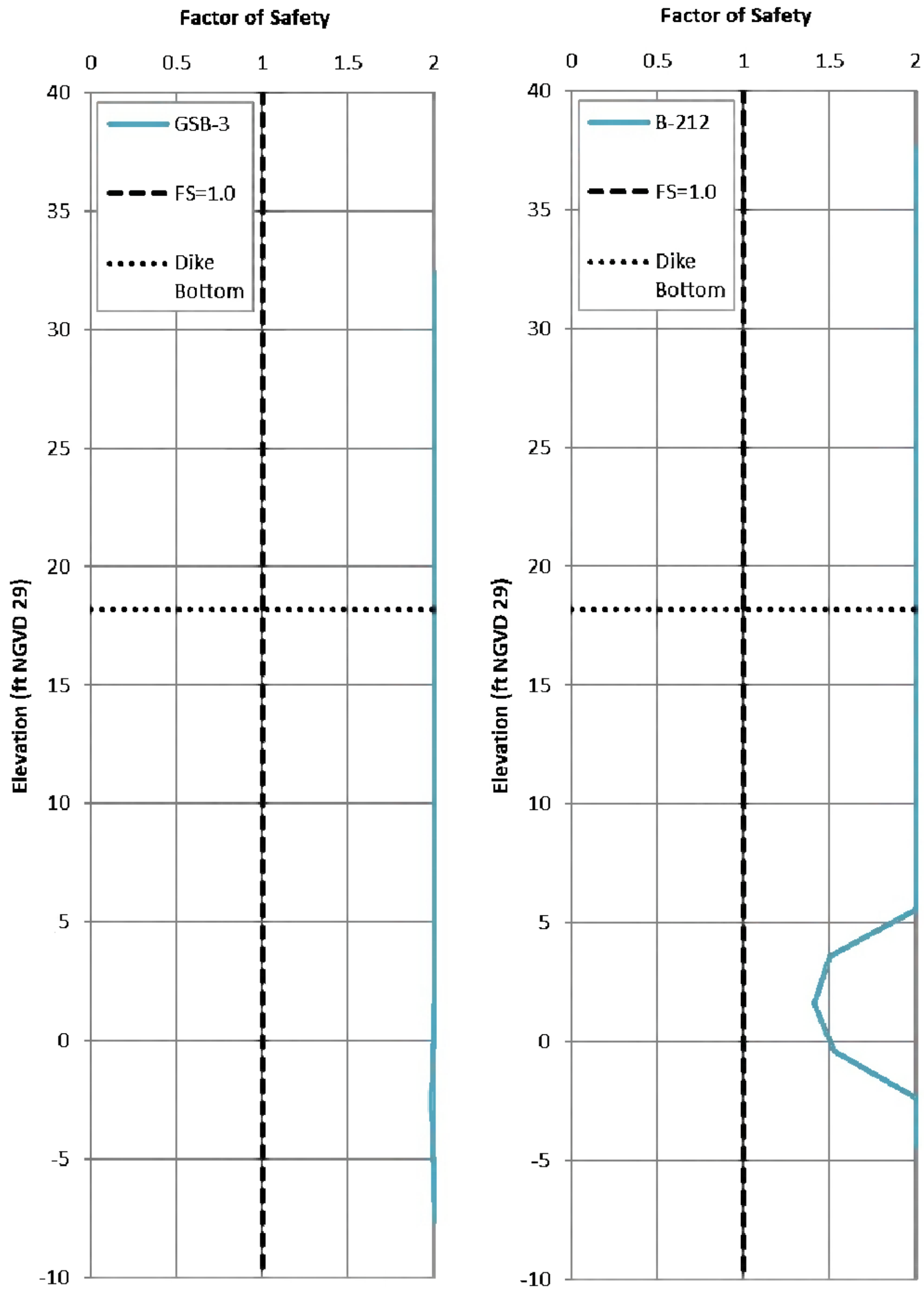


Figure 9. Liquefaction Results for Dike and Foundation Soils for GSB-3 and B-212

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings as provided in Table 5.

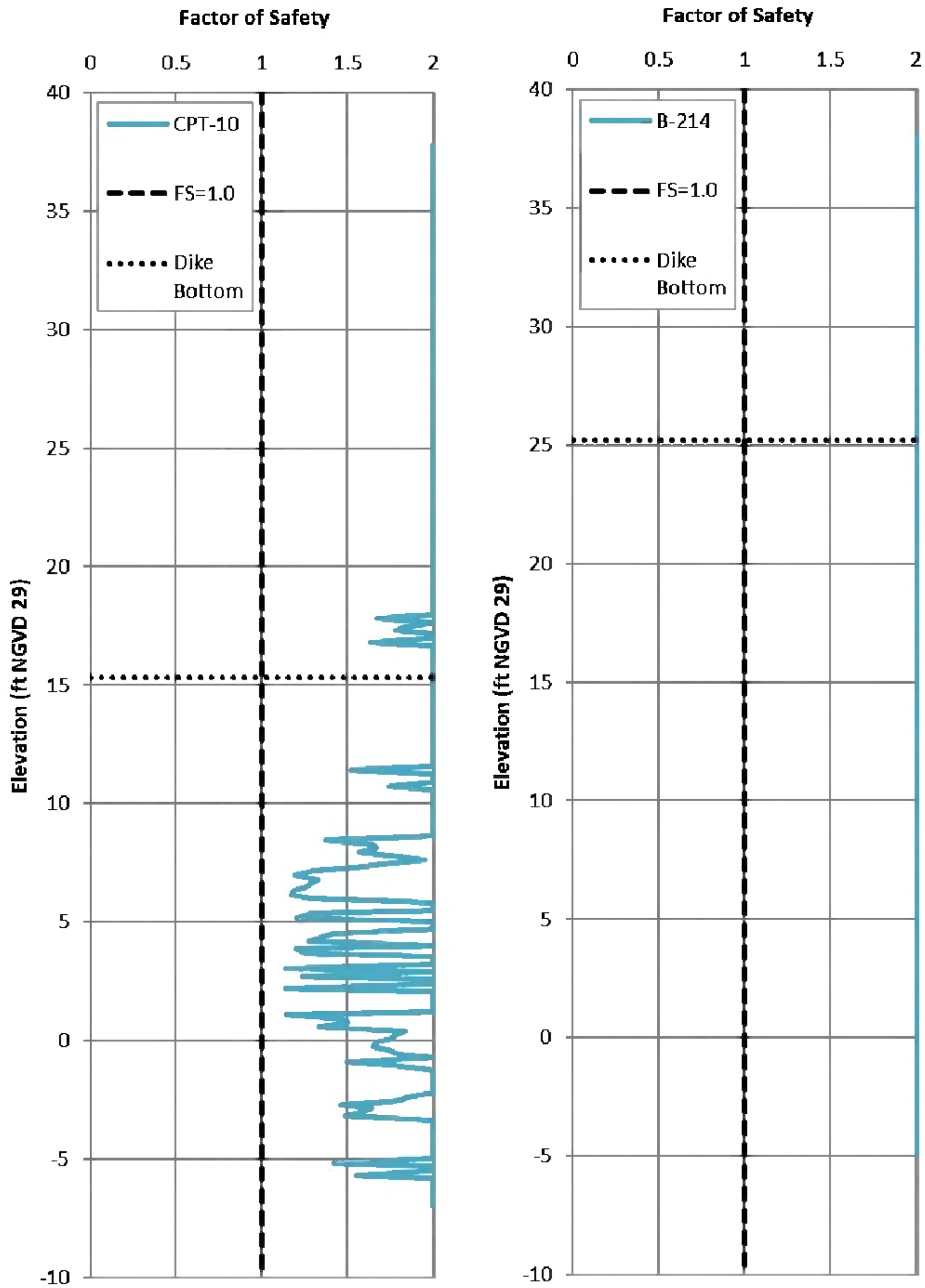


Figure 10. Liquefaction Results for Dike and Foundation Soils for CPT-10 and B-214

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings as provided in Table 5.

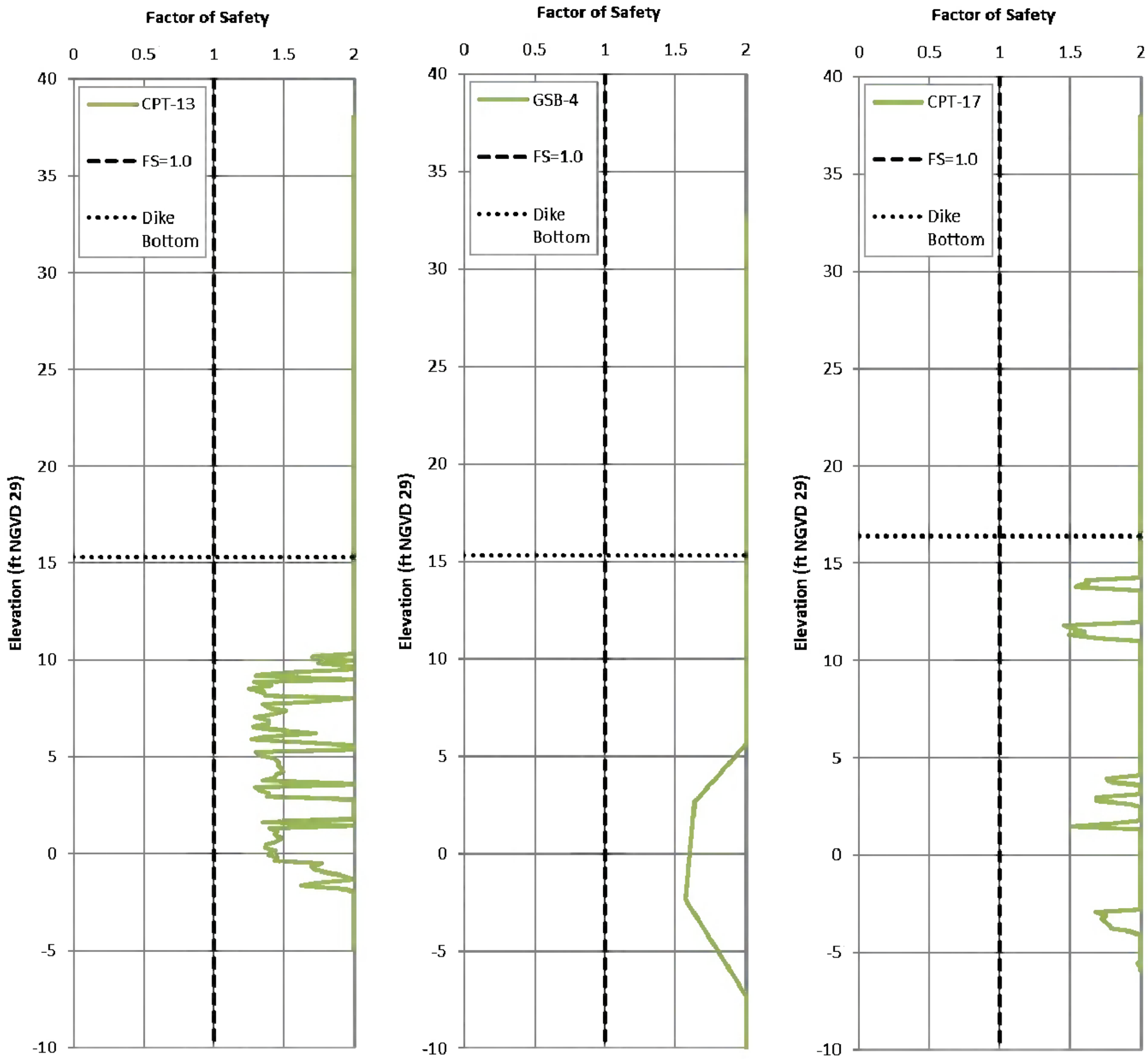


Figure 11. Liquefaction Results for Dike and Foundation Soils for CPT-13, GSB-4, and CPT-17

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings as provided in Table 5.

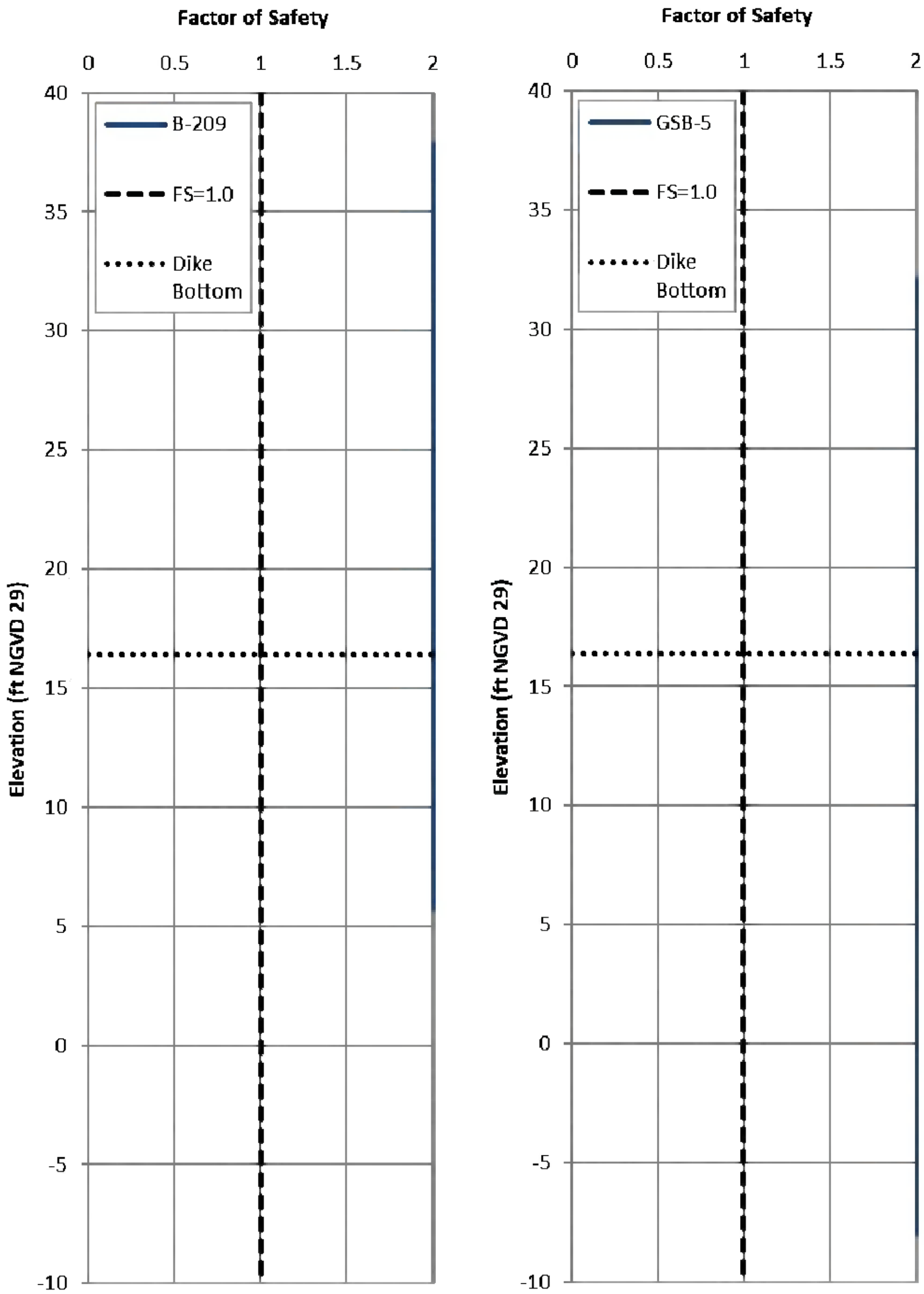


Figure 12. Liquefaction Results for Dike and Foundation Soils for B-209 and GSB-5

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings as provided in Table 5.

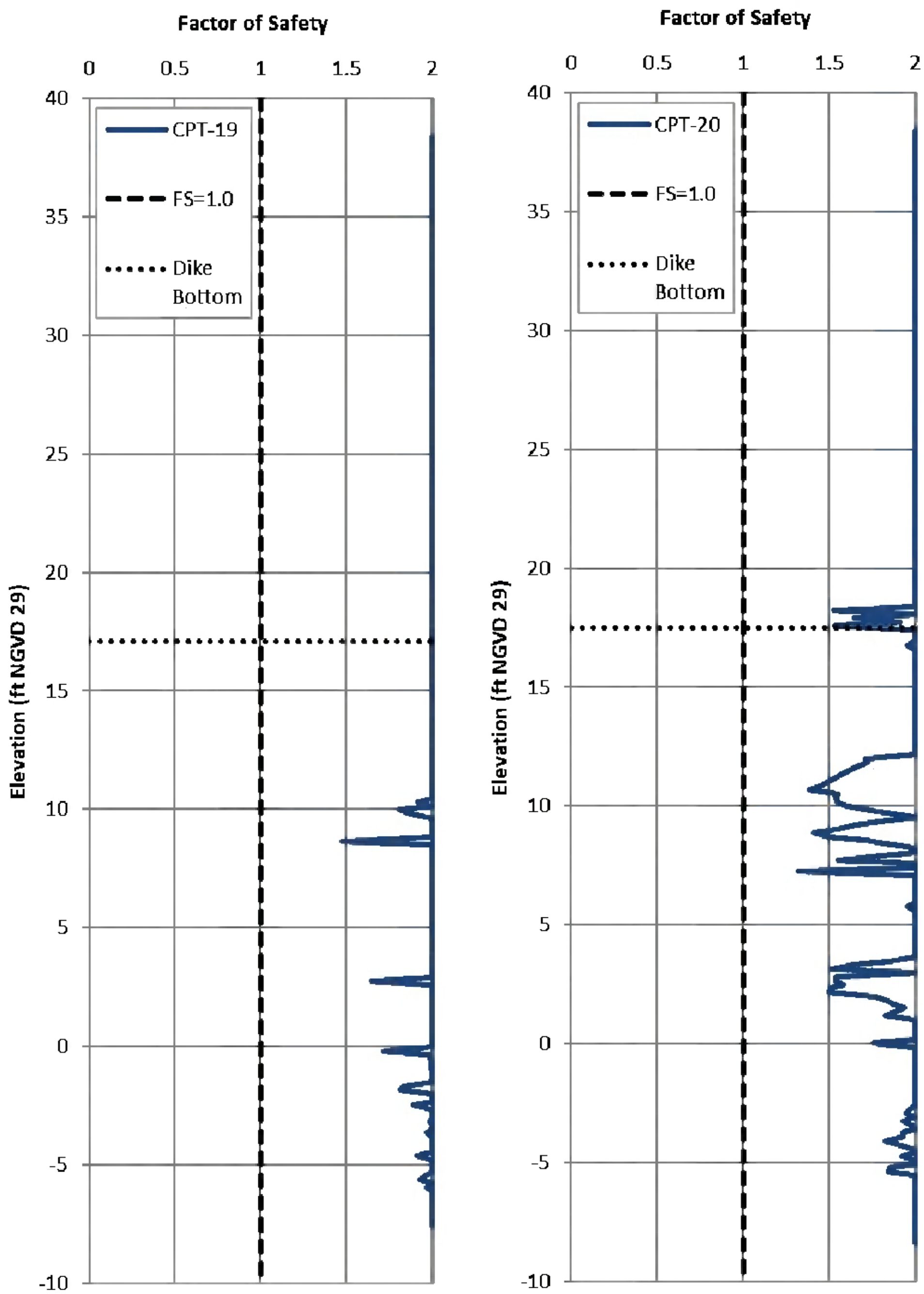


Figure 13. Liquefaction Results for Dike and Foundation Soils for CPT-19 and CPT-20

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings as provided in Table 5.

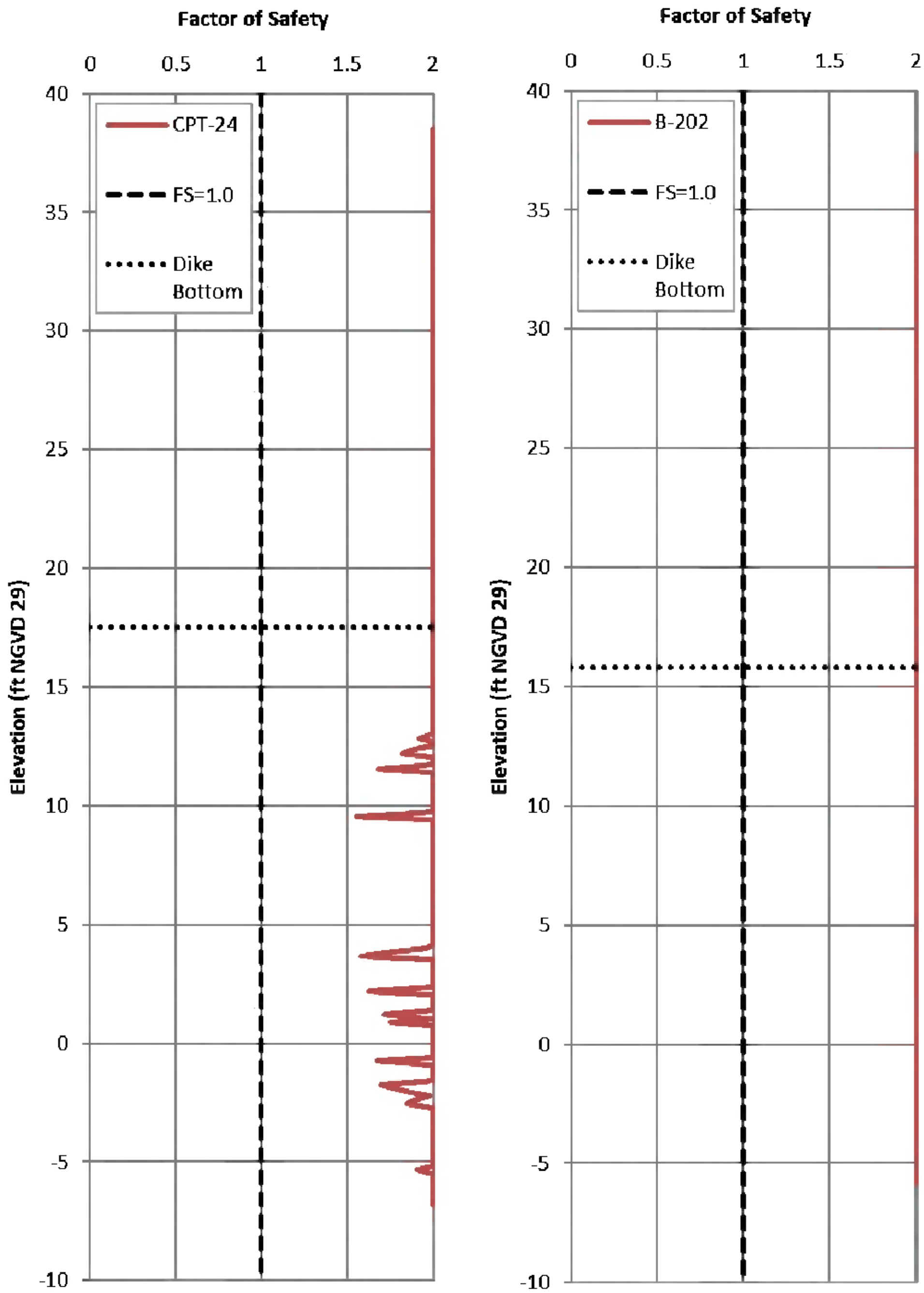


Figure 14. Liquefaction Results for Dike and Foundation Soils for CPT-24 and B-202

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings as provided in Table 5.

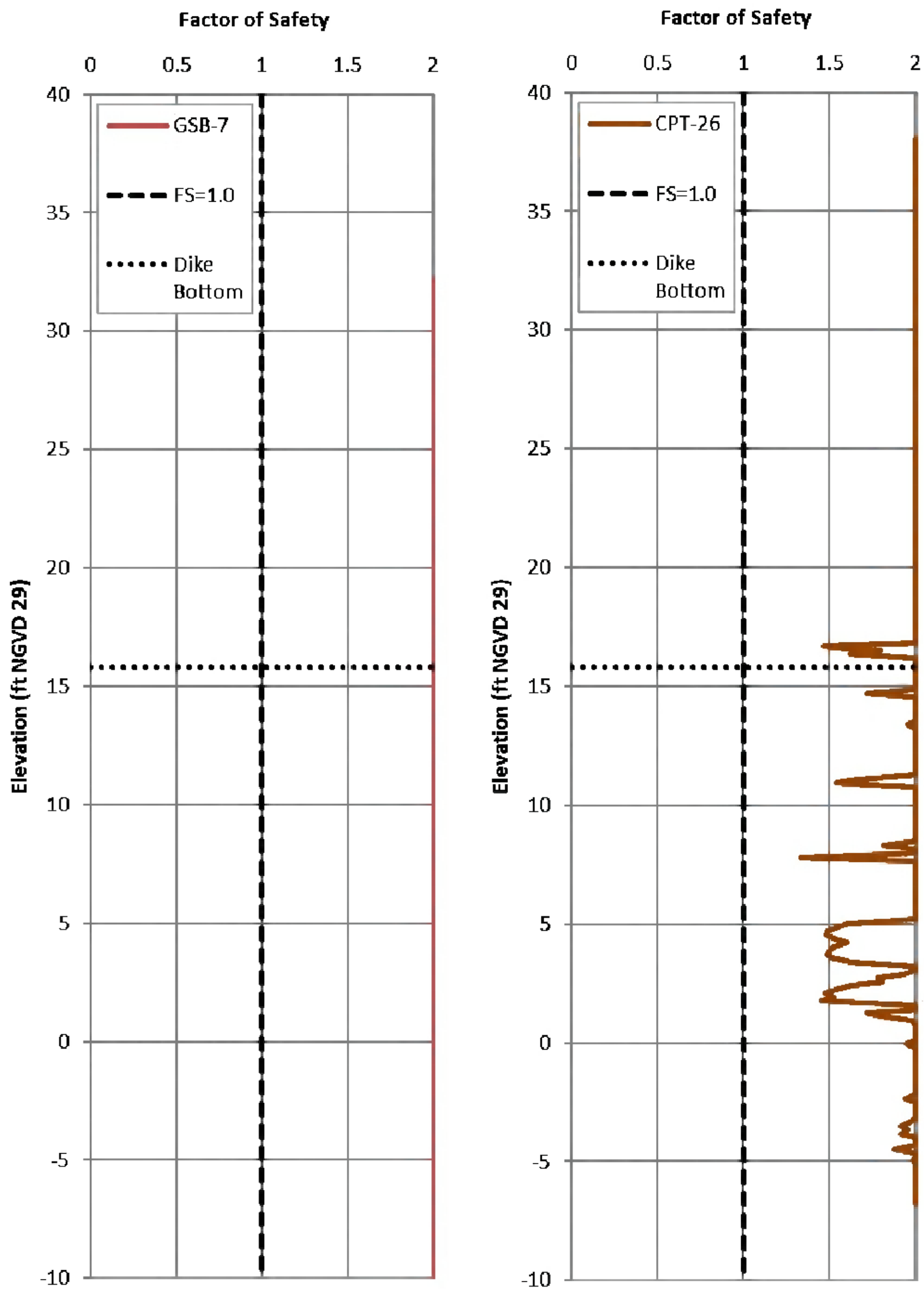


Figure 15. Liquefaction Results for Dike and Foundation Soils for GSB-7 and CPT-26

Notes:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical boring as provided in Table 5.

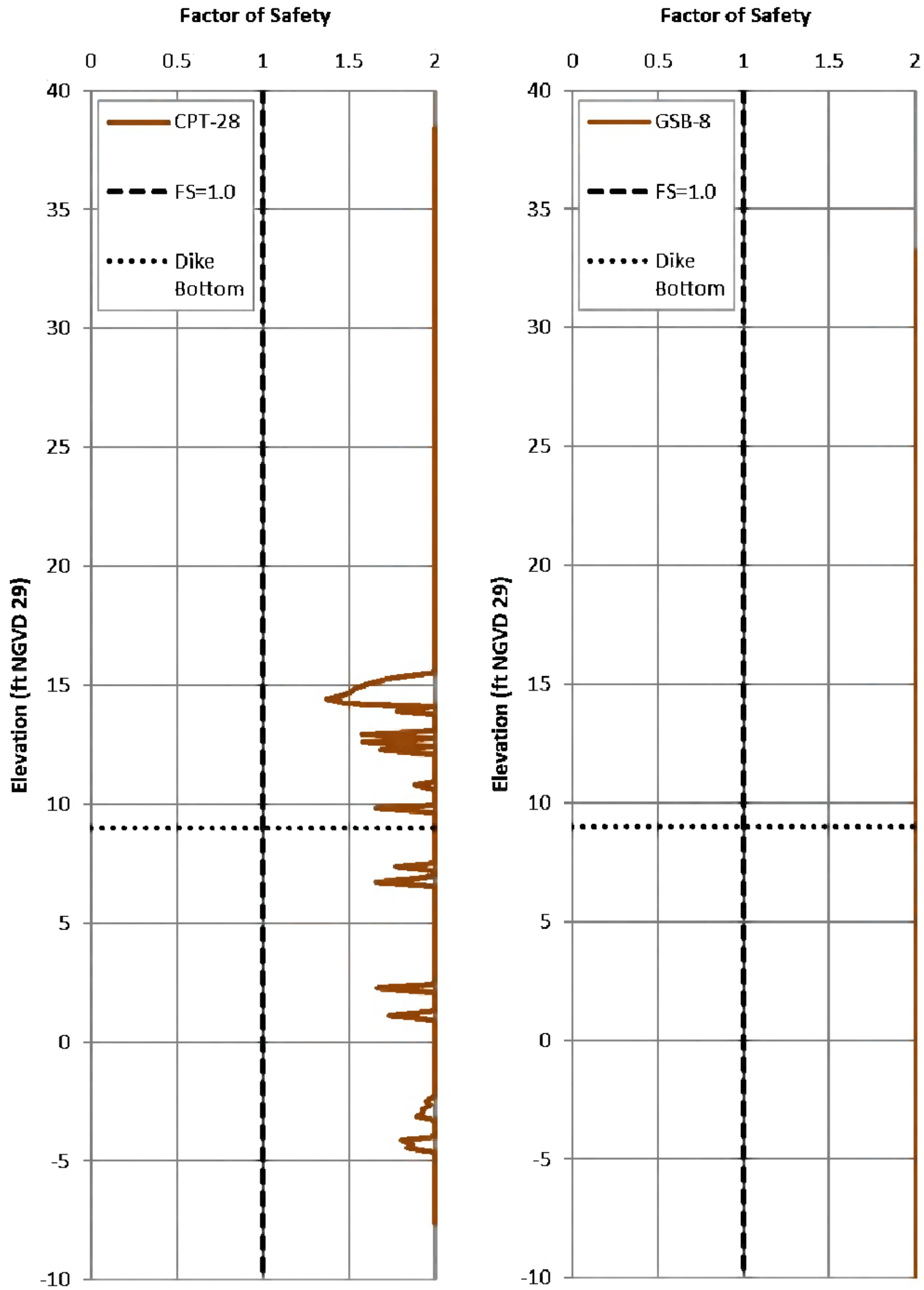


Figure 16. Liquefaction Results for Dike and Foundation Soils for CPT-28 and GSB-8

Notes:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the surface elevation of historical borings as provided in Table 5.

Appendix 1

MathCAD® Example Calculation

SPT - Based Liquefaction Analysis

$BoringID := \text{"B\_206"}$ $\qquad tsf := \frac{tonf}{ft^2}$ $\qquad kPa := \frac{1}{95.760518} \cdot tsf$

Site Parameters:

Age Correction Factor of Pleistocene Soils: $K_{dr} := 1.2$

Earthquake Magnitude: $M := 7.3$

Site Response Profile: $Prof := \text{"Profile1"}$

$$CyclicStress := \left\| \begin{array}{l} \text{if } Prof = \text{"Profile1"} \\ \quad \left\| \text{READEXCEL}(\text{"Summary Profile\_1.xlsx"}) \right. \end{array} \right\|$$

SPT-Specific Data:

Import the SPT-Specific Data in the form of Depth, Blow Count, Visual Classification ("Sand-Like"/"Clay-Like"), fines content, and USCS Classification. Upper two rows contain the headers and units for each field:

$Full := \text{READEXCEL}(\text{concat}(BoringID, \text{".xlsx"}))$

$Data := \text{submatrix}(Full, 2, \text{rows}(Full) - 1, 0, \text{cols}(Full) - 1)$

$depth := Data^{\langle 0 \rangle} \cdot ft$ $\qquad N_{blows} := Data^{\langle 1 \rangle}$ $\qquad Class := Data^{\langle 2 \rangle}$

$Fines := Data^{\langle 3 \rangle}$ $\qquad USCS := Data^{\langle 4 \rangle}$

Investigation Information:

Ground Surface Elevation: $Elevation := 38.9 \cdot ft$ NGVD29

Groundwater Depth at Time of Boring (TOB): $GWT_b := 12.0 \cdot ft$ bgs

Groundwater Depth at Time of Analysis (TOA): $GWT := 12.0 \cdot ft$ bgs

Boring Diameter: $Diameter := 4$ *inches*

Bottom of Holocene Elevation / Bottom of Dike Fill Soils: $Elev_h := 19.0 \cdot ft$ NGVD29

Energy Calibration: $ER := 70$ % (SCI, 2014)

Sampling Method: $C_S := 1.0$

$RodDepth := depth + 5 \cdot ft$ (Assume 5 ft of rod stick up during SPT test)

Compute Calibration Factors:

$$C_E := \frac{ER}{60}$$

$$C_B := \left\| \begin{array}{l} \text{if } Diameter \le 4.0 \\ \quad \| 1.0 \\ \text{also if } 4.0 < Diameter < 6.0 \\ \quad \| 1.05 \\ \text{else} \\ \quad \| 1.15 \end{array} \right.$$

$$C_R := \left\| \begin{array}{l} \text{for } i \in 0..\text{rows}(depth) - 1 \\ \quad \left\| \begin{array}{l} \text{if } RodDepth_i \le 13 \cdot ft \\ \quad \| rod_i \leftarrow 0.75 \\ \text{also if } 13 \cdot ft < RodDepth_i \le 20 \cdot ft \\ \quad \| rod_i \leftarrow 0.85 \\ \text{also if } 20 \cdot ft < RodDepth_i \le 33 \cdot ft \\ \quad \| rod_i \leftarrow 0.95 \\ \text{else} \\ \quad \| rod_i \leftarrow 1 \end{array} \right. \\ rod \end{array} \right.$$

Compute N60:

$$N_{60} := \left\| \begin{array}{l} \text{for } i \in 0..\text{rows}(depth) - 1 \\ \quad \| x_i \leftarrow C_B \cdot C_E \cdot C_S \cdot N_{blows_i} \cdot C_{R_i} \\ x \end{array} \right.$$

Compute CN:

Develop Representative Unit Weight Profile:

Unit weight values to be assigned based on density and material class:

<u>*Adjust according to specific site conditions*</u>

1. Coal Combustion Residuals $\gamma_1 := 90 \cdot pcf$
2. Loose Sands (Nblows <10) $\gamma_2 := 105 \cdot pcf$

3. Medium Dense Sands (10<Nblows <30) $\gamma_3 := 115 \cdot pcf$

4. Dense Sands $\gamma_4 := 120 \cdot pcf$

5. Soft Clays $\gamma_5 := 100 \cdot pcf$

6. Chicora Member $\gamma_6 := 130 \cdot pcf$

7. Williamsburg Formation Clay $\gamma_7 := 105 \cdot pcf$

Relate Depth to Elevation to Screen for Williamsburg Formation Clay

$Elev := (Elevation - depth)$ $\quad WMElev := -8 \cdot ft$ (Approx. Top of Williamsburg Formation Clay)

```
Class_2 := | for i ∈ 0 .. rows(depth) - 1
           |   y_i ← 7
           |   if Class_i = "SILT"
           |     y_i ← 1
           |   if Class_i = "SAND" ∧ N_blows_i ≤ 10
           |     y_i ← 2
           |   if Class_i = "SAND" ∧ 10 < N_blows_i ≤ 30
           |     y_i ← 3
           |   if Class_i = "SAND" ∧ 30 < N_blows_i
           |     y_i ← 4
           |   if Class_i = "CLAY" ∧ Elev_i > WMElev
           |     y_i ← 5
           |   if Class_i = "CHICORA"
           |     y_i ← 6
           | y
```

Unit Weight Based on Soil Classification:

$$\gamma_{fin} := \left\| \begin{array}{l} \text{for } i \in 0..\text{rows}(depth) - 1 \\ \quad \left\| \begin{array}{l} \text{for } m \in 1..7 \\ \quad \left\| \begin{array}{l} \text{if } Class_{2_i} = m \\ \quad \left\| \gamma2_i \leftarrow \gamma_m \right. \end{array} \right. \end{array} \right. \\ \gamma2 \end{array} \right.$$

$\gamma := \gamma_{fin}$ $\gamma_{water} := 62.4 \cdot pcf$

$i := 0..\text{rows}(depth) - 1$

Final Static Pore Pressure Calculation
at Time of Boring:

$$u_{0b_i} := \left\| \begin{array}{l} \text{if } depth_i > GWT_b \\ \quad \left\| \left(depth_i - GWT_b\right) \cdot \gamma_{water} \right. \\ \text{else} \\ \quad \left\| 0 \right. \end{array} \right.$$

Final Total and Effective Overburden Pressure:

$\sigma_{v0b_0} := depth_0 \cdot \gamma_0$

$$\sigma_{v0b_i} := \left\| \begin{array}{l} \text{if } i > 0 \\ \quad \left\| \left(depth_i - depth_{i-1}\right) \cdot \left(\dfrac{\gamma_i + \gamma_{i-1}}{2}\right) + \sigma_{v0b_{i-1}} \right. \\ \text{else} \\ \quad \left\| depth_0 \cdot \gamma_0 \right. \end{array} \right.$$

$\sigma_{v0beff} := \sigma_{v0b} - u_{0b}$

Calculation of CNL (for Liquefaction)

$$C_{NLit} := \left\| \begin{array}{l} c \leftarrow 0 \\ \text{"initial CN"} \\ \text{for } i \in 0..\text{rows}(depth) - 1 \\ \quad \left\| C_{N_i} \leftarrow 1.7 \right. \\ \text{for } i \in 0..\text{rows}(depth) - 1 \\ \quad \left\| \begin{array}{l} \text{while } c < 600 \\ \quad \left\| \begin{array}{l} N_{160L_i} \leftarrow C_{N_i} \cdot N_{60_i} \\ C_{N_i} \leftarrow min\left(1.7, \left(\dfrac{1 \cdot atm}{\sigma_{v0beff_i}}\right)^{\left(0.784 - 0.0768 \cdot \sqrt{min\left(46, N_{160L_i}\right)}\right)}\right) \\ c \leftarrow c + 1 \end{array} \right. \\ c \leftarrow 0 \end{array} \right. \\ \left[C_N \ N_{160L} \right] \end{array} \right.$$

$$C_{NL} := \left(C_{NLit}^{\langle 0 \rangle}\right)_0$$

Compute (N1)60:

$$N_{160_i} := C_{NL_i} \cdot N_{60_i}$$

Compute Clean Sand-Corrected (N1)60-L (For Liquefaction) [SCDOT 2019, Eq. 13-15]:

$$\Delta N_{160L} := \left\| \begin{array}{l} \text{for } i \in 0..\text{rows}(depth) - 1 \\ \quad \left\| x_i \leftarrow min\left(5.5, \exp\left(1.63 + \left(\dfrac{9.7}{\left(Fines_i + 0.01\right)}\right) - \left(\dfrac{15.7}{\left(Fines_i + 0.01\right)}\right)^2\right)\right) \right. \\ x \end{array} \right.$$

$$N_{160cs_i} := N_{160_i} + \Delta N_{160L_i}$$

Compute the CRR (Mw=7.5, 1 atm) based on the SPT Values [SCDOT 2019, Eq. 13-16]:

Cyclic Resistance Ratio (CRR):

$$CRR1_i := \exp\left(\left(\frac{N_{160cs_i}}{14.1}\right) + \left(\frac{N_{160cs_i}}{126}\right)^2 - \left(\frac{N_{160cs_i}}{23.6}\right)^3 + \left(\frac{N_{160cs_i}}{25.4}\right)^4 - 2.8\right)$$

Overburden Correction Factor (Kσ):

$$C_{\sigma_i} := min\left(\frac{1}{18.9 - 2.55 \cdot \left(min\left(N_{160cs_i}, 37\right)\right)^{0.50}}, 0.3\right)$$

$$K_{\sigma_i} := min\left(1 - C_{\sigma_i} \cdot \ln\left(\frac{\sigma_{v0beff_i}}{2117 \cdot psf}\right), 1.1\right)$$

Corrected CRR: $CRR2_i := CRR1_i \cdot K_{\sigma_i}$

Magnitude Scaling Factor (MSF) [SCDOT 2019, Eq. 13-11]:

$$MSF_i := min\left(1.80, 6.9 \cdot \exp(-0.25 \cdot M) - 0.058\right)$$

Corrected CRR: $CRR3_i := CRR2_i \cdot MSF_i$

Age Correction Factor for Pleistocene Sands (Kdr) [SCDOT 2019, Section 13.9.5]:

<u>Kdr is only applicbale for Sands that are of Pleistocene-Age or older (e.g., foundation soils)</u>

$$CRR_{final_i} := \begin{array}{|l} \text{if } Class\_{}_i = \text{“SAND”} \wedge Elev_i < Elev_h \\ \quad \| \; CRR3_i \cdot K_{dr} \\ \text{else} \\ \quad \| \; CRR3_i \end{array}$$

Compute the CSR for the Soil Profile:

Final Static Pore Pressure Calculation at Time of Analysis:

$$u_{0_i} := \begin{array}{|l} \text{if } depth_i > GWT \\ \quad \| \; \left(depth_i - GWT\right) \cdot \gamma_{water} \\ \text{else} \\ \quad \| \; 0 \end{array}$$

Final Total and Effective Overburden Pressure at Time of Analysis:

$$\sigma_{v0_0} := depth_0 \cdot \gamma_0$$

$$\sigma_{v0_i} := \begin{array}{|l} \text{if } i > 0 \\ \quad \| \; \left(depth_i - depth_{i-1}\right) \cdot \left(\frac{\gamma_i + \gamma_{i-1}}{2}\right) + \sigma_{v0_{i-1}} \\ \text{else} \\ \quad \| \; depth_0 \cdot \gamma_0 \end{array}$$

$$\sigma_{v0eff_i} := \sigma_{v0_i} - u_{0_i}$$

$$\tau_{cyc} := \text{submatrix}\left(CyclicStress, 1, \text{rows}\left(CyclicStress\right) - 1, 1, 1\right) \cdot psf$$

$$d_{cyc} := \text{submatrix}\left(CyclicStress, 1, \text{rows}\left(CyclicStress\right) - 1, 0, 0\right) \cdot ft$$

$$\tau_{max} := \text{linterp}\left(d_{cyc}, \tau_{cyc}, depth\right)$$

$$CSR_i := \frac{0.65 \cdot \tau_{max_i}}{\sigma_{v0eff_i}}$$

Compute Factor of Safety:

$$FS_i := \begin{Vmatrix} \text{if } Class_i = \text{"CHICORA"} \\ \quad \Vert\ 2.01 \\ \text{also if } Class_i = \text{"CLAY"} \\ \quad \Vert\ 2.01 \\ \text{also if } depth_i < GWT \\ \quad \Vert\ 2.01 \\ \text{else} \\ \quad \Vert\ min\left(\dfrac{CRR_{final_i}}{CSR_i}, 2.01\right) \end{Vmatrix}$$

Assume Chicora and Williamsburg Clay layers do NOT Liquefy

Export Results:

$$Headers := \text{augment}\left(\text{"Depth"}, \text{"Elevation"}, \text{"N160"}, \text{"Class"}, \text{"FScyclic"}\right)$$

$$Units := \text{augment}\left(\text{"ft"}, \text{"ft NGVD29"}, \text{"-"}, \text{"-"}, \text{"-"}\right)$$

$$Export := \text{augment}\left(\frac{depth}{ft}, \frac{Elev}{ft}, N_{160}, Class, FS\right)$$

$$Export2 := \text{stack}\left(Headers, Units, Export\right)$$

$$FileName := \text{concat}\left(BoringID, \text{"\_Results"}, \text{".xlsx"}\right)$$

$$Export3 := \text{WRITEEXCEL}\left(FileName, Export2\right)$$

CPT - Based Liquefaction Analysis

$BoringID := \text{"CPT\_1"}$ $\qquad$ $tsf := 2116 \cdot psf$

Site Parameters:

Age Correction Factor of Pleistocene Soils: $K_{dr} := 1.2$

Earthquake Magnitude: $M := 7.3$

Site Response Profile: $Prof := \text{"Profile1"}$

$$CyclicStress := \left\| \begin{array}{l} \text{if } Prof = \text{"Profile1"} \\ \quad \| \text{READEXCEL}(\text{"Summary Profile\_1.xlsx"}) \end{array} \right\|$$

CPT-Specific Data:

Import the CPT-Specific Data in the form of depth (ft), tip resistance (tsf), sleeve friction (tsf), porepressure (tsf), and fines content profile (%) with headers and units:

$Full := \text{READEXCEL}(\text{concat}(BoringID, \text{".xlsx"}))$

$Data := \text{submatrix}(Full, 2, \text{rows}(Full) - 1, 0, \text{cols}(Full) - 1)$

$depth := Data^{\langle 0 \rangle} \cdot ft$ $\qquad$ $qc := Data^{\langle 1 \rangle} \cdot tsf$ $\qquad$ $f_s := Data^{\langle 2 \rangle} \cdot tsf$

$u_2 := Data^{\langle 3 \rangle} \cdot tsf$ $\qquad$ $Fines := Data^{\langle 4 \rangle}$

Simple counter used in the Algorithm: $i := 0 .. \text{rows}(Data) - 1$

Tip net area ratio (correction applied when converting .cpt to .xls format): $a := 1$

Investigation Information:

Ground Surface Elevation: $Elevation := 39.38 \cdot ft$ NGVD 29

Groundwater Depth at Time of Boring (TOB): $GWT_b := 10.0 \cdot ft$ bgs

Groundwater Depth at Time of Analysis (TOA): $GWT := 10.0 \cdot ft$ bgs

Bottom of Holocene Elevation / Bottom of Dike Fill Soils: $Elev_h := 25.2 \cdot ft$ NGVD 29

Profile with Elevations: $Elev := Elevation - depth$

Initial Unit Weight Estimates to be Used with Robertson and Campanella (1983):

<u>*Adjust according to specific site conditions*</u>

1. Sand $\gamma_1 := 115 \cdot pcf$
2. Silty Sand $\gamma_2 := 105 \cdot pcf$
3. Sandy silt and silt $\gamma_3 := 100 \cdot pcf$
4. Silty clay/Clayey silt $\gamma_4 := 90 \cdot pcf$
5. Clay $\gamma_5 := 90 \cdot pcf$

Water $\gamma_{water} := 62.4 \cdot pcf$

Tip Resistance Back-Calculated from qt and Tip Net Area Ratio a Provided in the Original Data: $qt_i := qc_i - (1 - a) \cdot u_{2_i}$

Average Friction Ratio: $Rf_i := \left(\left(\frac{f_{s_i}}{qt_i}\right) \cdot 100\right)\%$

Robertson and Campanella 1983 Plot data:

Sand-Silty Sand $S01 := \text{submatrix}(\text{READPRN}(\text{"Robertson1983.txt"}), 0, 11, 0, 1)$

Silty Sand-Silts $S02 := \text{submatrix}(\text{READPRN}(\text{"Robertson1983.txt"}), 0, 12, 2, 3)$

Silts-Silty Clay $S03 := \text{submatrix}(\text{READPRN}(\text{"Robertson1983.txt"}), 0, 18, 4, 5)$

Clay $S04 := \text{submatrix}(\text{READPRN}(\text{"Robertson1983.txt"}), 0, 19, 6, 7)$

Linear interpolation used to evaluate Qt as a function of depth based on plot lines:

$s01(x) := \text{linterp}\left(S01^{\langle 0 \rangle}, S01^{\langle 1 \rangle}, x\right)$ $s02(x) := \text{linterp}\left(S02^{\langle 0 \rangle}, S02^{\langle 1 \rangle}, x\right)$

$s03(x) := \text{linterp}\left(S03^{\langle 0 \rangle}, S03^{\langle 1 \rangle}, x\right)$ $s04(x) := \text{linterp}\left(S04^{\langle 0 \rangle}, S04^{\langle 1 \rangle}, x\right)$

Initial Estimate of Unit Weight Based on Robertson 1983 Soil Classification:

```
class_1983 := | for i ∈ 0..rows(qt) - 1
              |   class_i ← 5
              |   if qt_i / (100·kPa) ≥ s04(Rf_i)
              |     class_i ← 4
              |   if qt_i / (100·kPa) ≥ s03(Rf_i)
              |     class_i ← 3
              |   if qt_i / (100·kPa) ≥ s02(Rf_i)
              |     class_i ← 2
              |   if qt_i / (100·kPa) ≥ s01(Rf_i)
              |     class_i ← 1
              | class
```

```
γ1 := | for i ∈ 0..rows(qt) - 1
      |   for m ∈ 1..5
      |     if class_1983_i = m
      |       γ1_i ← γ_m
      | γ1
```

Refined Soil Classification Using Robertson and Cabal 2010:

Static Pore Pressures at time of Sounding:

```
u_0b_i := | if depth_i > GWT_b
          |   (depth_i - GWT_b)·γ_water
          | else
          |   0
```

Total and Effective Overburden Pressure:

$$\sigma_{v0b_0} := depth_0 \cdot \gamma 1_0$$

```
σ_v0b_i := | if i > 0
           |   (depth_i - depth_(i-1))·((γ1_i + γ1_(i-1)) / 2) + σ_v0b_(i-1)
           | else
           |   depth_i·γ1_0
```

$$\sigma_{v0beff_i} := \sigma_{v0b_i} - u_{0b_i}$$

Normalized Parameters:

$$Q_{t_i} := \frac{qt_i - \sigma_{v0b_i}}{\sigma_{v0beff_i}} \qquad B_{q_i} := \frac{u_{2_i} - u_{0b_i}}{qt_i - \sigma_{v0b_i}} \qquad F_{r_i} := \frac{f_{s_i}}{qt_i - \sigma_{v0b_i}} \cdot 100$$

<u>Unit Weight Values to be Assigned to Robertson (1990) Classification:</u>

<u>Adjust according to specific site conditions</u>

1. Sensitive, fine grained $\gamma_1 := 85 \cdot pcf$
2. Organic Soils-peat to Clay $\gamma_2 := 100 \cdot pcf$
3. Clay mixtures $\gamma_3 := 100 \cdot pcf$
4. Silt mixtures $\gamma_4 := 100 \cdot pcf$
5. Sand mixtures $\gamma_5 := 110 \cdot pcf$
6. Sands $\gamma_6 := 120 \cdot pcf$
7. Gravelly sand to sand $\gamma_7 := 125 \cdot pcf$
8. Very stiff sand to clayey sand $\gamma_8 := 105 \cdot pcf$
9. Very stiff fine grained $\gamma_9 := 105 \cdot pcf$

Compute Soil Behavior Index (Ic) Corresponding to Initial Unit Weight Classification:

$$I_{c_i} := \left(\left(3.47 - \log\left(Q_{t_i}\right) \right)^2 + \left(\log\left(F_{r_i}\right) + 1.22 \right)^2 \right)^{.5}$$

Soil Classification for Robertson (2010) (updated from Robertson, 1990):

$$class_{2010} := \left\| \begin{array}{l} \text{for } i \in 0..\text{rows}(Q_t) - 1 \\ \quad \left\| \begin{array}{l} class_i \leftarrow 2 \\ \text{if } 2.95 < I_{c_i} \le 3.6 \\ \quad \| \; class_i \leftarrow 3 \\ \text{if } 2.60 < I_{c_i} \le 2.95 \\ \quad \| \; class_i \leftarrow 4 \\ \text{if } 2.05 < I_{c_i} \le 2.60 \\ \quad \| \; class_i \leftarrow 5 \\ \text{if } 1.31 < I_{c_i} \le 2.05 \\ \quad \| \; class_i \leftarrow 6 \\ \text{if } I_{c_i} \le 1.31 \\ \quad \| \; class_i \leftarrow 7 \end{array} \right. \\ class \end{array} \right.$$

Unit Weight based on Soil Classification:

$$\gamma_{fin} := \left\| \begin{array}{l} \text{for } i \in 0..\text{rows}(Q_t) - 1 \\ \quad \left\| \begin{array}{l} \text{for } m \in 1..9 \\ \quad \left\| \begin{array}{l} \text{if } class_{2010_i} = m \\ \quad \| \; \gamma 2_i \leftarrow \gamma_m \end{array} \right. \end{array} \right. \\ \gamma 2 \end{array} \right.$$

$\gamma := \gamma_{fin}$ $\qquad$ $class := class_{2010}$

Final Static Pore Pressure Calculation for CPT Interpretation:

$$u_{0b_i} := \left\| \begin{array}{l} \text{if } depth_i > GWT_b \\ \quad \| \; (depth_i - GWT_b) \cdot \gamma_{water} \\ \text{else} \\ \quad \| \; 0 \end{array} \right.$$

Final Total and Effective Overburden Pressure for CPT Interpretation:

$$\sigma_{v0b_i} := \begin{array}{l} \text{if } i > 0 \\ \quad \left(depth_i - depth_{i-1}\right) \cdot \left(\dfrac{\gamma_i + \gamma_{i-1}}{2}\right) + \sigma_{v0b_{i-1}} \\ \text{else} \\ \quad depth_0 \cdot \gamma_0 \end{array}$$

$$\sigma_{v0beff_i} := \sigma_{v0b_i} - u_{0b_i}$$

Final Normalized Parameters:

$$Q_{t_i} := \frac{qt_i - \sigma_{v0b_i}}{\sigma_{v0beff_i}} \qquad Q_i := \frac{qt_i - \sigma_{v0b_i}}{\sigma_{v0beff_i}} \qquad B_{q_i} := \frac{u_{2_i} - u_{0b_i}}{qt_i - \sigma_{v0b_i}} \qquad F_{r_i} := \frac{f_{s_i}}{qt_i - \sigma_{v0b_i}} \cdot 100$$

Recompute Soil Behavior Index (Ic) corresponding to Final Unit Weight Classification:

$$I_{c_i} := \left(\left(3.47 - \log\left(Q_{t_i}\right)\right)^2 + \left(\log\left(F_{r_i}\right) + 1.22\right)^2\right)^{.5}$$

Corrected Normalized CPT Sounding:

Overburden Corrected Tip Resistance:

$$q_{c1\_it} := \begin{array}{l} c \leftarrow 0 \\ \text{"initial CN"} \\ \text{for } i \in 0 .. \text{rows}(qt) - 1 \\ \quad C_{N_i} \leftarrow 1.7 \\ \text{for } i \in 0 .. \text{rows}(qt) - 1 \\ \quad \text{while } c < 500 \\ \quad\quad q_{c1_i} \leftarrow C_{N_i} \cdot qt_i \\ \quad\quad q_{c1N_i} \leftarrow \dfrac{q_{c1_i}}{1 \cdot atm} \\ \quad\quad C_{N_i} \leftarrow min\left(1.7, \left(\dfrac{1 \cdot atm}{\sigma_{v0beff_i}}\right)^{1.338 - 0.249 \cdot \left(\max\left(21, min\left(q_{c1N_i}, 254\right)\right)\right)^{0.264}}\right) \\ \quad\quad c \leftarrow c + 1 \\ \quad c \leftarrow 0 \\ \left[\dfrac{q_{c1}}{psf} \;\; q_{c1N}\right] \end{array}$$

$$q_{c1} := \left(q_{c1\_it}{}^{\langle 0 \rangle}\right)_0 \cdot psf \qquad q_{c1N} := \left(q_{c1\_it}{}^{\langle 1 \rangle}\right)_0$$

Compute the CRR (Mw=7.5, 1 atm) based on the CPT Data:

Cyclic Resistance Ratio (CRR):

$i := 0 .. \text{rows}(qc) - 1$

Correction Factor for Soils with Fines:

$$\Delta q_{c1N_i} := \left(11.9 + \frac{q_{c1N_i}}{14.6}\right) \cdot \exp\left(1.63 - \frac{9.7}{Fines_i + 2} - \left(\frac{15.7}{Fines_i + 2}\right)^2\right)$$

Equivalent Clean Sand Corrected Tip Resistance: $q_{c1Ncs_i} := q_{c1N_i} + \Delta q_{c1N_i}$

$$CRR_i := \begin{cases} \exp\left(\frac{q_{c1Ncs_i}}{113} + \left(\frac{q_{c1Ncs_i}}{1000}\right)^2 - \left(\frac{q_{c1Ncs_i}}{140}\right)^3 + \left(\frac{q_{c1Ncs_i}}{137}\right)^4 - 2.8\right) & \text{if } I_{c_i} \le 2.60 \wedge q_{c1Ncs_i} < 211 \\ 2.0 & \text{also if } I_{c_i} \le 2.60 \wedge q_{c1Ncs_i} > 211 \\ 2.0 & \text{else} \end{cases}$$

Overburden Correction Factor (Kσ) for Sands [SCDOT 2019, Eq. 13-22, 13-25]:

$$C_{\sigma_i} := min\left(\frac{1}{37.3 - 8.27 \cdot \left(min\left(q_{c1Ncs_i}, 211\right)\right)^{0.264}}, 0.3\right)$$

$$K_{\sigma_i} := \begin{cases} min\left(1 - C_{\sigma_i} \cdot \ln\left(\frac{\sigma_{v0beff_i}}{1 \cdot tsf}\right), 1.1\right) & \text{if } I_{c_i} \le 2.60 \\ 1.0 & \text{else} \end{cases}$$

Corrected CRR: $CRR1_i := CRR_i \cdot K_{\sigma_i}$

Magnitude Scaling Factor (MSF) [SCDOT 2019, Eq. 13-11]:

MSF is dependent on material type and for cyclic softening calculations, two MSF correlations are applicable

$$MSF_i := min\left(1.80, 6.9 \cdot \exp(-0.25 \cdot M) - 0.058\right)$$

Corrected CRR: $CRR2_i := CRR1_i \cdot MSF_i$

Age Correction Factor for Pleistocene Sands (Kdr) [SCDOT 2019, Section 13.9.5]:

Kdr is only applicbale for Sands that are of Pleistocene-Age or older (e.g., foundation soils)

$$CRR_{final_i} := \begin{cases} CRR2_i \cdot K_{dr} & \text{if } I_{c_i} \le 2.60 \land Elev_i < Elev_h \\ CRR2_i & \text{else} \end{cases}$$

Compute the CSR for the Soil Profile:

Final Static Pore Pressure Calculation at Time of Analysis:

$$u_{0_i} := \begin{cases} \left(depth_i - GWT\right) \cdot \gamma_{water} & \text{if } depth_i > GWT \\ 0 & \text{else} \end{cases}$$

Final Total and Effective Overburden Pressure at Time of Analysis:

$$\sigma_{v0_0} := depth_0 \cdot \gamma_0$$

$$\sigma_{v0_i} := \begin{cases} \left(depth_i - depth_{i-1}\right) \cdot \left(\dfrac{\gamma_i + \gamma_{i-1}}{2}\right) + \sigma_{v0_{i-1}} & \text{if } i > 0 \\ depth_0 \cdot \gamma_0 & \text{else} \end{cases}$$

$$\sigma_{v0eff_i} := \sigma_{v0_i} - u_{0_i}$$

$$\tau_{cyc} := \text{submatrix}\left(CyclicStress, 1, \text{rows}\left(CyclicStress\right) - 1, 1, 1\right) \cdot psf$$

$$d_{cyc} := \text{submatrix}\left(CyclicStress, 1, \text{rows}\left(CyclicStress\right) - 1, 0, 0\right) \cdot ft$$

$$\tau_{max} := \text{linterp}\left(d_{cyc}, \tau_{cyc}, depth\right)$$

$$CSR_i := \frac{0.65 \cdot \tau_{max_i}}{\sigma_{v0eff_i}}$$

Compute Factor of Safety:

$$FS_i := \begin{array}{l} \text{if } depth_i < GWT_b \\ \quad \| \; 2.00 \\ \text{else} \\ \quad \| \; min\left(\dfrac{CRR_{final_i}}{CSR_i}, 2.00\right) \end{array}$$

Export Results:

$Headers := \text{augment}(\text{“Depth”}, \text{“Elevation”}, \text{“qc1N”}, \text{“SBT Index”}, \text{“FScyclic”})$

$Units := \text{augment}(\text{“ft”}, \text{“ft NGVD29”}, \text{“-”}, \text{“-”}, \text{“-”})$

$$Export := \text{augment}\left(\frac{depth}{ft}, \frac{Elev}{ft}, q_{c1N}, I_c, FS\right)$$

$Export2 := \text{stack}(Headers, Units, Export)$

$FileName := \text{concat}(BoringID, \text{“\_Results”}, \text{“.xlsx”})$

$Export3 := \text{WRITEEXCEL}(FileName, Export2)$

ATTACHMENT 5
SAFETY FACTOR ASSESSMENT

Written by:	**Z. Li/C. Carson**	Date:	**11/04/2021**	Reviewed by:	**C. Carlson/B. Gin**	Date:	**11/08/2021**
Client:	**Santee Cooper**	Project:	**Winyah Generating Station**	Project/ Proposal No.:	**GC8100**	Task No.:	**03**

SAFETY FACTOR ASSESSMENT: SLURRY POND

INTRODUCTION

This calculation package was prepared as Attachment 5 to the *2021 Periodic Safety Factor Assessment: Slurry Pond* (2021 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 Attachment 4: *Liquefaction Potential Analysis: Slurry Pond* (Liquefaction Package) of the 2021 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.

METHODOLOGY

Static Slope Stability

Global slope stability analyses were performed using Spencer's method (Spencer, 1973), as implemented in the computer program SLIDE®, version 6.039 (Rocscience, 2016). 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 Attachment 3: *Seismic Hazard Evaluation and Site Response Analysis: Slurry Pond* (Site Response Package) to the 2021 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.

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).

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 15-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. The locations of the five cross sections analyzed in the 2016 Safety Factor Assessment are depicted in Figure 1.

For the 2021 Safety Factor Assessment Report, only the critical cross sections identified in the 2016 Safety Factor Assessment are analyzed. 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 2021 Safety Factor Assessment Report. Updated topographic survey data from September 2021 were also incorporated into these cross sections.

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 Surcharge 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. The operating level of 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. Santee Cooper provided water level measurements from wells located outside the downstream toe of the Slurry Pond perimeter dikes. The recorded water levels in these wells have generally been steady over the last five years. Based on the review of the limited water level measurements adjacent to the Slurry Pond perimeter dikes, the water level within the perimeter dike may be similar to the water level used for the 2016 Safety Factor Assessment. Based on the provided water level data from wells located outside the downstream toe, water levels at the toe were determined as identical to those in the 2016 Safety Factor Assessment.

Maximum Surcharge 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 surcharge pool" elevation within the Slurry Pond was established based on the maximum surface water elevation within the Slurry Pond computed from the hydrologic and hydraulic (H&H) analysis with the IDF and selected as a more conservative water level (35.1 ft NGVD29) than the maximum surface water level (34.1 ft NGVD 29) from the

H&H analyses. Details of the H&H analyses are provided in a document titled "*Inflow Design Flood Control System Plan: Slurry Pond*" and the H&H analysis results are included as Attachment 2 to the 2021 Safety Factor Assessment Report.

Final Cross Section Geometry

The final geometric models implemented within SLIDE® for Cross Sections C, D, and E are provided in Figures 2 through 4 for the maximum normal storage pool conditions, respectively (maximum surcharge storage pool conditions not shown).

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. 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 is presented in the Site Response Package of the 2021 Safety Factor Assessment Report. Within that package, 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). Preliminary seismic slope stability analyses of the perimeter dike structures of the Slurry Pond indicated that the critical depth of the anticipated slip surfaces is approximately 20 ft below the dike crest. The MHEA at the anticipated critical slip surface was selected assuming the critical slip surface is located at 20 ft below the dike crest for Cross Sections C through E. 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. The MHEA profiles for Profiles 1 and 2 to an approximate depth of 100 ft below ground surface (bgs) are provided in Table 2. MHEA values of 0.096g and 0.088g and was 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 5. Thus, k_h values of 0.044, 0.048, and 0.048 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 4. Computed safety factors were found to exceed the minimum safety factors required by §257.73(e)(1) of the CCR Rule. Figures 6 through 14 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.

REFERENCES

Bishop, A. (1955), "The Use of the Slip Circle in the Stability Analysis of Slopes," *Géotechnique*, Vol. 5, No. 1, Jan 1955, pp. 7-17.

Bray, J.D., Augello, A.J., Leonards, G.A., Repetto, P.C., & Byrne, R.J. (1995), "Seismic Stability Procedures for Solid-Waste Landfills," *Journal of Geotechnical Engineering*, Vol. 121, No. 2, pp. 139-151.

Geosyntec Consultants, Inc. (2016), "2016 Surface Impoundment Periodic Safety Factor Assessment Report: Slurry Pond, Winyah Generating Station, Georgetown, South Carolina", submitted to Santee Cooper.

Geosyntec Consultants (2021), "Periodic Hazard Potential Classification Assessment: Slurry Pond."

Hynes-Griffin, M. and Franklin, A. (1984), "Rationalizing the Seismic Coefficient Method", Department of the Army, Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi, Miscellaneous Paper GL-84-14, Jul.

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

Kavazanjian, E., (1999), "Seismic Design of Solid Waste Containment Facilities", Proceedings of the 8th Canadian Conference on Earthquake Engineering, Vancouver, BC, June, pp. 51-89.

Rocscience (2016), "SLIDE® – 2-D Limit Equilibrium Slope Stability for Soil and Rock Slopes," User's Guide, Rocscience Software, Inc., Toronto, Ontario, Canada.

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

Thomas and Hutton (2012), "Topographic Survey of a Portion of Santee Cooper Winyah Generating Station", prepared for Santee Cooper, 14 Jan.

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 [4]	95	40	0	0.5	0

Notes:

1. Undrained strength parameters for clayey foundation soils were applied for seismic slope stability case only.
2. 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.
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. Maximum Equivalent Horizontal Acceleration (MHEA) from Site Response Analysis for the Slurry Pond Perimeter Dikes and Varying Water Table Elevation

Profile 1		Profile 2	
Depth (ft)	MHEA	Depth (ft)	MHEA
2.5	0.121	2.5	0.112
7.5	0.100	7.5	0.100
12.5	0.097	12.5	0.088
17.5	0.094	17.5	0.085
20	0.096	20	0.088
22.5	0.092	22.5	0.084
27.5	0.087	27.5	0.079
32	0.082	32	0.073
36	0.077	36	0.067
40	0.071	40	0.061
44	0.064	44	0.055
48	0.061	48	0.052
52.5	0.067	52.5	0.059
60	0.074	60	0.066

Note:

1. Critical slip surfaces were found to have depths to 20 ft. MHEA values of 0.096g and 0.088g were selected for Profile 1 (Cross Sections D and E) and Profile 2 (Cross Section C).

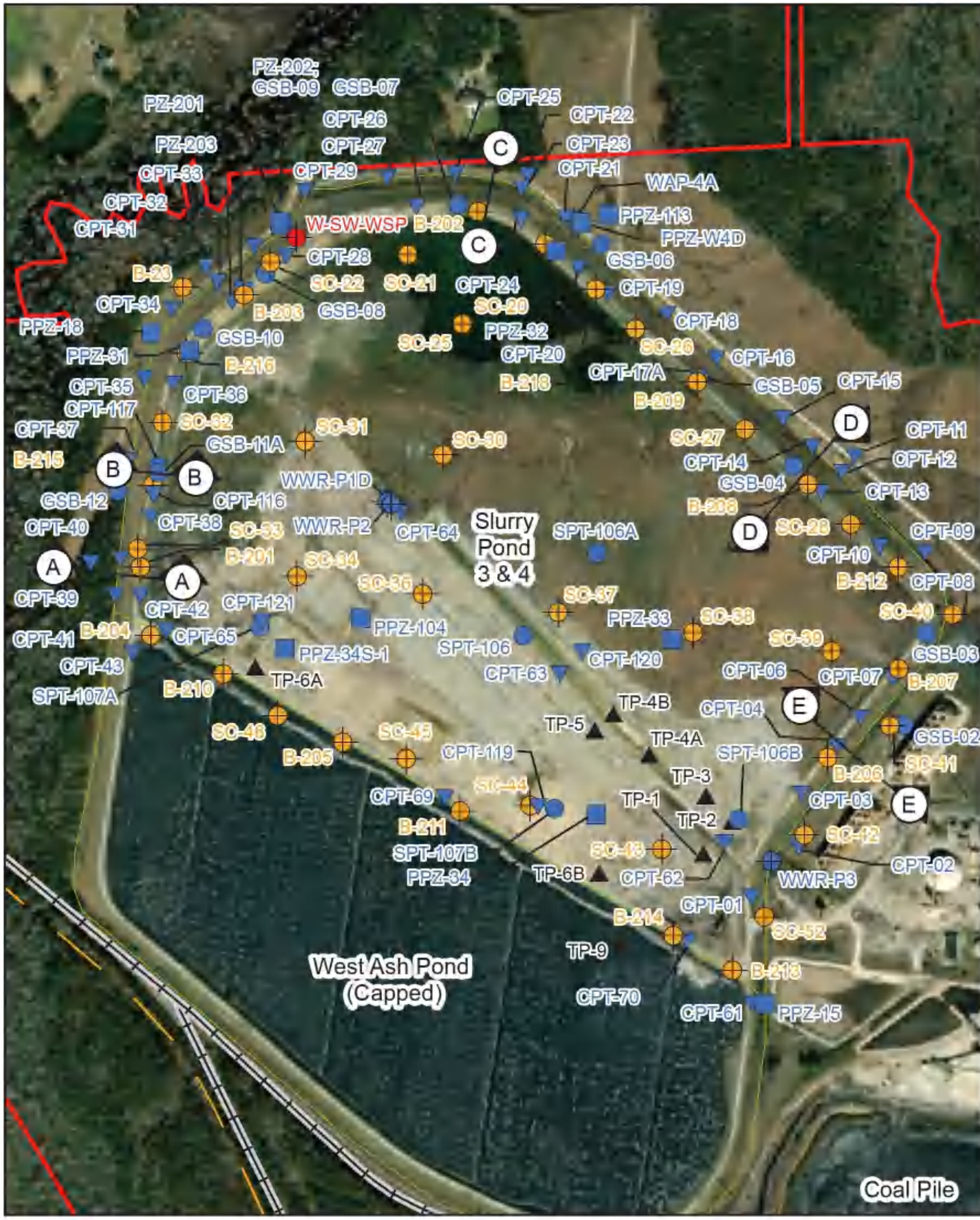
Table 3. Summary of Calculated Factors of Safety (FS)

Factor of Safety Case	Target FS	Cross Section C	Cross Section D	Cross Section E
Static - Maximum Normal Storage Pool	1.50	1.72	*1.62*	1.68
Static - Maximum Surcharge Pool	1.40	1.69	*1.41*	*1.41*
Seismic - Maximum Normal Storage Pool	1.00	1.09	1.14	*1.06*
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 (Attachment 4).
2. Only critical failure surfaces passing through the perimeter dikes were considered.
3. The lowest computed safety factor for each analysis case was italicized.

FIGURES



N

LEGEND

W-SW-APB	EXISTING STAFF GAUGE
CPT-144	GEOSYNTEC CONE PENETRATION TEST
PPZ-5	PIEZOMETER
SPT-111	GEOSYNTEC SOIL BORING
B-1	HISTORICAL BORING
WAP-9	GROUNDWATER MONITORING WELL
TP-1	TEST PIT
	POND BOUNDARY
	PROPERTY BOUNDARY
	EXISTING GAS LINE
	EXISTING RAILROAD
A A	CROSS SECTION LOCATION

NOTES:
1. Aerial imagery was obtaied from ESRI online database.
2. The position of underground utilities shown should be considered approximate.

600 300 0 600 Feet

SLURRY POND
WINYAH GENERATING STATION

GEORGETOWN, SOUTH CAROLINA

Geosyntec consultants

Figure 1

Charlotte, NC | October 2021

Y:\S\Santee Cooper\Santee Cooper - Winyah\2021 - 5 Yr CCR Rule Requirements\7 - Safety Factor Assessment\GIS\MXD\September 2021\FigureX_Slurry Pond.mxd; Abby.Wesley; 10/12/2021

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Dike Fill		125	Mohr-Coulomb	100	33			Water Surface	Custom
FGD Slurry		95	Strength=F(overburden)			0.5	0	Water Surface	Custom
Chicora Member		130	Mohr-Coulomb	0	50			Water Surface	Custom
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom
Sandy Foundation Material		115	Mohr-Coulomb	0	32			Water Surface	Custom
Riprap		150	Mohr-Coulomb	0	45			Water Surface	Custom
Clayey Foundation Material		100	Mohr-Coulomb	0	28			Water Surface	Custom

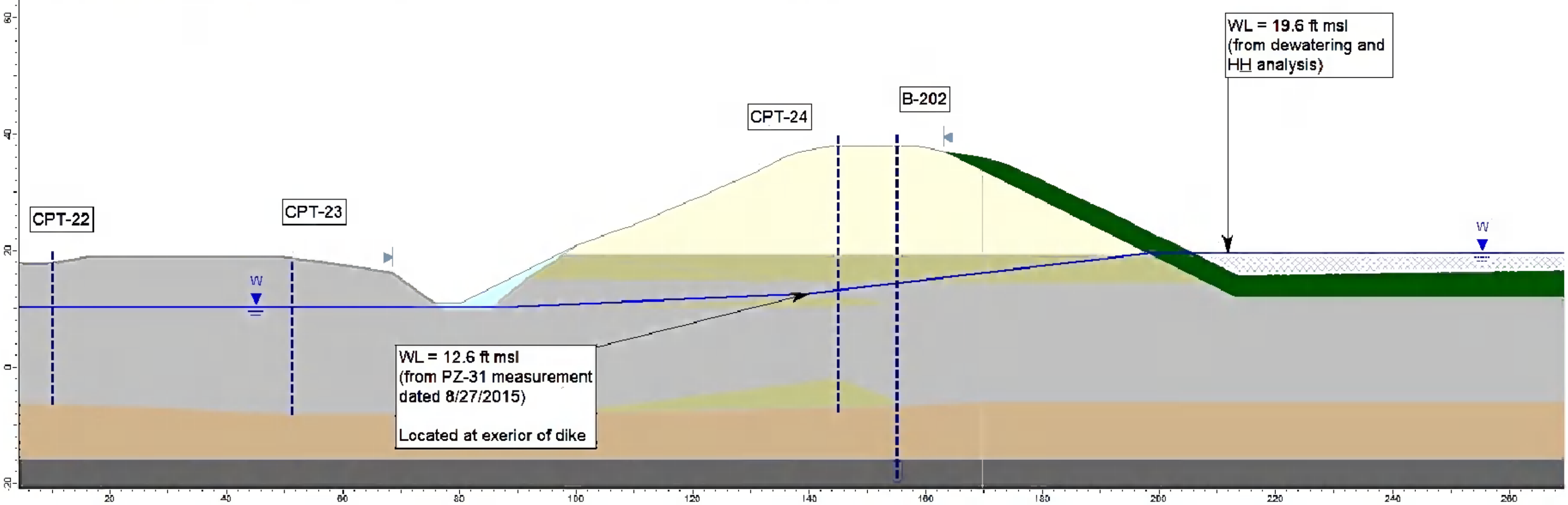


Figure 2. Cross Section C Geometry for Maximum Normal Storage Pool

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.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Dike Fill		125	Mohr-Coulomb	100	33			Water Surface	Custom
FGD Slurry		95	Strength=F(overburden)			0.5	0	Water Surface	Custom
Chicora Member		130	Mohr-Coulomb	0	50			Water Surface	Custom
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom
Sandy Foundation Material		115	Mohr-Coulomb	0	32			Water Surface	Custom
Clayey Foundation Material		100	Mohr-Coulomb	0	28			Water Surface	Custom

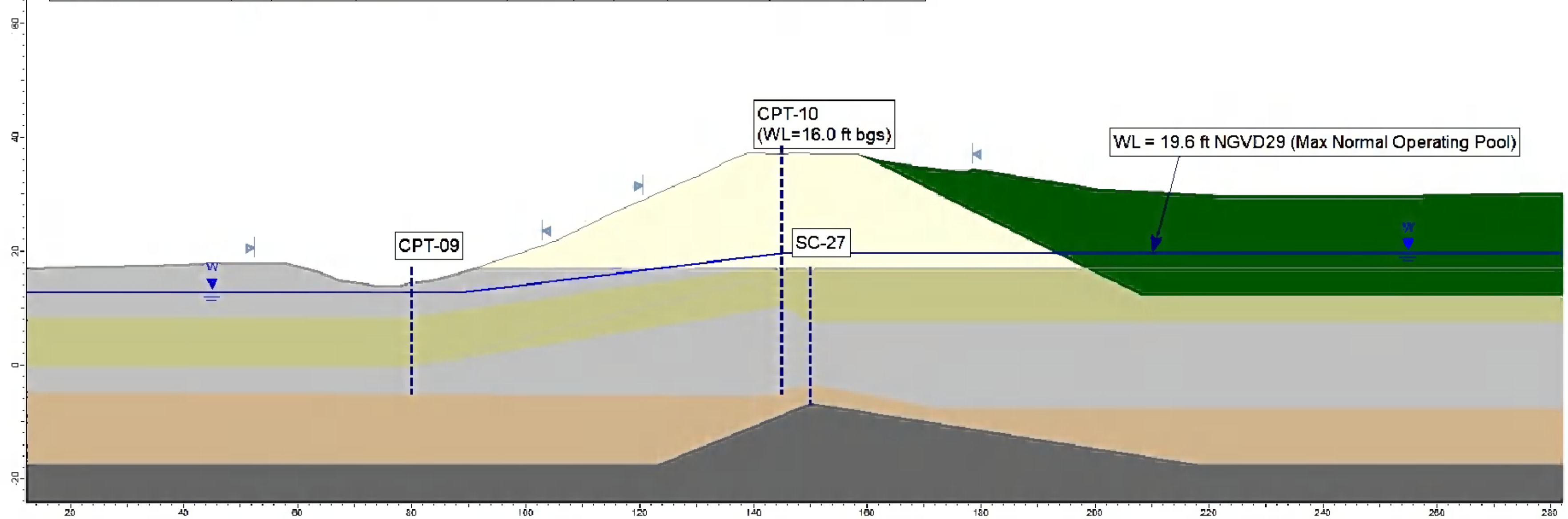


Figure 3. Cross Section D Geometry for Maximum Normal Storage Pool

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Dike Fill		125	Mohr-Coulomb	100	33			Water Surface	Custom
FGD Slurry		95	Strength=F(overburden)			0.5	0	Water Surface	Custom
Chicora Member		130	Mohr-Coulomb	0	50			Water Surface	Custom
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom
Sandy Foundation Material		115	Mohr-Coulomb	0	32			Water Surface	Custom
Clayey Foundation Material		100	Mohr-Coulomb	50	28			Water Surface	Custom

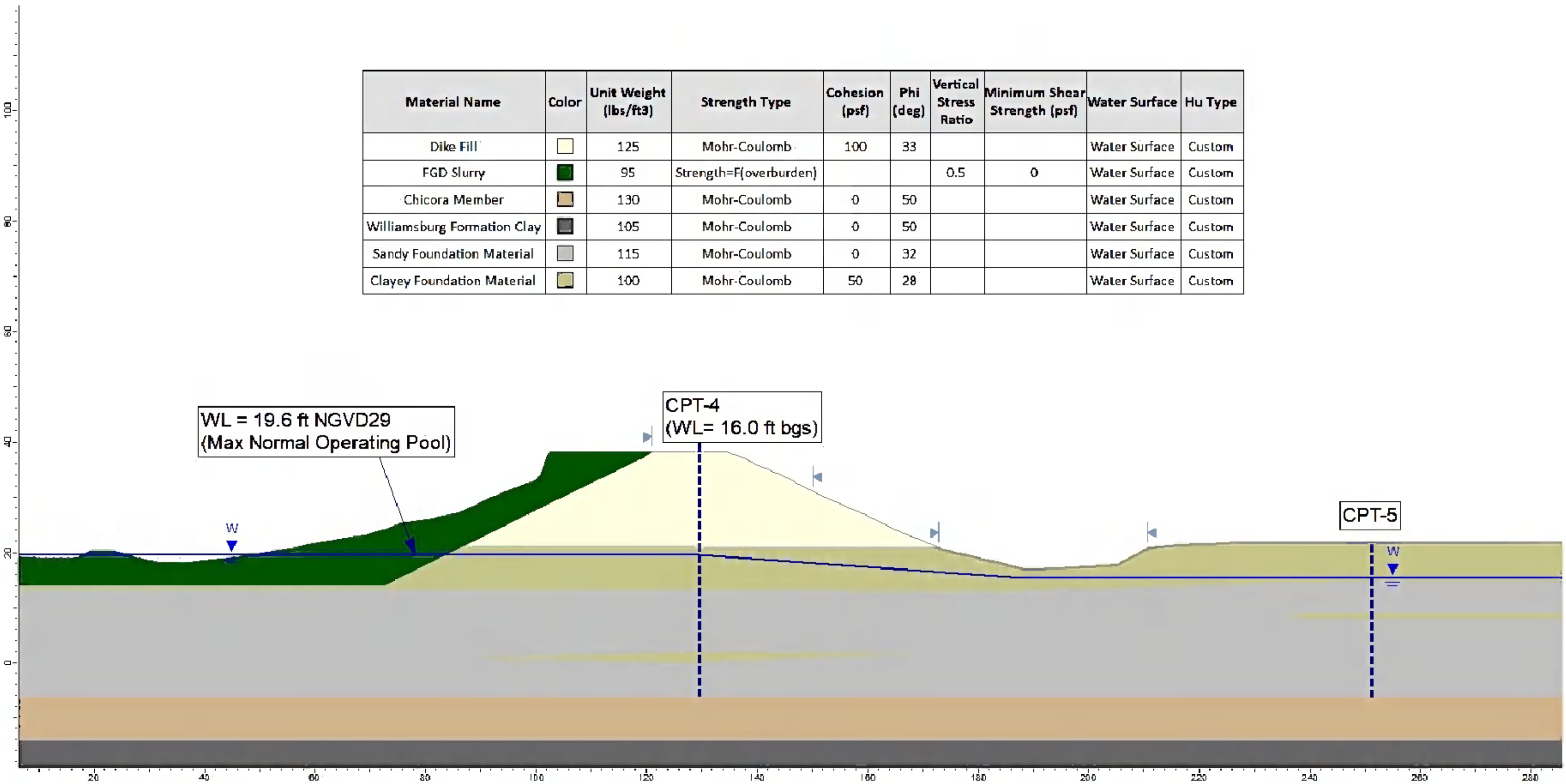


Figure 4. Cross Section E Geometry for Maximum Normal Storage Pool

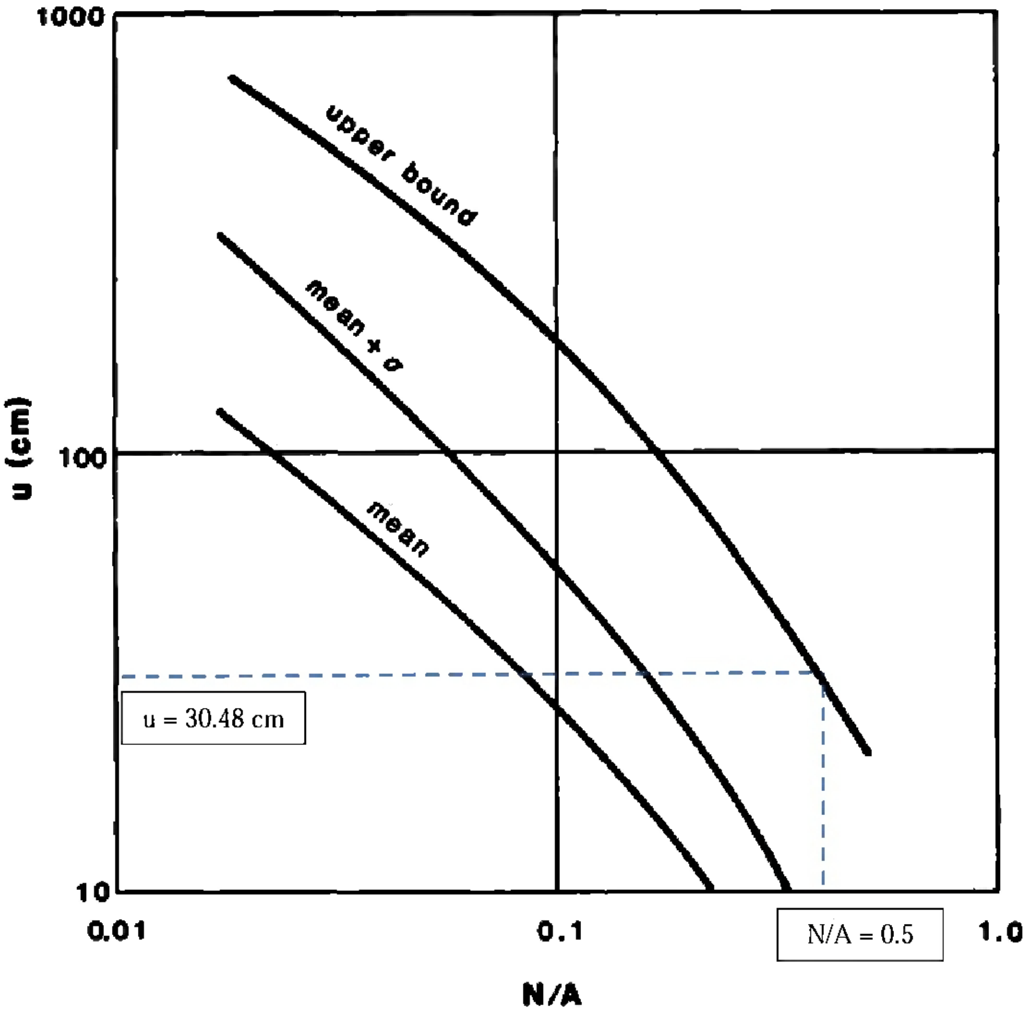
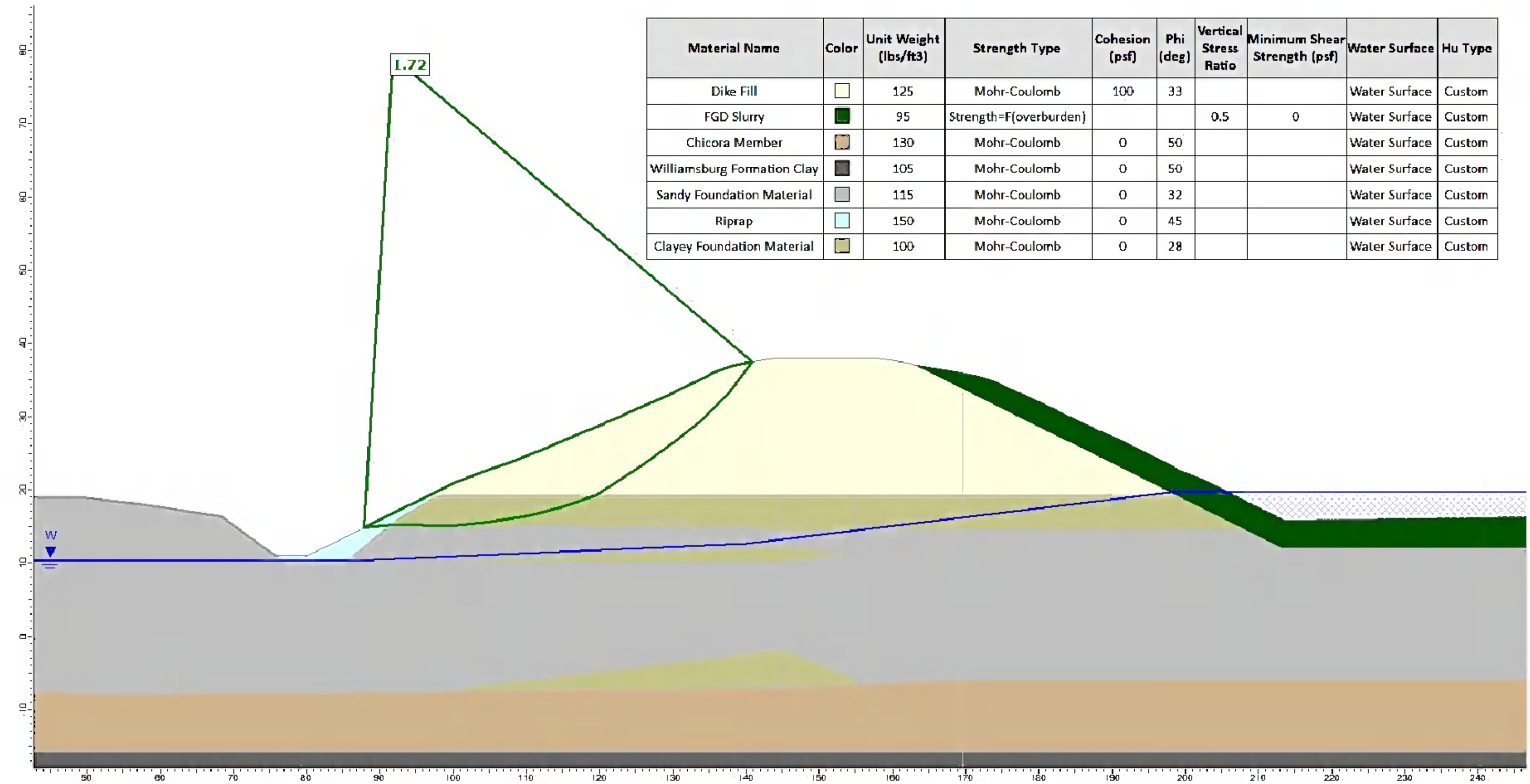


Figure 5. Allowable Deformation (u) vs. N/A
(from Figure 7 of Hynes-Griffin and Franklin, 1984)

Notes:

1. An allowable deformation (u) of 12 in. (30.48 cm) and the "Upper Bound" curve were selected for the 2021 Safety Factor Assessment.
2. A ratio of N/A of 0.50 was selected assuming 12 in. of allowable deformation.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Dike Fill		125	Mohr-Coulomb	100	33			Water Surface	Custom
FGD Slurry		95	Strength=F(overburden)			0.5	0	Water Surface	Custom
Chicora Member		130	Mohr-Coulomb	0	50			Water Surface	Custom
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom
Sandy Foundation Material		115	Mohr-Coulomb	0	32			Water Surface	Custom
Riprap		150	Mohr-Coulomb	0	45			Water Surface	Custom
Clayey Foundation Material		100	Mohr-Coulomb	0	28			Water Surface	Custom

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

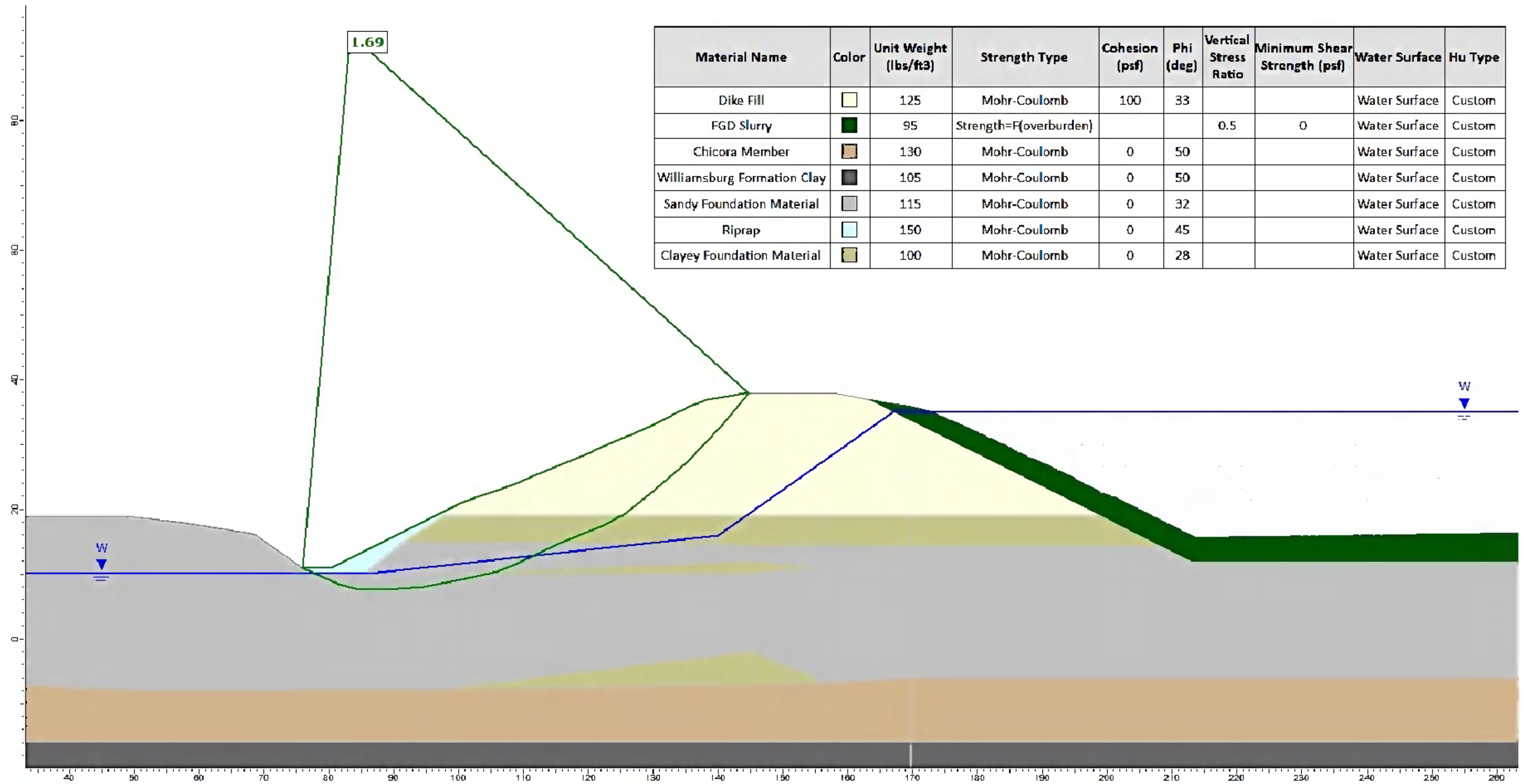


Figure 7. Calculated Factor of Safety for Cross Section C: Static Factor of Safety - Maximum Surcharge Pool

Note:

[1] "Maximum Surcharge Pool" was conservatively selected at 35.1 ft NGVD29 within the Slurry Pond.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Dike Fill		125	Strength=F(overburden)			0.52	100	Water Surface	Custom
FGD Slurry		95	Strength=F(overburden)			0.4	0	Water Surface	Custom
Chicora Member		130	Mohr-Coulomb	0	50			Water Surface	Custom
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom
Sandy Foundation Material		115	Mohr-Coulomb	0	32			Water Surface	Custom
Riprap		150	Mohr-Coulomb	0	45			Water Surface	Custom
Clayey Foundation Material		100	Strength=F(overburden)			0.28	0	Water Surface	Custom

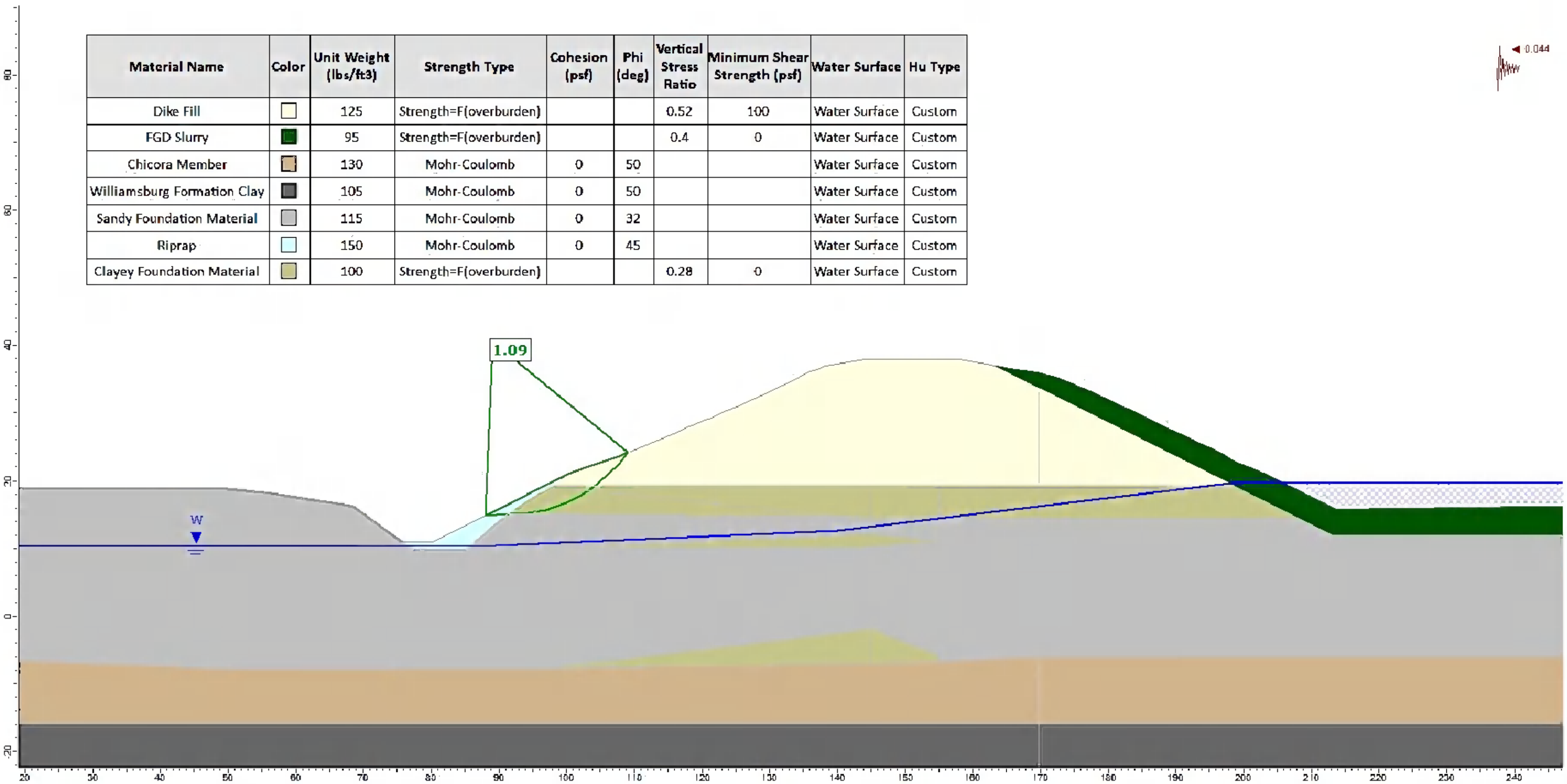
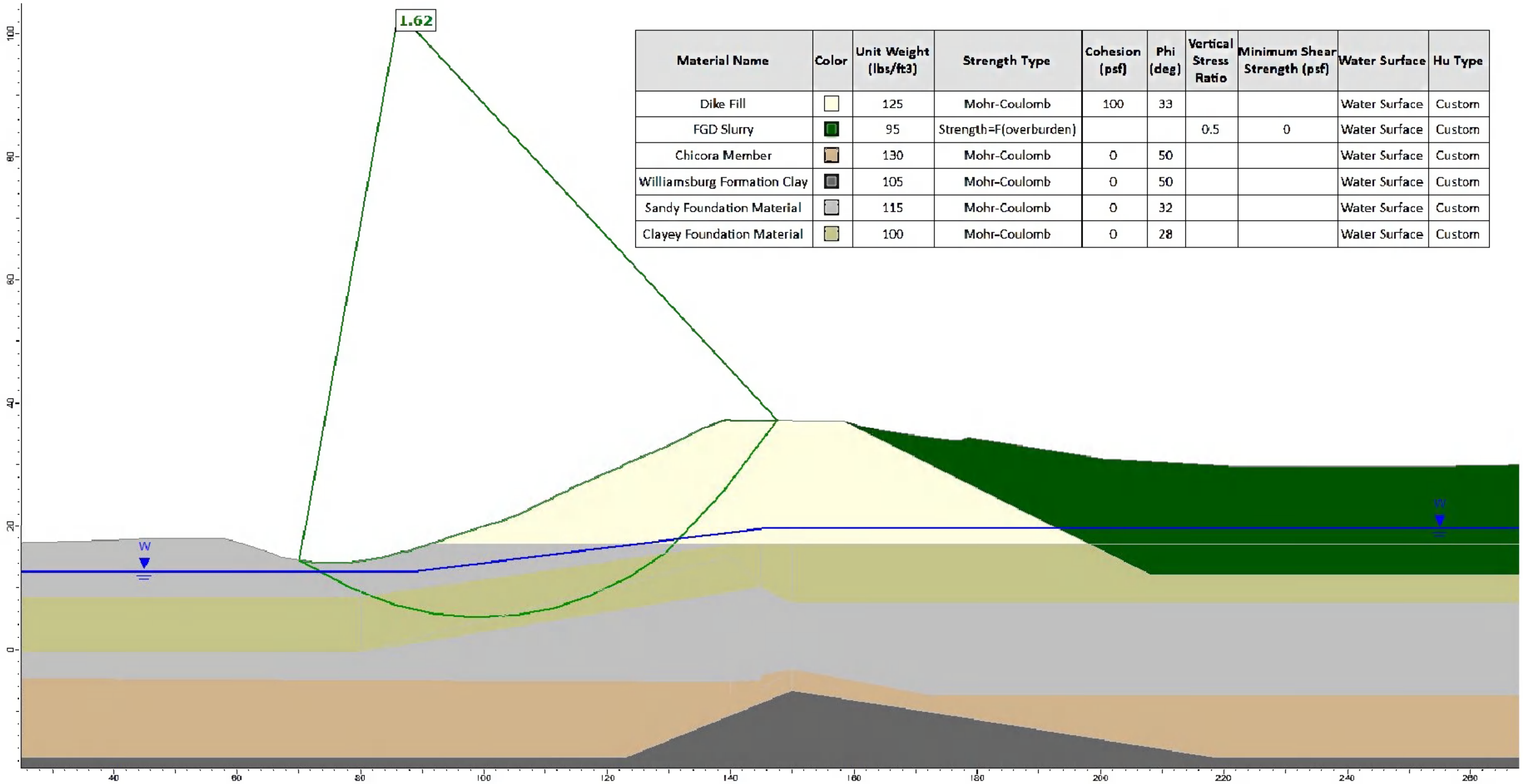
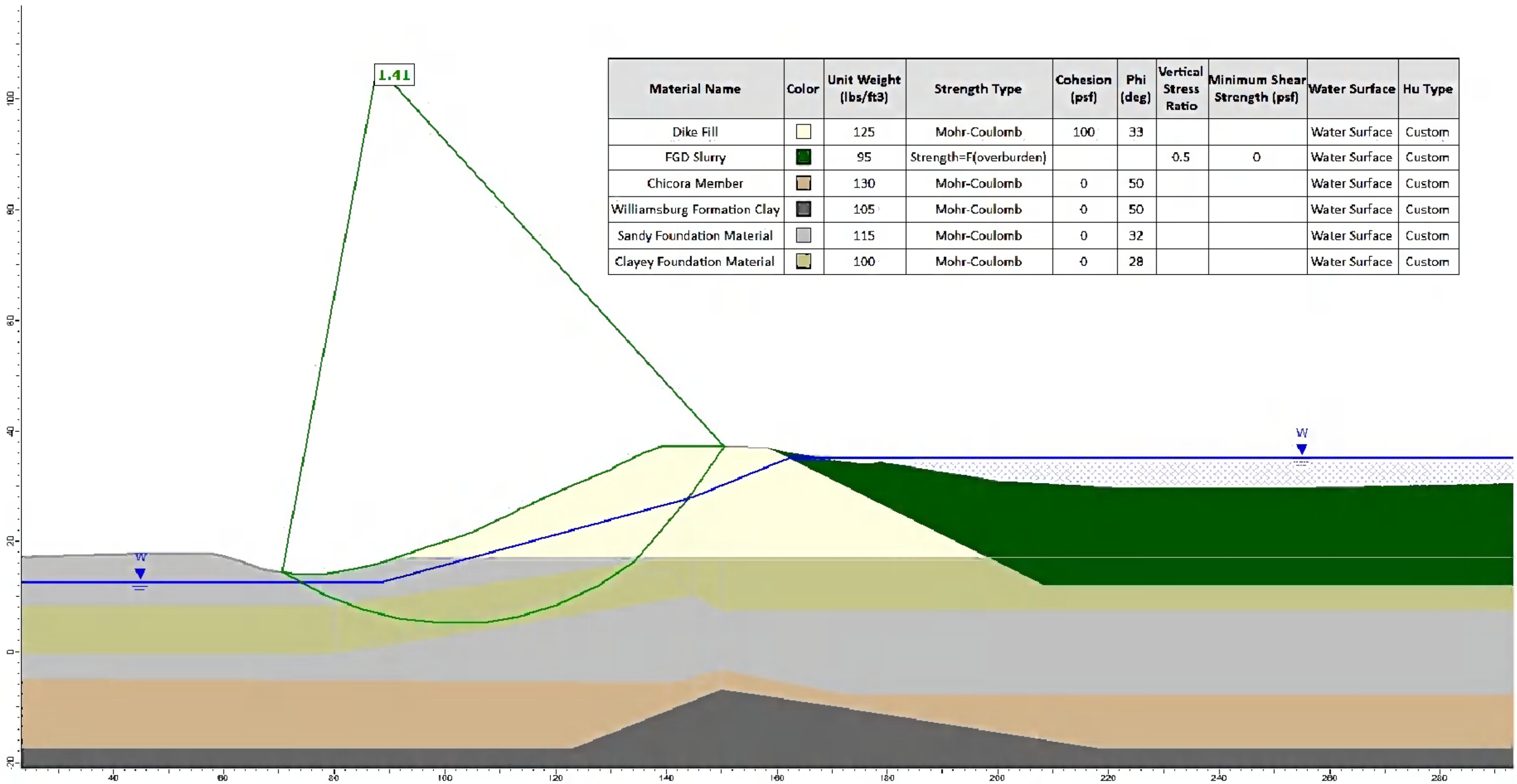


Figure 8. Calculated Factor of Safety for Cross Section C: Seismic Factor of Safety – Maximum Normal Storage Pool



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Dike Fill		125	Mohr-Coulomb	100	33			Water Surface	Custom
FGD Slurry		95	Strength=F(overburden)			0.5	0	Water Surface	Custom
Chicora Member		130	Mohr-Coulomb	0	50			Water Surface	Custom
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom
Sandy Foundation Material		115	Mohr-Coulomb	0	32			Water Surface	Custom
Clayey Foundation Material		100	Mohr-Coulomb	0	28			Water Surface	Custom

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

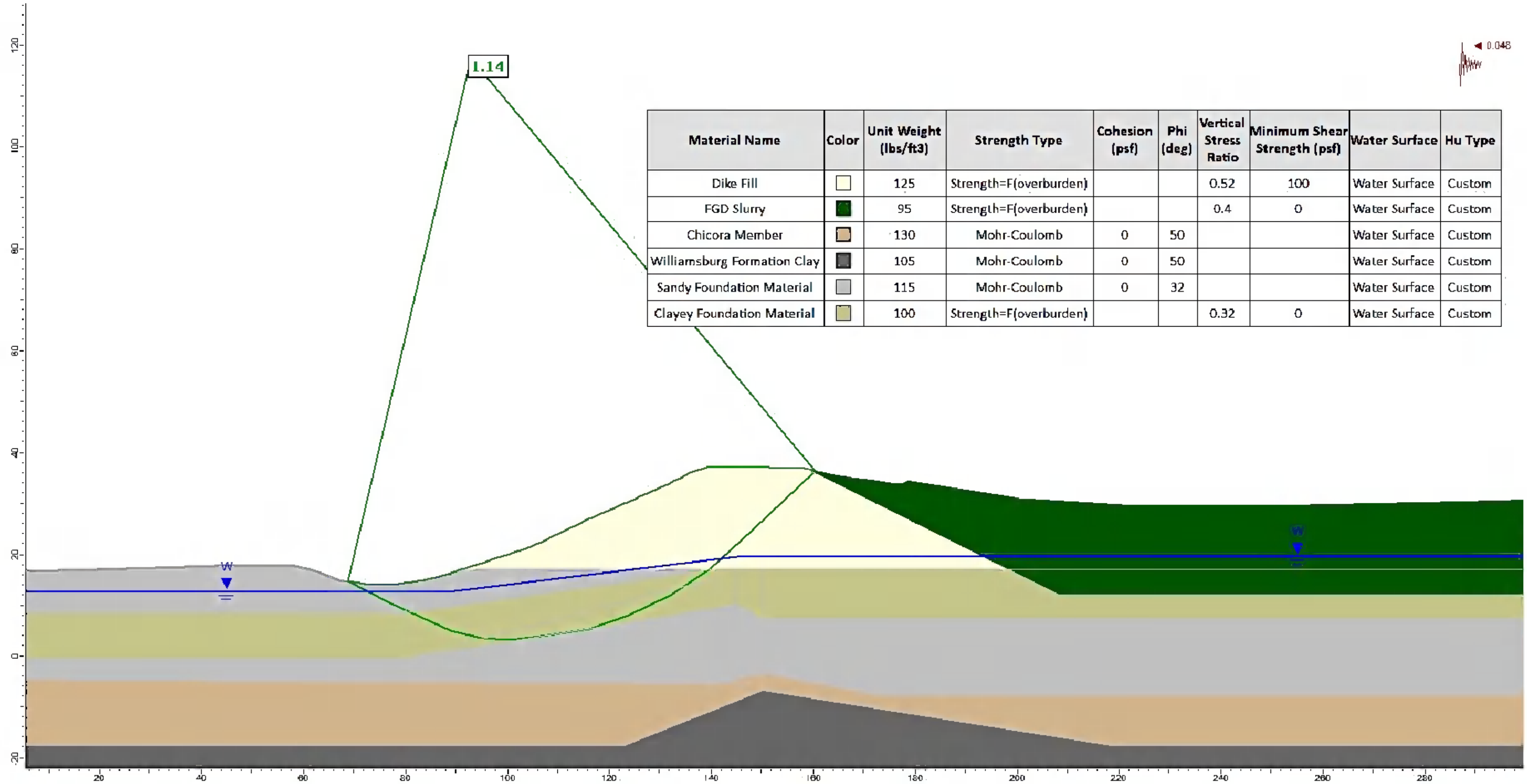


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Dike Fill		125	Mohr-Coulomb	100	33			Water Surface	Custom
FGD Slurry		95	Strength=F(overburden)			0.5	0	Water Surface	Custom
Chicora Member		130	Mohr-Coulomb	0	50			Water Surface	Custom
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom
Sandy Foundation Material		115	Mohr-Coulomb	0	32			Water Surface	Custom
Clayey Foundation Material		100	Mohr-Coulomb	0	28			Water Surface	Custom

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

Note:

1. "Maximum Surcharge Pool" was conservatively selected at 35.1 ft NGVD29 within the Slurry Pond.

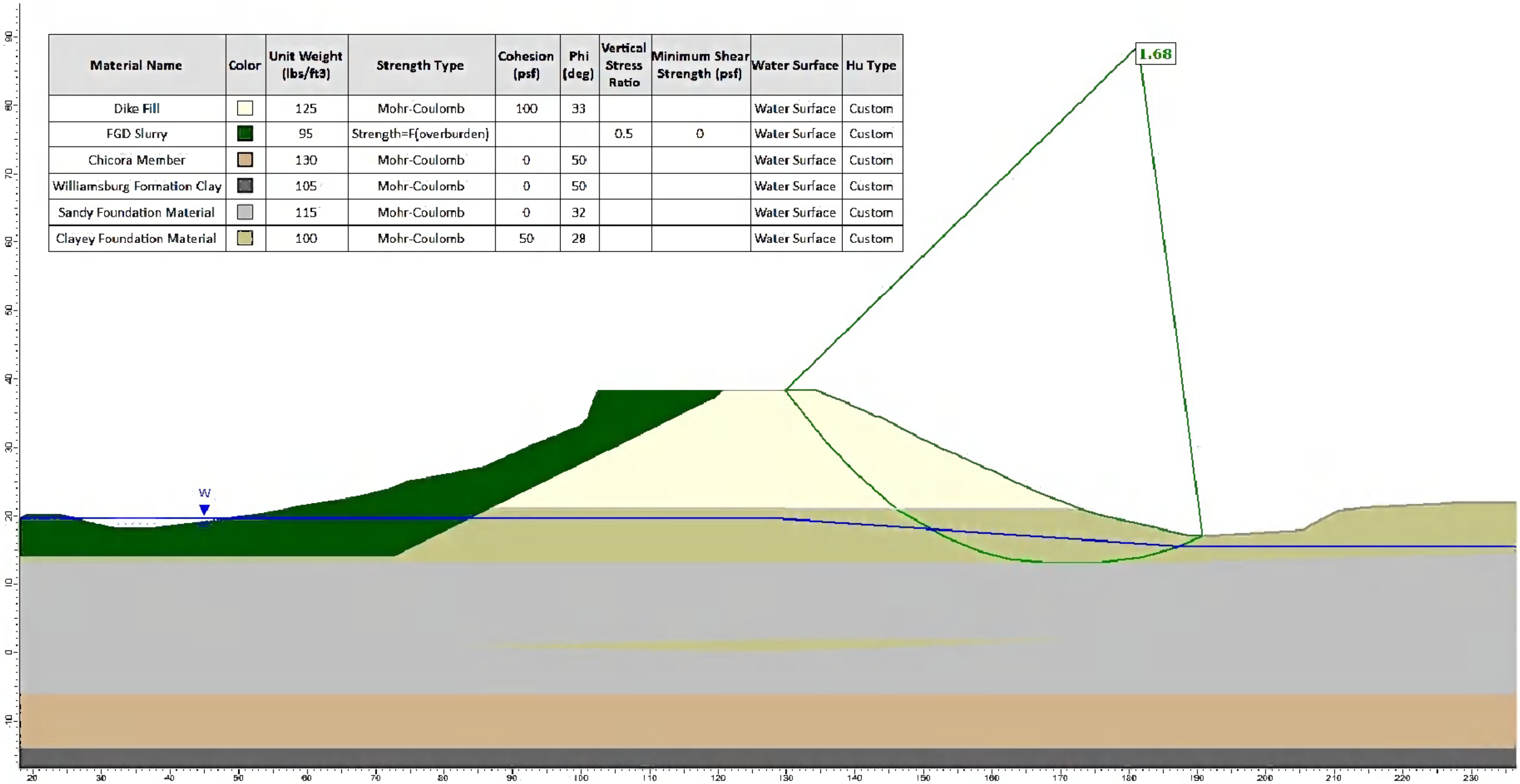


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Dike Fill		125	Strength=F(overburden)			0.52	100	Water Surface	Custom
FGD Slurry		95	Strength=F(overburden)			0.4	0	Water Surface	Custom
Chicora Member		130	Mohr-Coulomb	0	50			Water Surface	Custom
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom
Sandy Foundation Material		115	Mohr-Coulomb	0	32			Water Surface	Custom
Clayey Foundation Material		100	Strength=F(overburden)			0.32	0	Water Surface	Custom

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

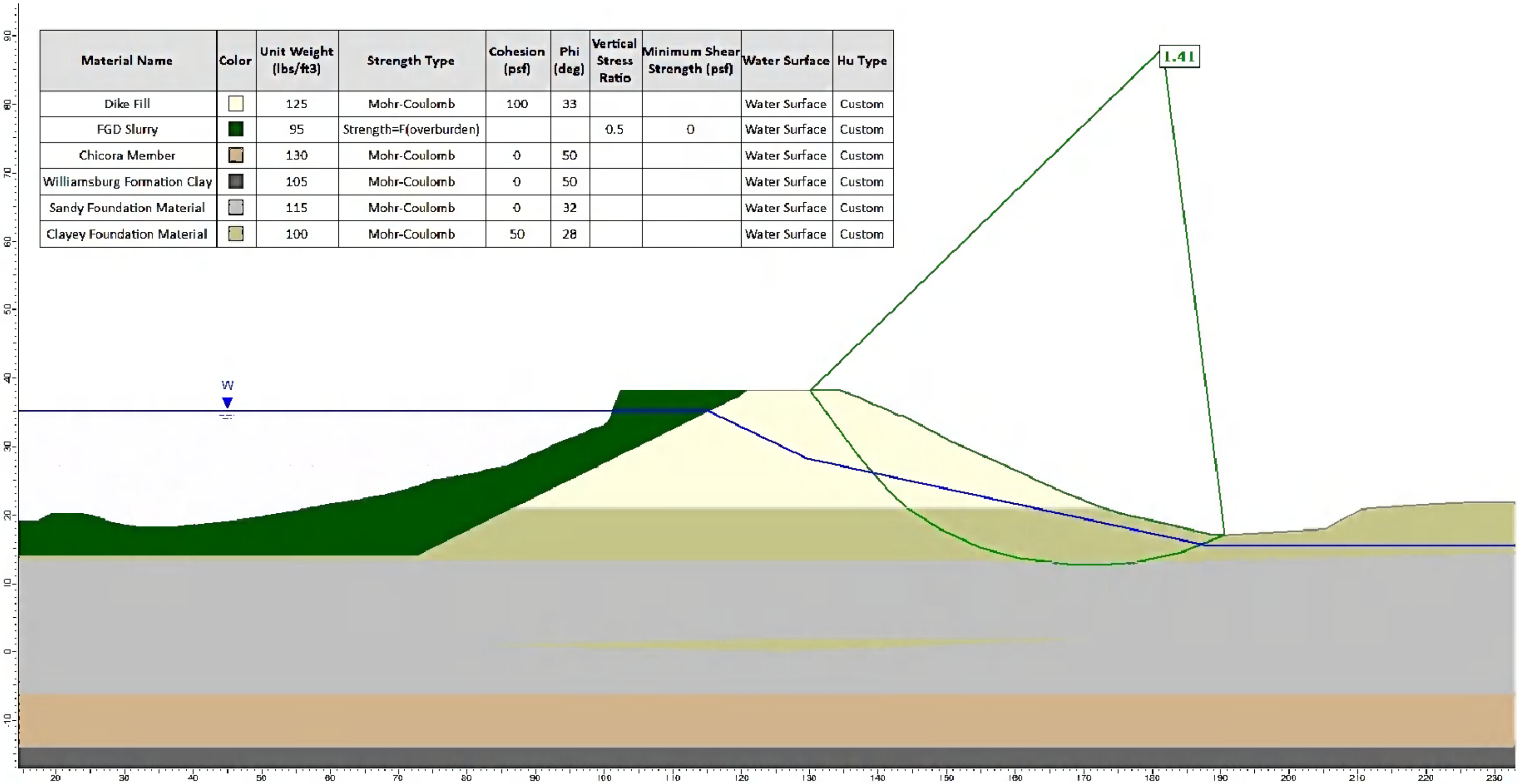
Note:

1. The undrained shear strength ratio was reduced by 20% to account for the influence of cyclic degradation.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Dike Fill		125	Mohr-Coulomb	100	33			Water Surface	Custom
FGD Slurry		95	Strength=F(overburden)			0.5	0	Water Surface	Custom
Chicora Member		130	Mohr-Coulomb	0	50			Water Surface	Custom
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom
Sandy Foundation Material		115	Mohr-Coulomb	0	32			Water Surface	Custom
Clayey Foundation Material		100	Mohr-Coulomb	50	28			Water Surface	Custom

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

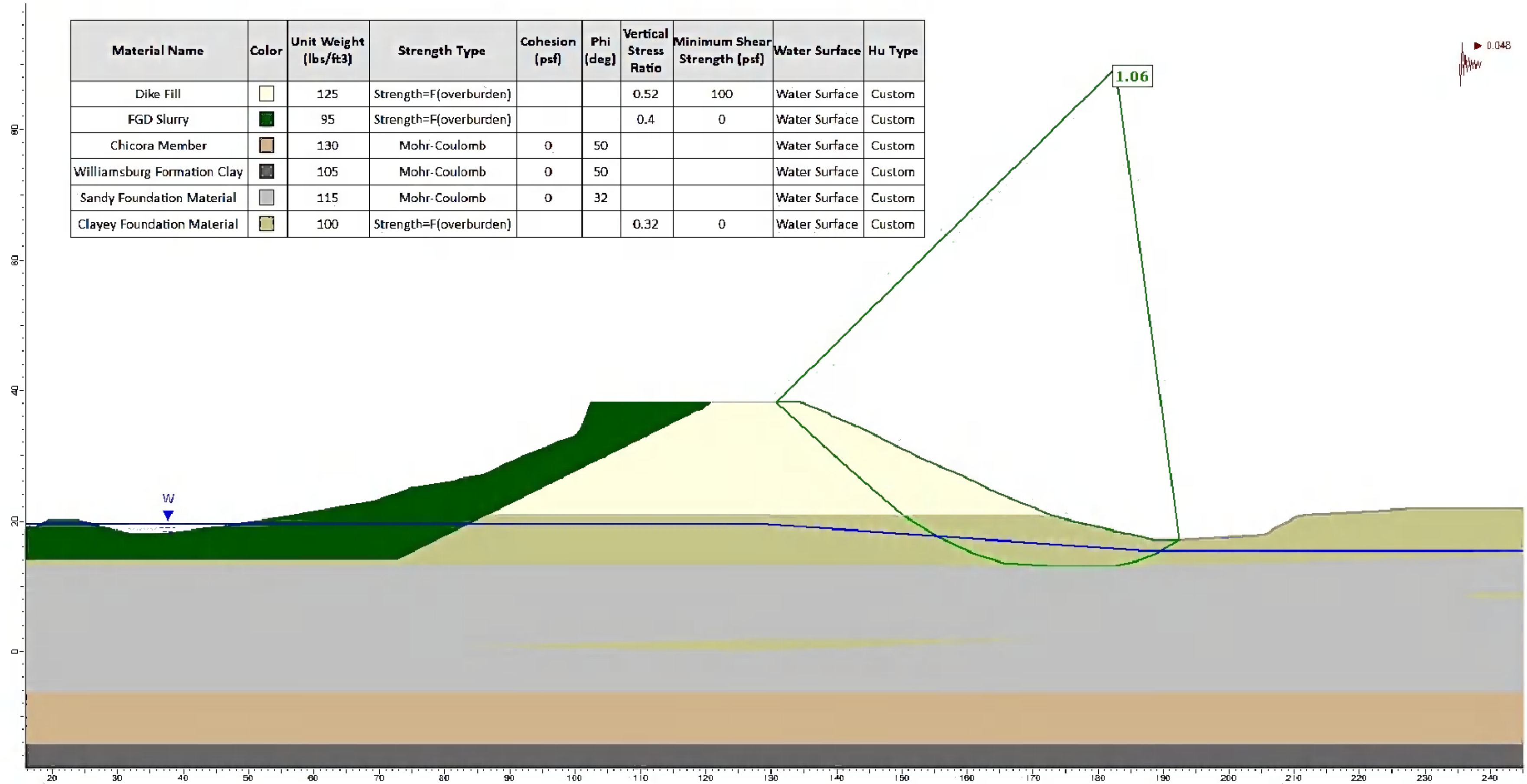


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Dike Fill		125	Mohr-Coulomb	100	33			Water Surface	Custom
FGD Slurry		95	Strength=F(overburden)			0.5	0	Water Surface	Custom
Chicora Member		130	Mohr-Coulomb	0	50			Water Surface	Custom
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom
Sandy Foundation Material		115	Mohr-Coulomb	0	32			Water Surface	Custom
Clayey Foundation Material		100	Mohr-Coulomb	50	28			Water Surface	Custom

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

Notes:

1. The phreatic surface within the dike fill was assumed to be approximately 10 ft below the dike crest, which is consistent with the 24-hr water level measurement from a hollow stem auger boring nearby (HSA-1).
2. "Maximum Surcharge Pool" was conservatively selected at 35.1 ft NGVD29 within the Slurry Pond.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type
Dike Fill		125	Strength=F(overburden)			0.52	100	Water Surface	Custom
FGD Slurry		95	Strength=F(overburden)			0.4	0	Water Surface	Custom
Chicora Member		130	Mohr-Coulomb	0	50			Water Surface	Custom
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom
Sandy Foundation Material		115	Mohr-Coulomb	0	32			Water Surface	Custom
Clayey Foundation Material		100	Strength=F(overburden)			0.32	0	Water Surface	Custom

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

Note:

1. The undrained shear strength ratio was reduced by 20% to account for the influence of cyclic degradation.