Prepared for

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2021 PERIODIC SAFETY FACTOR ASSESSMENT, Revision 1
SOUTH ASH POND

WINYAH GENERATING STATION
Georgetown, South Carolina

Prepared by

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Project No. GC8100

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CERTIFICATION STATEMENT

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

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1. INTRODUCTION

1.1 Project Background

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

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 South Ash Pond is situated immediately south of the Coal Pile and power block and west of the Discharge Canal (Figure 2). The South Ash Pond contains CCR in the form of fly ash, boiler slag, and bottom ash 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 South Ash Pond was posted to the Operating Record on 9 April 2021 to initiate pond closure, and CCR and wastewater inflow to the South Ash 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 Report: South Ash Pond* (Safety Factor Assessment Report) was prepared by Geosyntec Consultants, Inc. (Geosyntec) on behalf of Santee Cooper to demonstrate that the South Ash 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 South Ash Pond spans approximately 76 acres. This unlined surface impoundment was commissioned in 1980 and was designated for the disposal of fly ash, bottom ash, and boiler slag. The South Ash Pond is bounded by the Coal Pile and power block to the north, Pennyroyal Creek to the west, a forested area to the south, and an access road and the Discharge Canal to the east. The South Ash Pond was assigned "Low Hazard Potential" classification (Geosyntec, 2021a).

The South Ash Pond was constructed by recompacting excavated soils from the surface impoundment interior to form perimeter dikes. The South Ash Pond perimeter dikes have a maximum height of approximately 24 feet (ft). The interior and downstream side slopes of the dikes are approximately 3 horizontal to 1 vertical (3H:1V), except in the western corner where the downstream side slopes are approximately 4H:1V. The dike crest is typically 12 to 15 ft wide (Thomas and Hutton, 2012). The minimum elevation of the dike crest is 36.9 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 assessment for the South Ash Pond at WGS based on the results of the initial periodic safety factor assessment (2016 Safety Factor Assessment) (Geosyntec, 2016), recent survey dated August 2021 (McKim & Creed, 2021), subsequent hydrologic and hydraulic (H&H) analysis 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 Assessment is presented in Section 2.
- H&H evaluation of the South Ash Pond is presented in Section 3;
- Seismic hazard evaluations for WGS and the site response analysis of the South Ash 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 South Ash 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. However, a review of the topographic survey dated August 2021 (McKim & Creed, 2021) (Attachment 1) and the topographic survey used in the 2016 Safety Factor Assessment indicated that dewatering lowered the free water level in the east side of the South Ash Pond and CCR have been excavated from the east side of the surface impoundment (top of CCR surface in the west side of the South Ash Pond is similar to that used for the 2016 Safety Factor Assessment). The volume of CCR impounded within the surface impoundment has not been changed significantly since the last assessment.

Santee Cooper provided available water level measurements from wells in the South Ash 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 South Ash Pond perimeter dikes, the water level within the perimeter dike may be similar to the water level used for the 2016 Safety Factor Assessment or lower due to dewatering in the east side of the pond. As discussed above, CCR and wastewater inflow to the South Ash 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 South Ash 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 *"100-year flood for a low hazard potential CCR surface impoundment."* Additionally, §257.73(d)(1)(v)(A) indicates that "*All spillways must be either:*

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

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

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

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

Because the South Ash Pond has been classified as a "Low Hazard Potential" surface impoundment, the 100-year rainfall 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 South Ash Pond spillway 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). 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 South Ash Pond computed from the H&H analyses.

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 South Ash Pond during the IDF. The 100-year rainfall event with a 72-hour duration precipitation event resulted in a rainfall depth of 12.8 in. (NOAA, 2021) and was modeled within HydroCAD® using a SCS Type III rainfall distribution.

The normal operating level in the South Ash Pond is maintained by a rectangular concrete riser structure with 4 ft-long stoplogs on a single face. The water elevation within the South Ash Pond is at elevation 15 ft NGVD 29 and no stoplogs are in place on the riser at the time of this assessment to facilitate ongoing closure activities. A 36-inch diameter reinforced concrete pipe with an upstream invert elevation of 16.93 ft NGVD 29 conveys water from the riser structure to the Discharge Canal (Lockwood Greene, 1978).

Details of the H&H analyses are provided in a document titled *"Inflow Design Flood Control System Plan: South Ash 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 free water level or "maximum surcharge pool" level during and following the IDF event was computed as 28.1 ft NGVD29, assuming the free water is maintained at a normal operating elevation of 15 ft NGVD 29. The H&H analysis results (i.e., HydroCAD® results) are included as Attachment 2 of this Safety Factor Assessment Report.

4. SEISMIC HAZARD EVALUATION AND SITE RESPONSE ANALYSIS

This section presents the results of seismic hazard evaluation and site response analysis of the South Ash 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 of 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 South Ash 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 assumed 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. One critical profile was selected for the site response analyses of the South Ash Pond perimeter dikes and is 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 calculated maximum cyclic shear strains greater than five percent in some layers, which is 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 discussed in Attachment 3.

The site response analyses for the South Ash Pond as part of the 2016 Safety Factor Assessment considered a water table 10 ft below ground surface (bgs). As described 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 or lower due to dewatering in the east side of the pond. Therefore, site response analyses were performed with the water table modeled at 10 as well as 18 ft bgs to account for a potentially lowered water table.

4.2.2 Site Response Analysis Results

Maximum shear stresses within the representative soil profiles were computed and presented on Figures 9 and 11 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 South Ash 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 South Ash 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 where the factor of safety against liquefaction (FS_{liq}) were relatively lower compared to other locations in the 2016 Safety Factor Assessment. The 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 expected to be similar to the water level used in the 2016 Safety Factor Assessment or lower due to dewatering in the east side of the South Ash Pond. 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 and with the phreatic surface assumed to be near or at the bottom of the dike.

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 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 for 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. The location of the interface between dike fill soil and foundation soils was estimated as 1 ft below the toe drains elevations shown on the Lockwood-Greene (1978) design drawings.

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 South Ash Pond perimeter dikes. Analysis results for each soil boring and CPT sounding analyzed are provided on Figures 3 and 10 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 section of the South Ash Pond perimeter dikes).

6. SAFETY FACTOR ASSESSMENT

This section presents the periodic safety factor assessments for the South Ash 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 South Ash Pond perimeter dikes satisfiy the safety factor (also referred to as "factor of safety") criteria of §257.73(e)(1) of the CCR Rule. Specifically, §257.73(e)(1) requires that:

"(i) The calculated static factor of safety under the long-term, maximum storage pool loading condition must equal or exceed 1.50.

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

(iii) The calculated seismic factor of safety must equal or exceed 1.00.

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

Because the dike fills and foundation soils beneath the dike fill along the critical section of the South Ash 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). Two representative cross sections were selected for the assessment based on factors of safety calculated in the 2016 Safety Factor Assessment. Consistent with observations regarding the water level described in Section 2, the water level within the perimeter dike is the maximum normal storage water level used in the 2016 Safety Factor Assessment and assumed to be near or at the bottom of the dike.

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 surface 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 an embankment or dike structure. For the purpose of this Safety Factor Assessment Report, the allowable displacement of the South Ash Pond perimeter dike structures was selected as 12 inches (in.). 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®. The selected k_h values are shown on Figures 11, 12, 15, and 16 of Attachment 5.

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 factors were evaluated for the two representative cross sections with two different water levels, as shown on Figures 2 through 5 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 for the two representative cross sections assuming a more conservative water level (30.7 ft NGVD29) within the South Ash Pond than the maximum surface water level (28.1 ft NGVD 29) from the H&H analyses (Section 3).

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 two representative cross sections with the two water levels discussed in Section 6.2 by applying computed seismic horizontal force coefficients to each slice within SLIDE®. During the evaluation of the seismic safety factor, soil shear strengths for cohesive soils were conservatively reduced by 20% to account for the influence of cyclic degradation (Hynes-Griffin and Franklin, 1984).

6.7 Summary of Results

The calculated minimum safety factor for each analysis case and each of the two representative cross sections are presented in Attachment 5. These analysis results indicate that the perimeter dikes of the South Ash 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 South Ash 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 South Ash Pond for the safety factor assessment was established based on the H&H performance of the South Ash 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 two percent probability of exceedance in 15 years, established conservatively with consideration of the remaining operating life

of the South Ash 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 South Ash 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 two representative cross sections of the South Ash Pond perimeter dikes, the South Ash Pond satisfies the required safety factors presented in §257.73(e)(1) as shown below.

Safety Factor Case	Target FS	Cross Section A*	Cross Section B*
Static - Maximum Normal Storage Pool (Base Water Table)	1.50	1.69	1.79
Static - Maximum Surcharge Pool (Base Water Table)	1.40	1.67	1.73
Seismic - Maximum Normal Storage Pool (Base Water Table)	1.00	1.24	1.09
Seismic – Maximum Normal Storage Pool (Lowered Water Table)	1.00	1.19	1.12
Liquefaction Slope Stability	1.20	Not Applicable	Not Applicable

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

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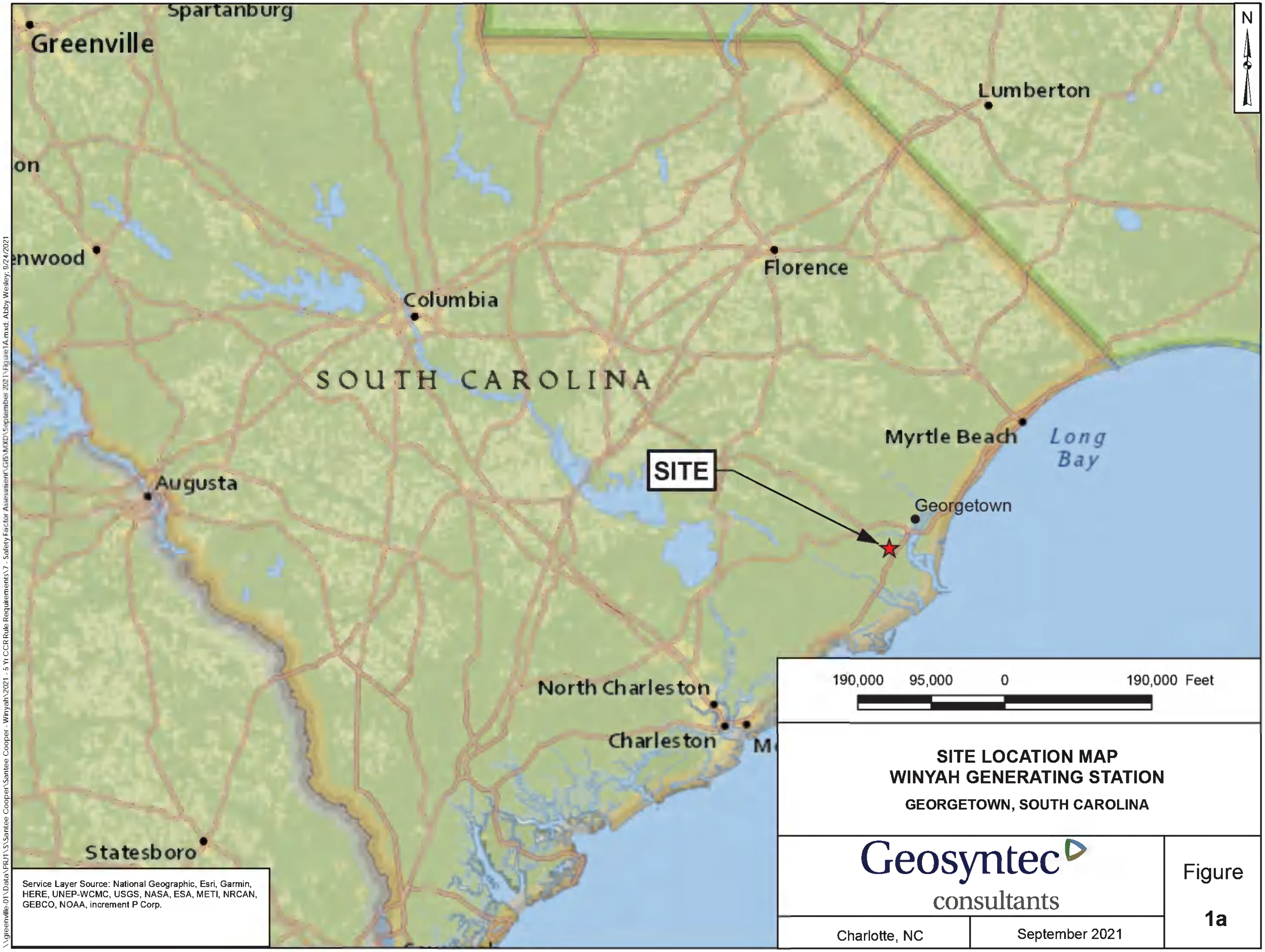
FIGURES



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

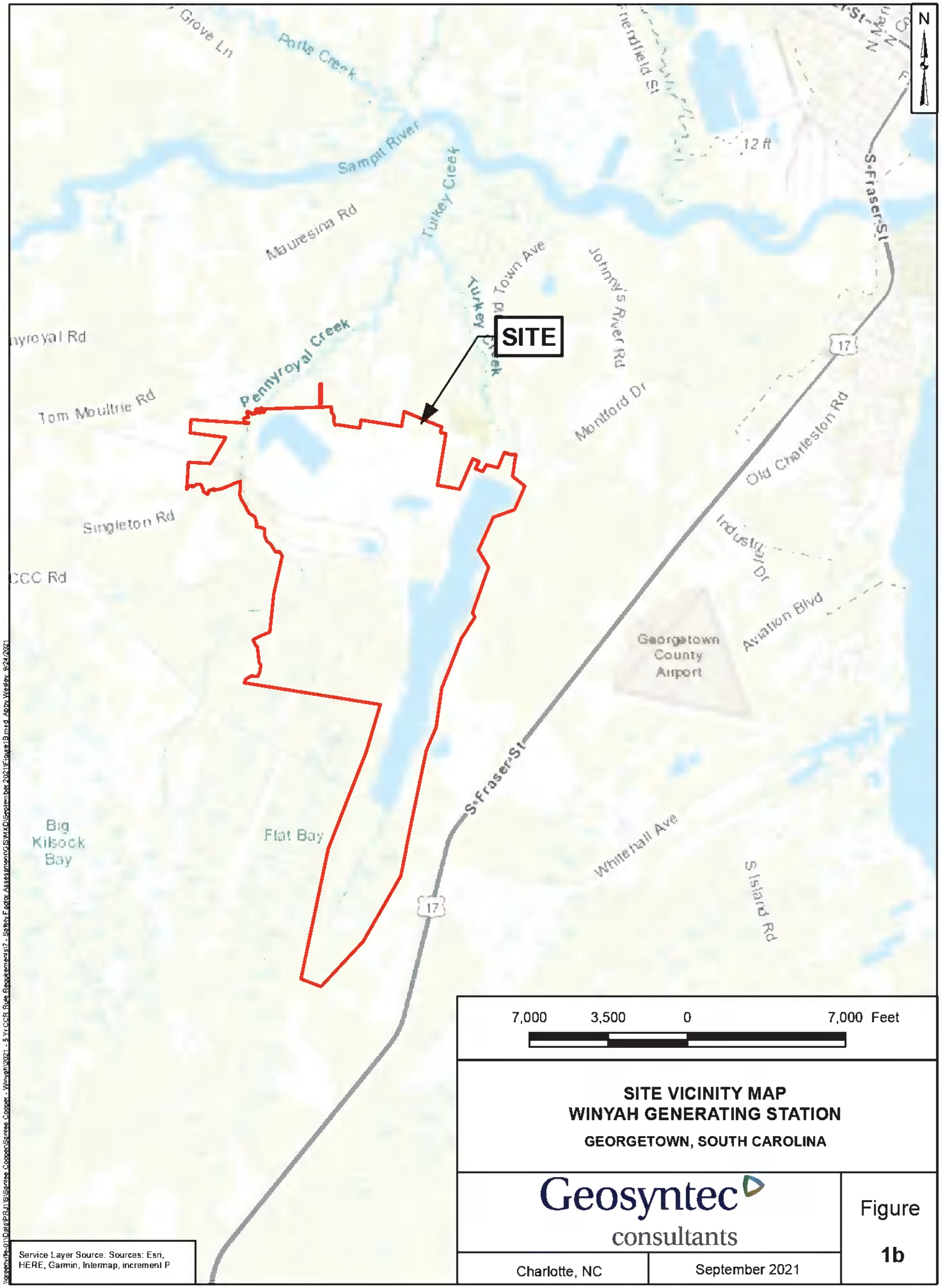
SITE LOCATION MAP
WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

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



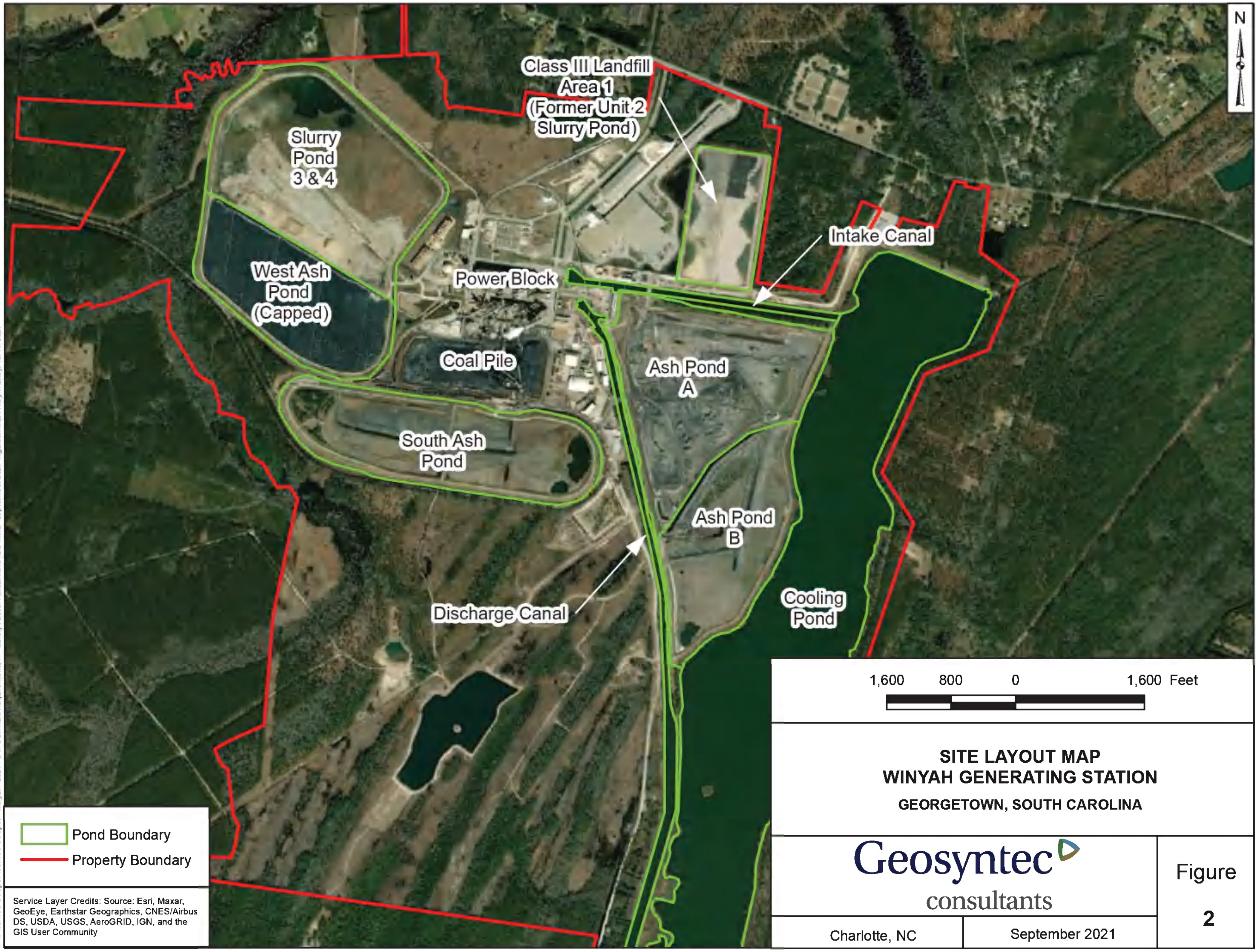
Service Layer Source: Sources: Esri, HERE, Garmin, Intermap, increment P

SITE VICINITY MAP
WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

Geosyntec consultants

Charlotte, NC | September 2021

Figure **1b**



SITE LAYOUT MAP
WINYAH GENERATING STATION

GEORGETOWN, SOUTH CAROLINA

Geosyntec consultants

Figure

2

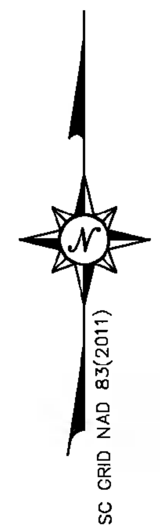
Charlotte, NC | September 2021

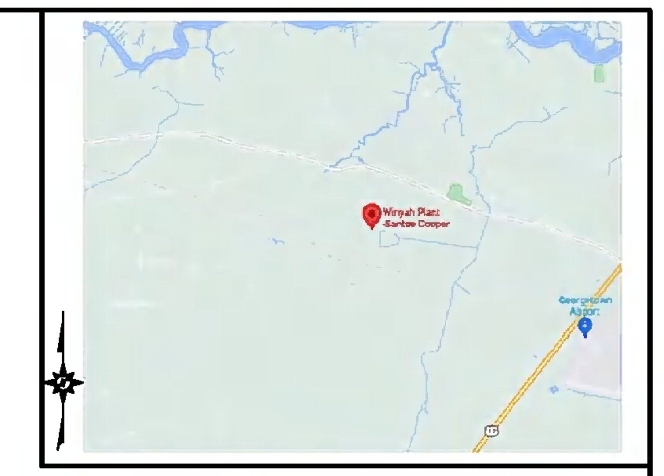
Y:\S\Santee Cooper\Santee Cooper - Winyah\2021 - 5 Yr CCR Rule Requirements\7 - Safety Factor Assessment\GIS\MXD\September 2021\Figure2.mxd; Abby Wesley; 10/11/2021

ATTACHMENTS

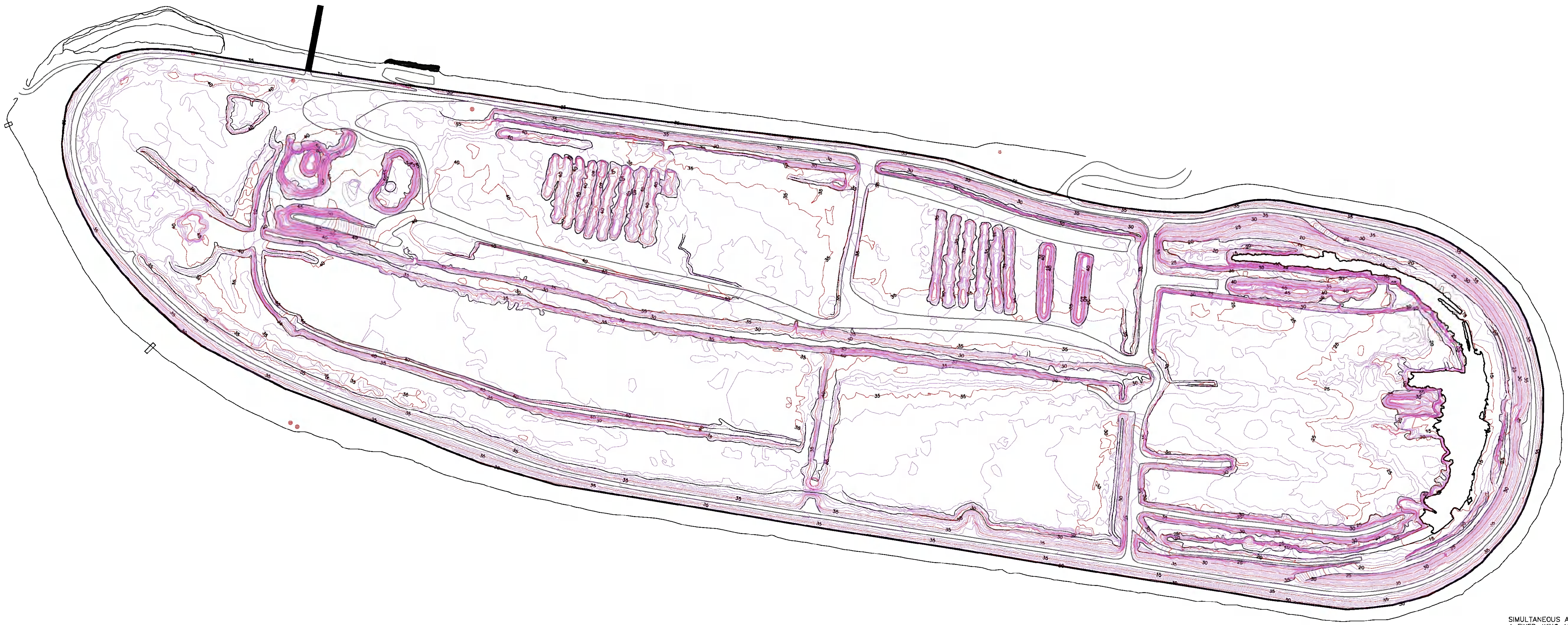
ATTACHMENT 1

TOPOGRAPHIC SURVEY (AUGUST 2021)





VICINITY MAP – NOT TO SCALE



SURVEYOR'S NOTES

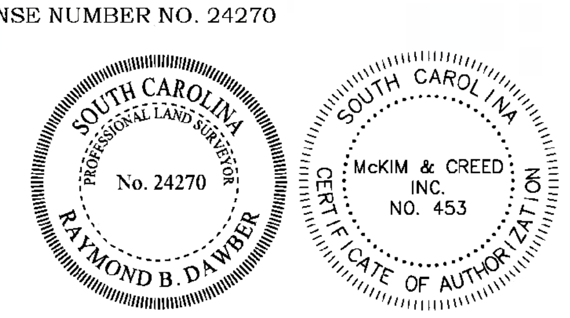
1. ALL DISTANCES ARE HORIZONTAL GROUND IN INTERNATIONAL FEET UNLESS OTHERWISE SHOWN.
2. BEARINGS BASED ON SOUTH CAROLINA NAD83/2011.
3. ELEVATIONS AND CONTOURS SHOWN HERON ARE BASED ON NAVD88. CONTOURS ARE SHOWN AT 1' INTERVALS.
4. SUBSURFACE AND ENVIRONMENTAL CONDITIONS WERE NOT EXAMINED OR CONSIDERED 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). TOPOGRAPHIO 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 8/13/2021

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



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

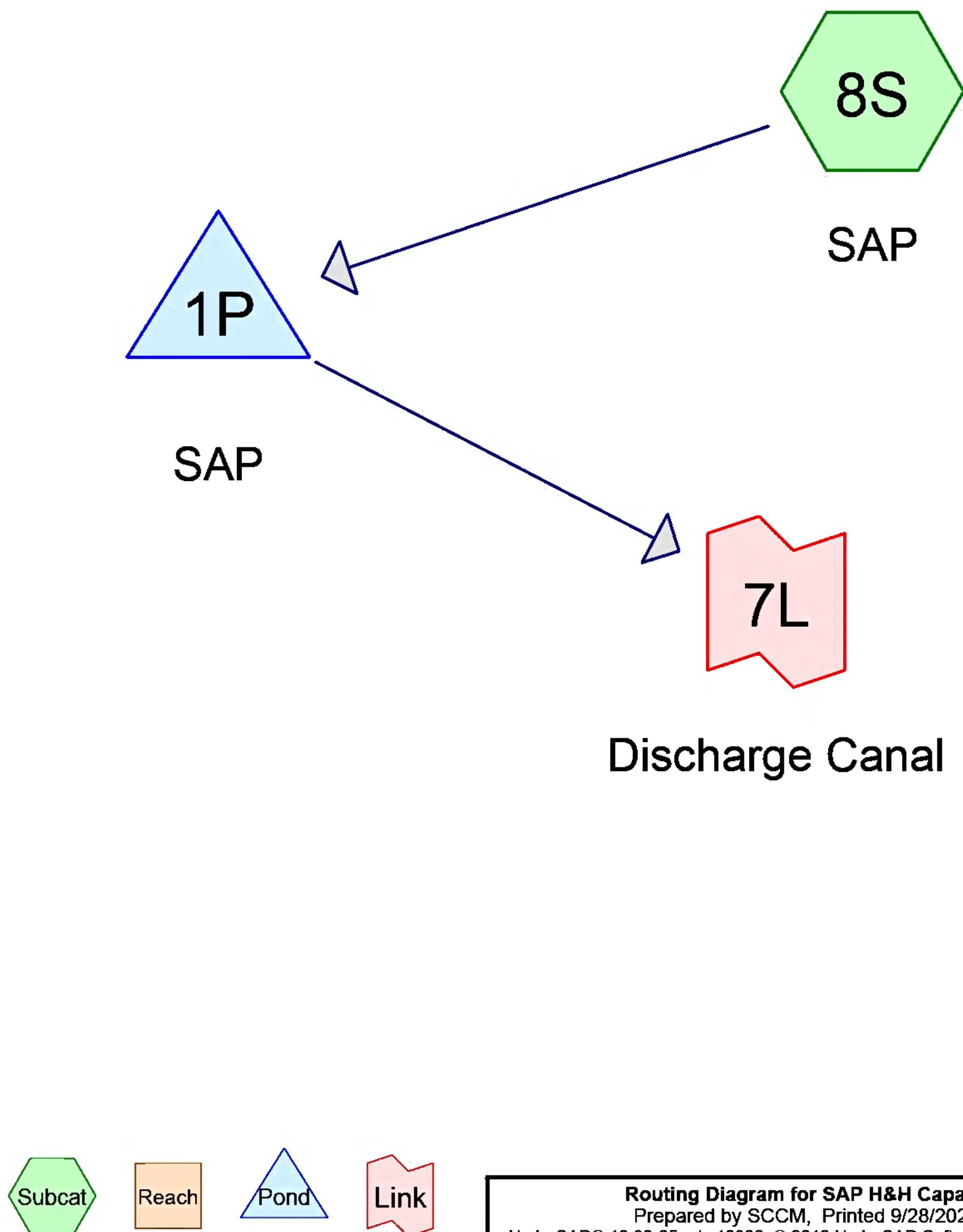
TOPOPGRAPHIC SURVEY
FOR
WINYAH GENERATING STATION
SOUTH ASH POND
SANTEE COOPER
LOCATION
07-24-2021

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 SAP H&H Capacity
Prepared by SCCM, Printed 9/28/2021

Area Listing (selected nodes)

Area (acres)	CN	Description (subcatchment-numbers)
75.600	86	(8S)
75.600	**86**	**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	
75.600	Other	8S
75.600		**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	75.600	75.600		8S
0.000	**0.000**	**0.000**	**0.000**	**75.600**	**75.600**	**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	1P	15.93	15.93	350.0	0.0000	0.013	36.0	0.0	0.0

Time span=0.00-500.00 hrs, dt=0.05 hrs, 10001 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Dyn-Stor-Ind method - Pond routing by Dyn-Stor-Ind method

Subcatchment 8S: SAP Runoff Area=75.600 ac 0.00% Impervious Runoff Depth=11.13"
Flow Length=4,000' Tc=48.9 min CN=86 Runoff=412.33 cfs 70.139 af

Pond 1P: SAP Peak Elev=27.06' Storage=70.139 af Inflow=412.33 cfs 70.139 af
Outflow=0.00 cfs 0.000 af

Link 7L: Discharge Canal Inflow=0.00 cfs 0.000 af
Primary=0.00 cfs 0.000 af

Total Runoff Area = 75.600 ac Runoff Volume = 70.139 af Average Runoff Depth = 11.13"
100.00% Pervious = 75.600 ac 0.00% Impervious = 0.000 ac

Summary for Subcatchment 8S: SAP

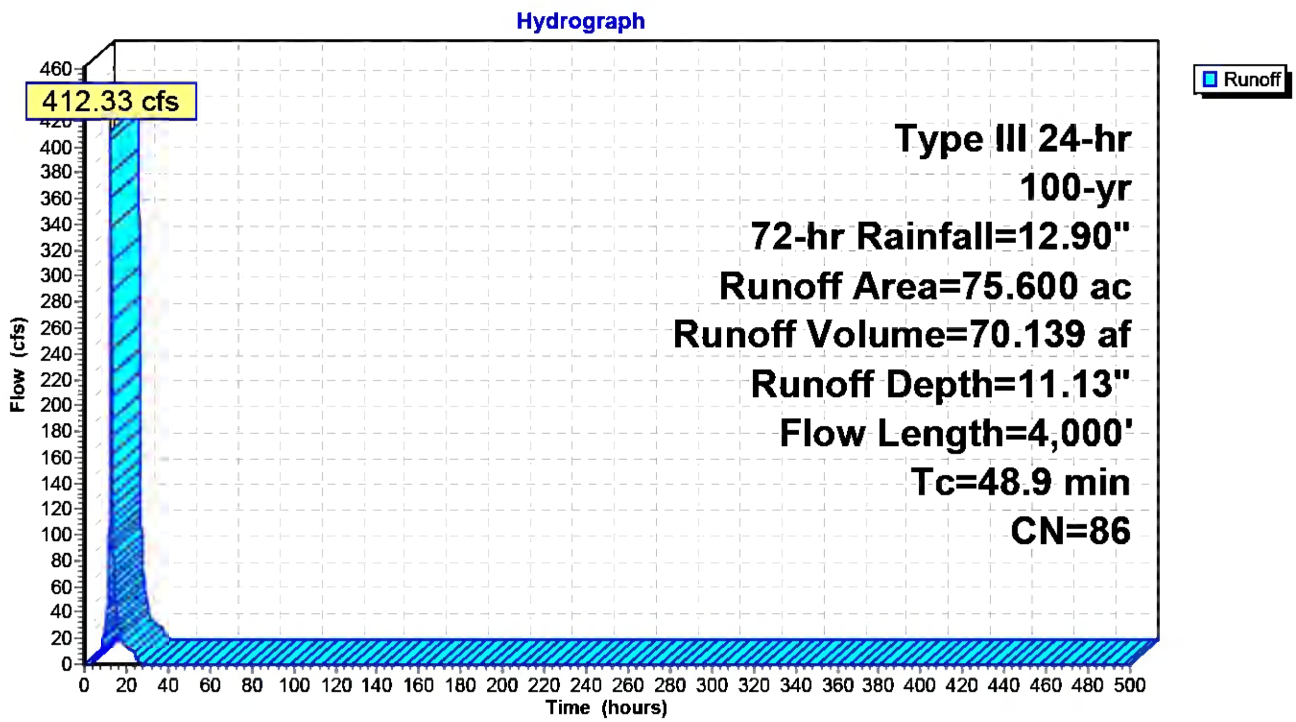
Runoff = 412.33 cfs @ 12.64 hrs, Volume= 70.139 af, Depth=11.13"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-500.00 hrs, dt= 0.05 hrs
Type III 24-hr 100-yr, 72-hr Rainfall=12.90"

	Area (ac)	CN	Description
*	75.600	86	
	75.600		100.00% Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
0.6	100	0.0700	2.66		**Sheet Flow,** Smooth surfaces n= 0.011 P2= 4.38"
48.3	3,900	0.0070	1.35		**Shallow Concentrated Flow,** Unpaved Kv= 16.1 fps
48.9	4,000	Total			

Subcatchment 8S: SAP



Summary for Pond 1P: SAP

Inflow Area = 75.600 ac, 0.00% Impervious, Inflow Depth = 11.13" for 100-yr, 72-hr event
Inflow = 412.33 cfs @ 12.64 hrs, Volume= 70.139 af
Outflow = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af, Atten= 100%, Lag= 0.0 min
Primary = 0.00 cfs @ 0.00 hrs, Volume= 0.000 af

Routing by Dyn-Stor-Ind method, Time Span= 0.00-500.00 hrs, dt= 0.05 hrs
Peak Elev= 27.06' @ 26.80 hrs Surf.Area= 13.195 ac Storage= 70.139 af

Plug-Flow detention time= (not calculated: initial storage exceeds outflow)
Center-of-Mass det. time= (not calculated: no outflow)

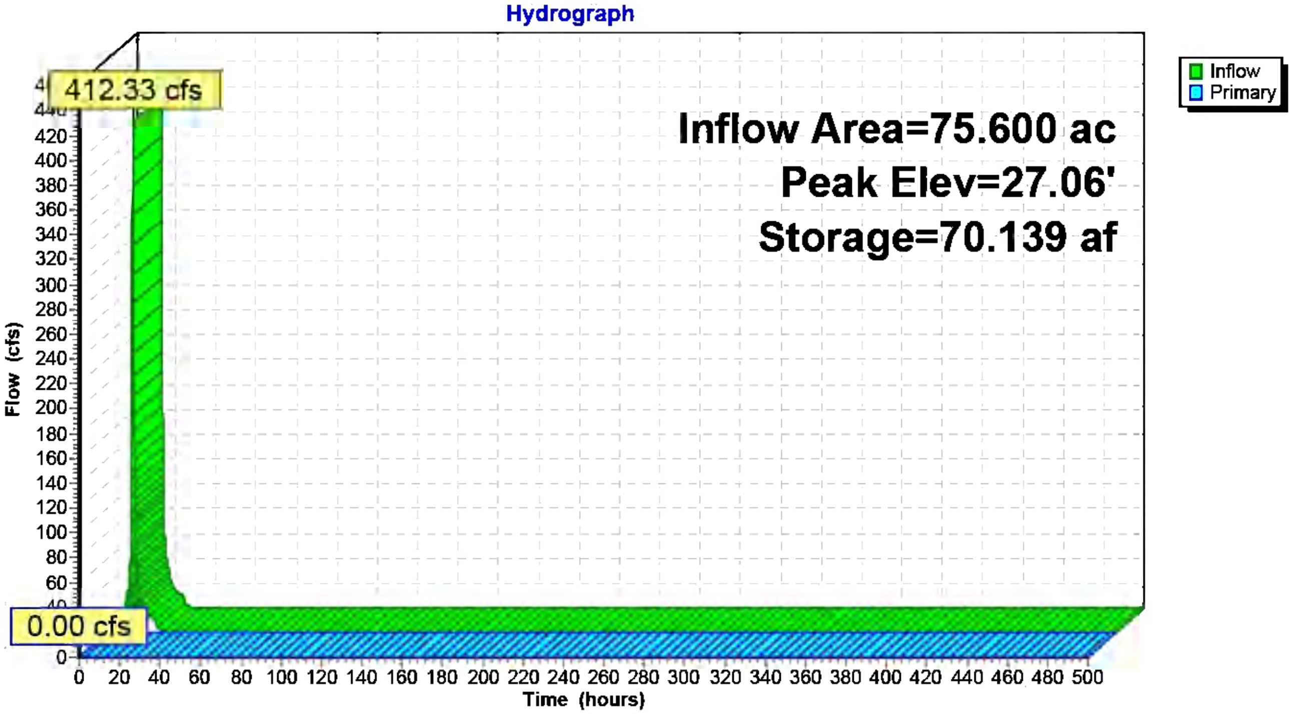
Volume	Invert	Avail.Storage	Storage Description
#1	15.00'	295.442 af	**Custom Stage Data (Prismatic)** Listed below (Recalc)

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
15.00	1.960	0.000	0.000
16.00	2.612	2.286	2.286
17.00	3.067	2.839	5.125
18.00	3.458	3.262	8.388
19.00	3.920	3.689	12.077
20.00	4.387	4.154	16.231
21.00	4.880	4.634	20.864
22.00	5.823	5.351	26.216
23.00	6.392	6.108	32.323
24.00	7.238	6.815	39.138
25.00	8.760	7.999	47.137
26.00	11.244	10.002	57.139
27.00	13.092	12.168	69.307
28.00	14.725	13.908	83.216
29.00	16.826	15.776	98.991
30.00	18.354	17.590	116.581
31.00	20.207	19.281	135.862
32.00	22.975	21.591	157.453
33.00	28.168	25.571	183.024
34.00	33.283	30.725	213.750
35.00	41.370	37.326	251.076
36.00	47.362	44.366	295.442

Device	Routing	Invert	Outlet Devices
#1	Primary	15.93'	**36.0" Round Culvert** L= 350.0' RCP, groove end w/headwall, Ke= 0.200 Inlet / Outlet Invert= 15.93' / 15.93' S= 0.0000 '/' Cc= 0.900 n= 0.013 Concrete pipe, bends & connections, Flow Area= 7.07 sf
#2	Device 1	27.73'	**4.0' long Sharp-Crested Rectangular Weir** 2 End Contraction(s)

Primary OutFlow Max=0.00 cfs @ 0.00 hrs HW=15.00' TW=23.15' (Dynamic Tailwater)
└1=**Culvert** (Controls 0.00 cfs)
  └2=**Sharp-Crested Rectangular Weir** (Controls 0.00 cfs)

Pond 1P: SAP



Elevation in NAVD 88

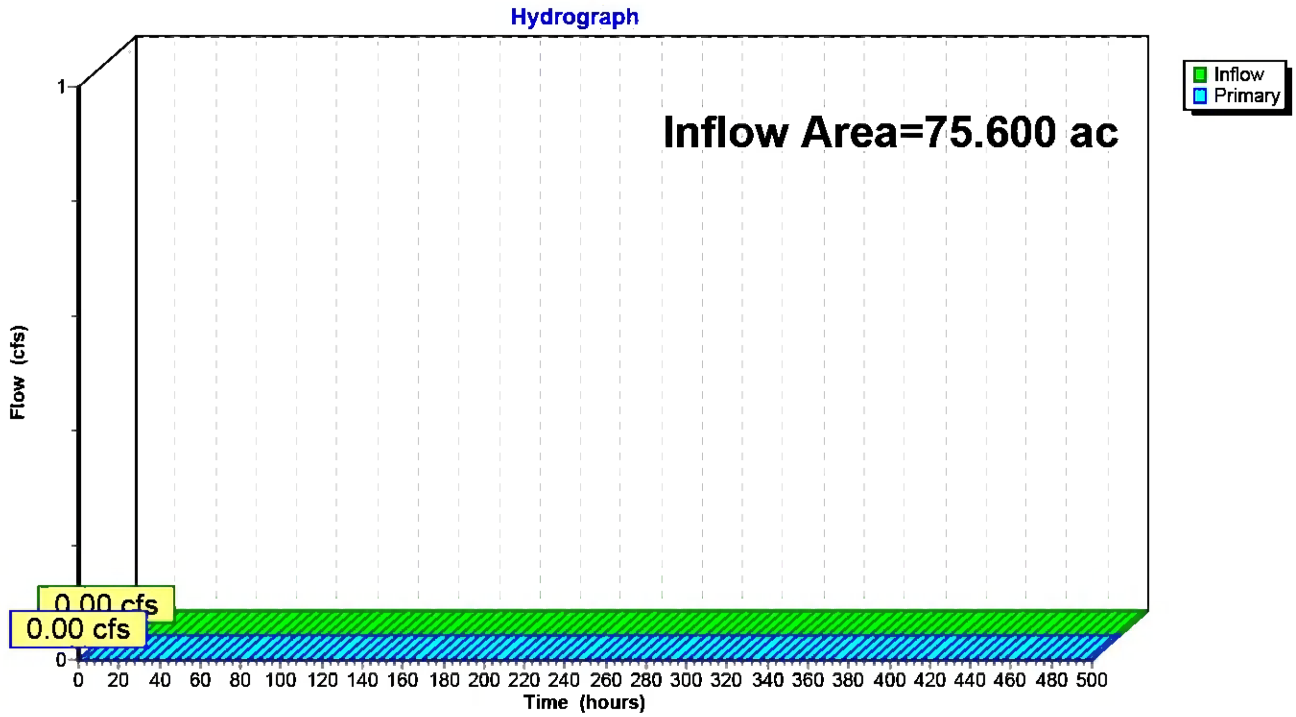
Summary for Link 7L: Discharge Canal

Inflow Area =	75.600 ac,	0.00% Impervious,	Inflow Depth = 0.00"	for 100-yr, 72-hr event	
Inflow =	0.00 cfs @	0.00 hrs,	Volume=	0.000 af	
Primary =	0.00 cfs @	0.00 hrs,	Volume=	0.000 af,	Atten= 0%, Lag= 0.0 min

Primary outflow = Inflow, Time Span= 0.00-500.00 hrs, dt= 0.05 hrs

Fixed water surface Elevation= 23.15'

Link 7L: Discharge Canal



ATTACHMENT 3
SEISMIC HAZARD EVALUATION AND SITE RESPONSE ANALYSIS

Written by:	M. Downing/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: SOUTH ASH 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 South Ash Pond at the Winyah Generating Station (WGS or Site). This calculation package is provided as Attachment 3 to the *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 profile of the South Ash 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 of 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 2 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 South Ash 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 year ground motion (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) uses pre-calculated hazard values at nearby grid locations and interpolates 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. As shown in Figure 4, this suite of six time histories provides a reasonable 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 updated accelerations and shear stresses at the critical profile observed in the 2016 Safety Factor Assessment (Geosyntec, 2016). Additionally, the site response analysis for the 2021 Safety Factor Assessment evaluates the impact of the expected range of water table elevations encountered within the South Ash Pond on the response of the Site. Calculated shear stresses are used to evaluate the liquefaction potential analysis (Attachment 4 of the 2021 Safety Factor Assessment Report) and seismic stability analysis (Attachment 5 of the 2021 Safety Factor Assessment Report).

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 material damping ratio (D). Preliminary equivalent-linear site response analyses yielded calculated maximum shear strains greater than five percent in some layers, which is 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 South Ash 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 specific to the site response analysis is presented herein.

To develop representative soil profiles, the South Ash Pond perimeter dike was divided into two sections comprised of dike fills and foundation soils. The soil stratigraphy is similar around the South Ash Pond. However, the profile on the west end of the South Ash Pond (Profile 1 shown in Figure 5) contains a layer of highly plastic foundation soils (Plasticity Index = 75) at depths between 36 and 48 ft below ground surface (bgs). Two representative profiles to 100 ft bgs were developed for the perimeter dike: (i) one for the west end of the South Ash Pond (Profile 1); and (ii) the other for the east end of the South Ash Pond (Profile 2). The 2016 Safety Factor Assessment identified zones of subsurface materials expected to undergo triggering of liquefaction and sections with lower calculated factors of safety for slope stability along the west end of the South Ash Pond (Geosyntec, 2016). Therefore, site response analyses were only performed for Profile 1, which contains the identified zone of potential liquefaction and sections in the 2016 Safety Factor Assessment, for the 2021 Safety Factor Assessment Report to provide an updated evaluation of the critical area of the South Ash Pond. The representative profile is shown in Figure 6.

A review of the topographic survey data from August 2021 (McKim & Creed, 2021) and the topographic survey used in the 2016 Safety Factor Assessment indicated that dewatering lowered the free water level in the east side of the South Ash Pond and CCR have been excavated from the east side of the pond (top of ash surface in the west side of the South Ash Pond is similar to the surface used for the 2016 Safety Factor Assessment). Santee Cooper provided available water level measurements from wells in the South Ash 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 South Ash Pond perimeter dikes, the water level within the perimeter dike may be similar to the water level used for the 2016 Safety Factor Assessment or lower due to dewatering in the east side of the pond.

The site response analyses for the South Ash Pond in the 2016 Safety Factor Assessment Report considered a water table 10 ft bgs. Additionally, a water table at the bottom of dike (i.e., 18 ft bgs) was modeled to account for a potentially lowered water table. Therefore, site response analyses were performed with the water table modeled at 10 and 18 ft bgs. Liquefaction potential analysis and seismic stability analysis are also performed for both water table elevations.

Profile 1 was 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 column (i.e., 500 ft bgs), but results are presented for the soil column to a depth of approximately 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 profile. Figure 8 shows the shallow (depth less than 100 ft bgs) V_s profile 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 weight of the dike fill was 125 pcf. The selected unit weight of the foundation soils was 115 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

Figure 9 shows calculated maximum shear strain and shear stress profiles for the case with the water table modeled at 10 ft bgs. For the case with the water table modeled at 18 ft bgs, Figure 10 shows the calculated maximum shear strain and shear stress profiles. 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 accelerations within the soil profiles are presented in Appendix 4. The envelopes of maximum shear strain and shear stress for the six motions for each water table elevation are presented in Figure 11. The calculated envelopes of maximum shear stress (τ_{max}) values for different depths are presented in Table 2 for each water table elevation modeled. The calculated maximum shear strain and shear stress profiles are slightly affected by the change in water table elevation, particularly at depths less than 30 ft bgs. The calculated maximum shear stresses are slightly larger for the case with the water table modeled at 18 ft bgs. These values were used to calculate cyclic stress ratios for the evaluation of liquefaction potential (Attachment 4 in the Safety Factor Assessment Report) and to calculate the seismic coefficient for seismic stability analyses (Attachment 5 in the Safety Factor Assessment Report).

CONCLUSIONS

- The design PGA was conservatively selected to be 0.15g. 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 3.

- 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 site response analyses.

- Nonlinear site response analyses were conducted using DEEPSOIL® (Hashash et al., 2020). The critical soil profile identified in the 2016 Safety Factor Assessment was used for the site response analyses. The analyses used region-specific shear modulus reduction and damping curves. The shear wave velocity profile was estimated from measured SCPT values and correlations between V_s and measured CPT sleeve frictions. Two water levels were considered for the analyses based on the review of information provided by Santee Cooper. The inputs used for the profile in DEEPSOIL® are shown in Appendix 5.

- The site response analysis results are presented in Figures 9 through 11. The calculated maximum shear stresses for each water table elevation are presented in Table 2 and are used for evaluation of soil liquefaction potential and calculation of the seismic coefficient for seismic stability analyses for both water table elevations.

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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 – WT 10 ft bgs		Profile 1 – WT 18 ft bgs	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	29	2.5	47
7.5	67	7.5	104
12.5	90	12.5	135
16.5	103	16.5	156
19.5	114	19.5	179
23.5	149	23.5	199
28.5	178	28.5	210
33.5	202	33.5	219
38.0	269	38.0	278
42.0	307	42.0	318
46.0	334	46.0	344
50.5	380	50.5	374
58.0	480	58.0	473
68.0	597	68.0	591
78.0	708	78.0	710
88.0	816	88.0	824
98.0	929	98.0	934
108.0	1060	108.0	1064

Figures

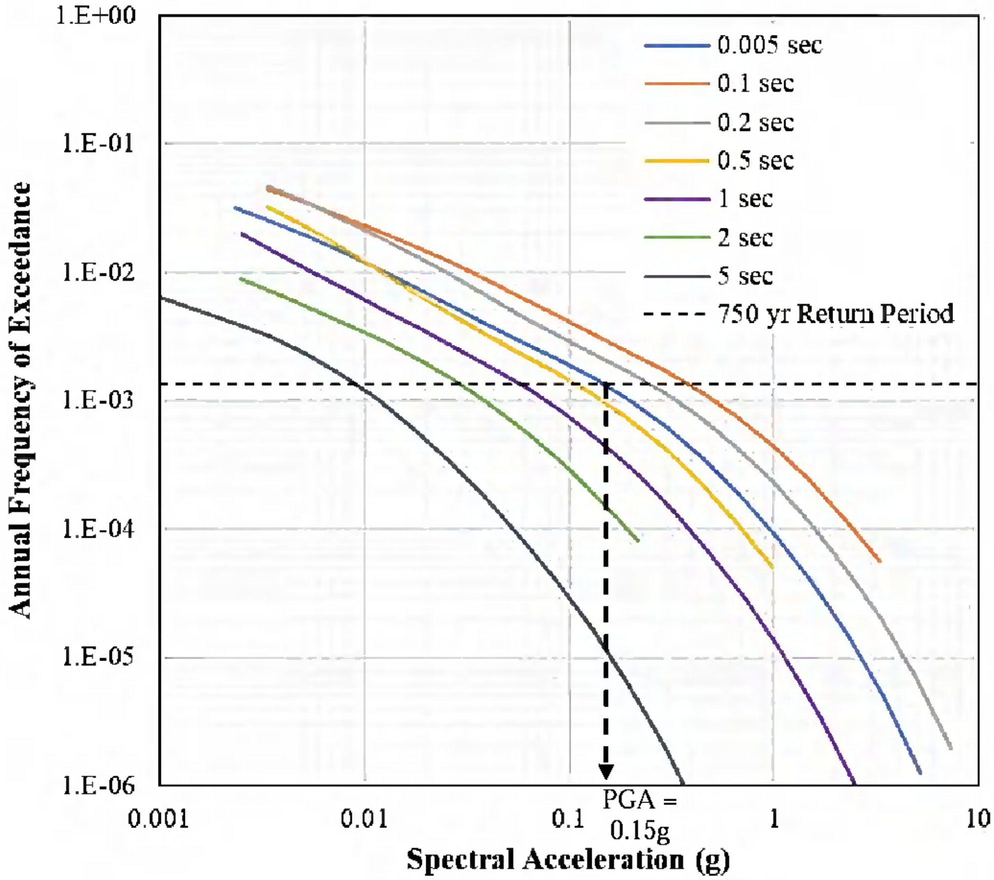
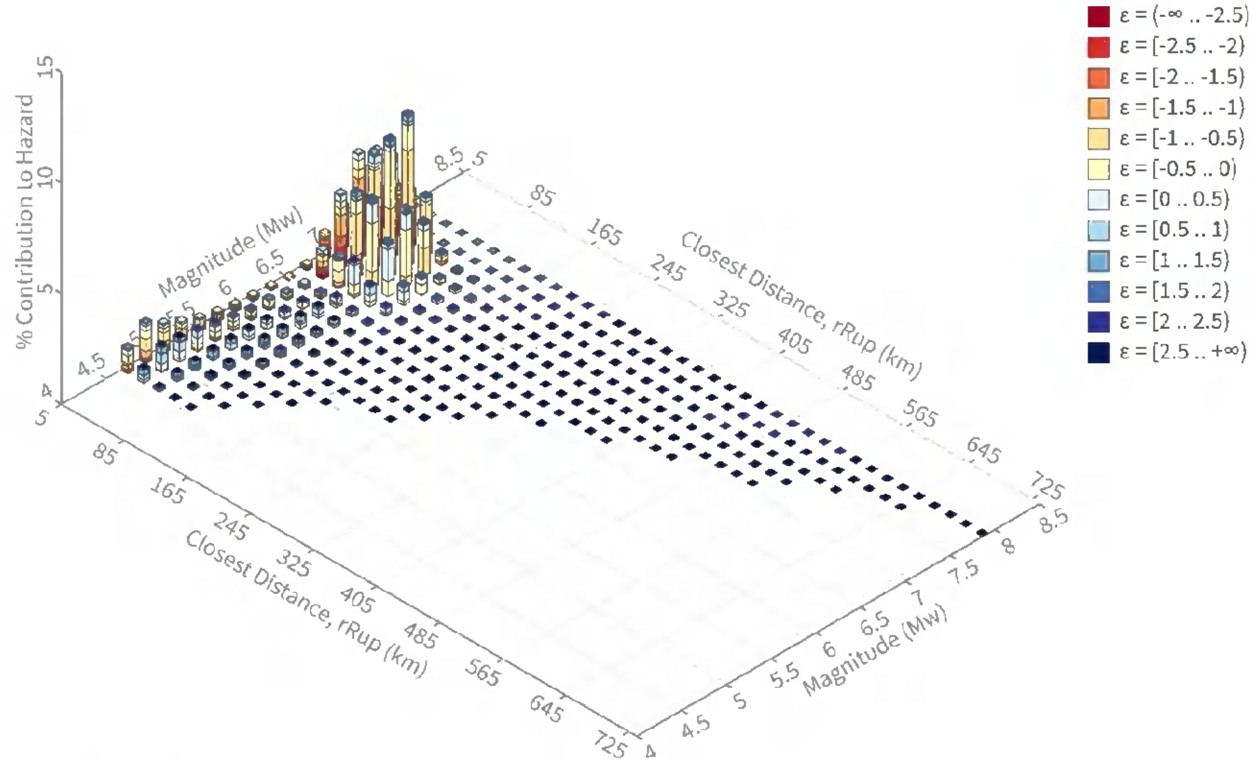


Figure 1. USGS Hazard Curves for Geologically Realistic Conditions (BC Boundary) at the Site (Petersen et al., 2019)

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

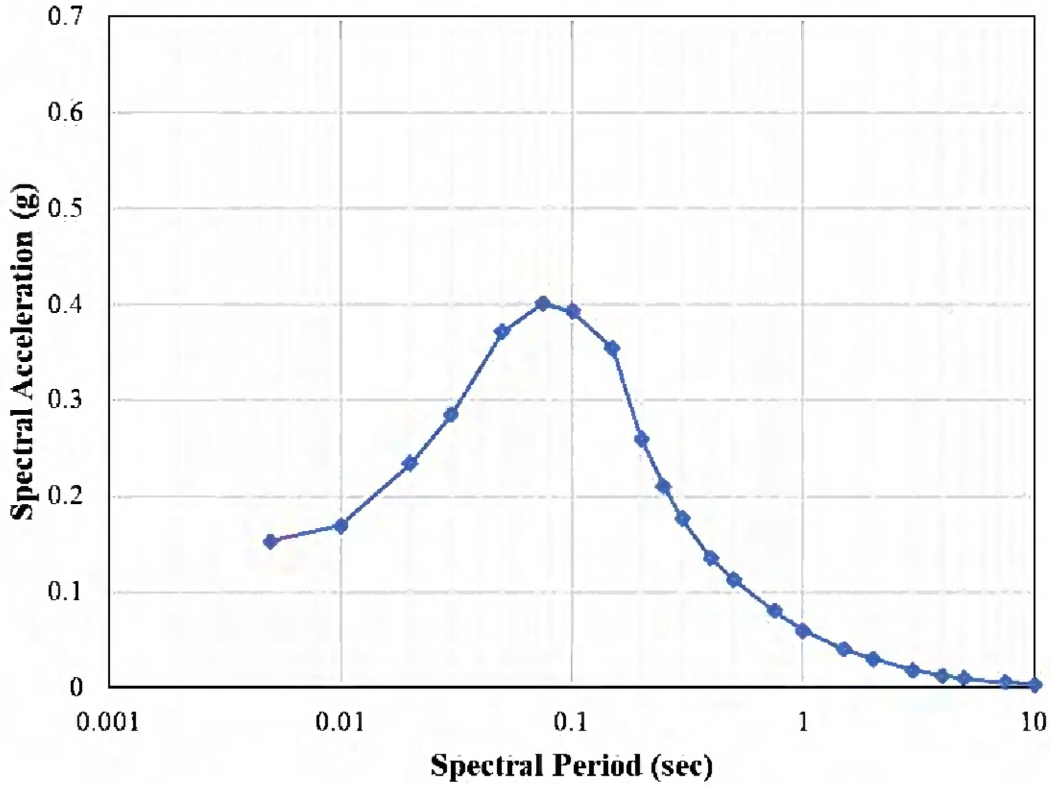


Figure 3. "Geologically Realistic" Target Response Spectrum for WGS

Note:

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

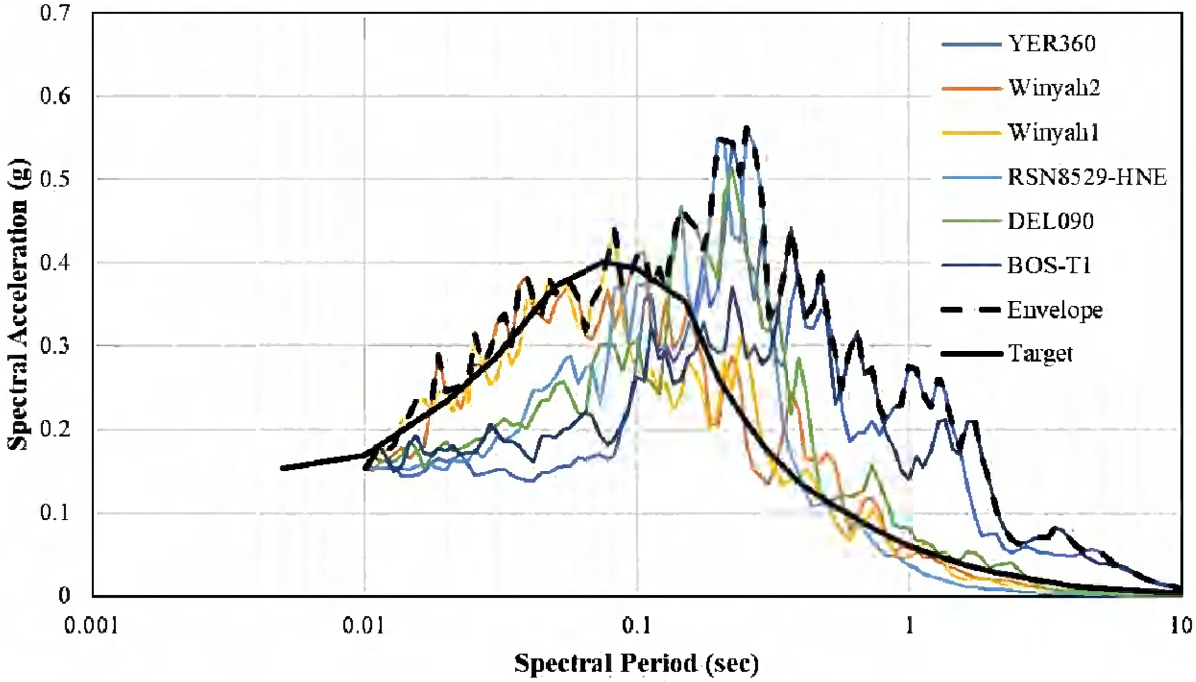


Figure 4. Response Spectra of Scaled Time Histories Selected for Seismic Evaluations

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

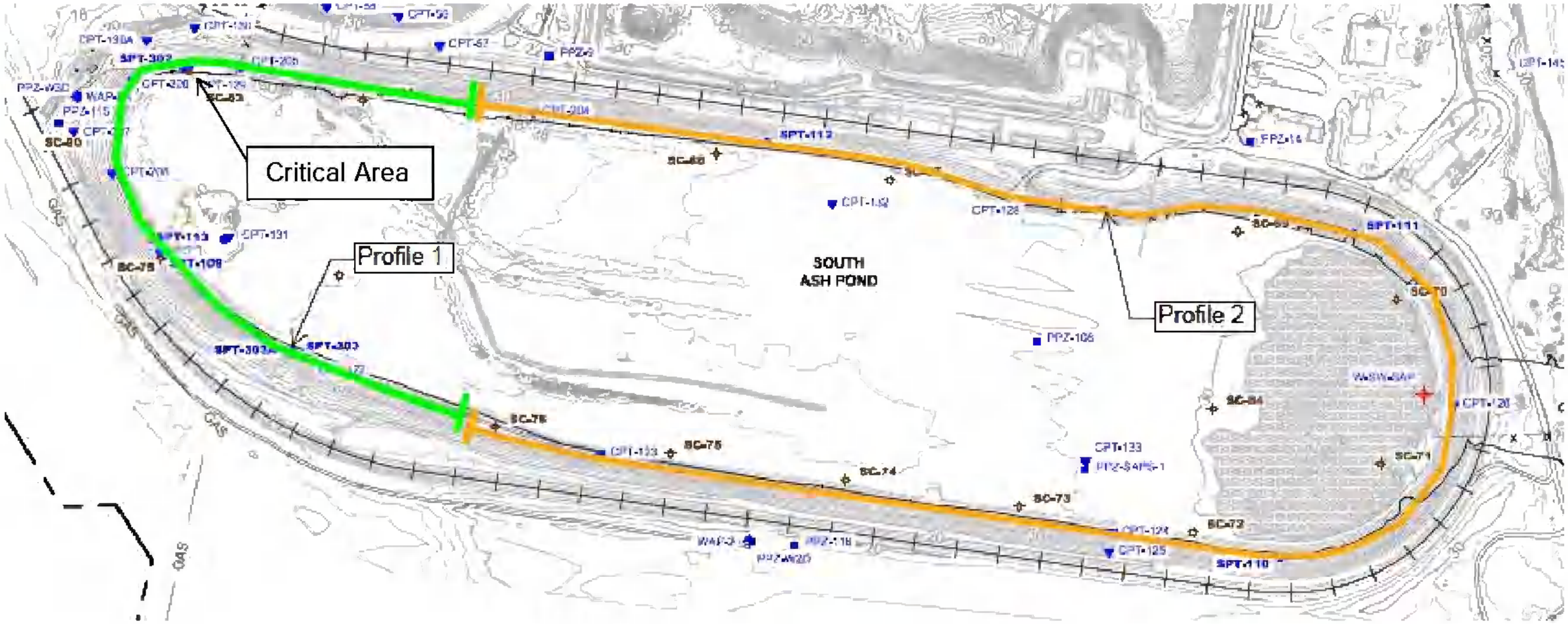


Figure 5. Locations of Representative Soil Profiles

Note:

1. The site response analyses were performed for Profile 1 only.

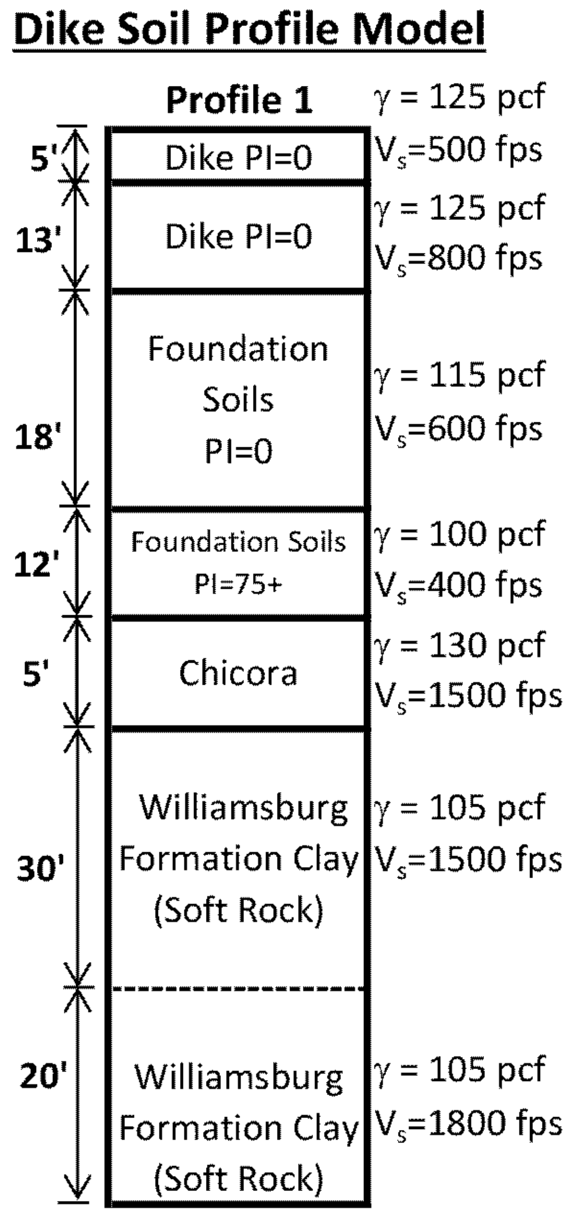


Figure 6. DEEPSOIL® Soil Profile

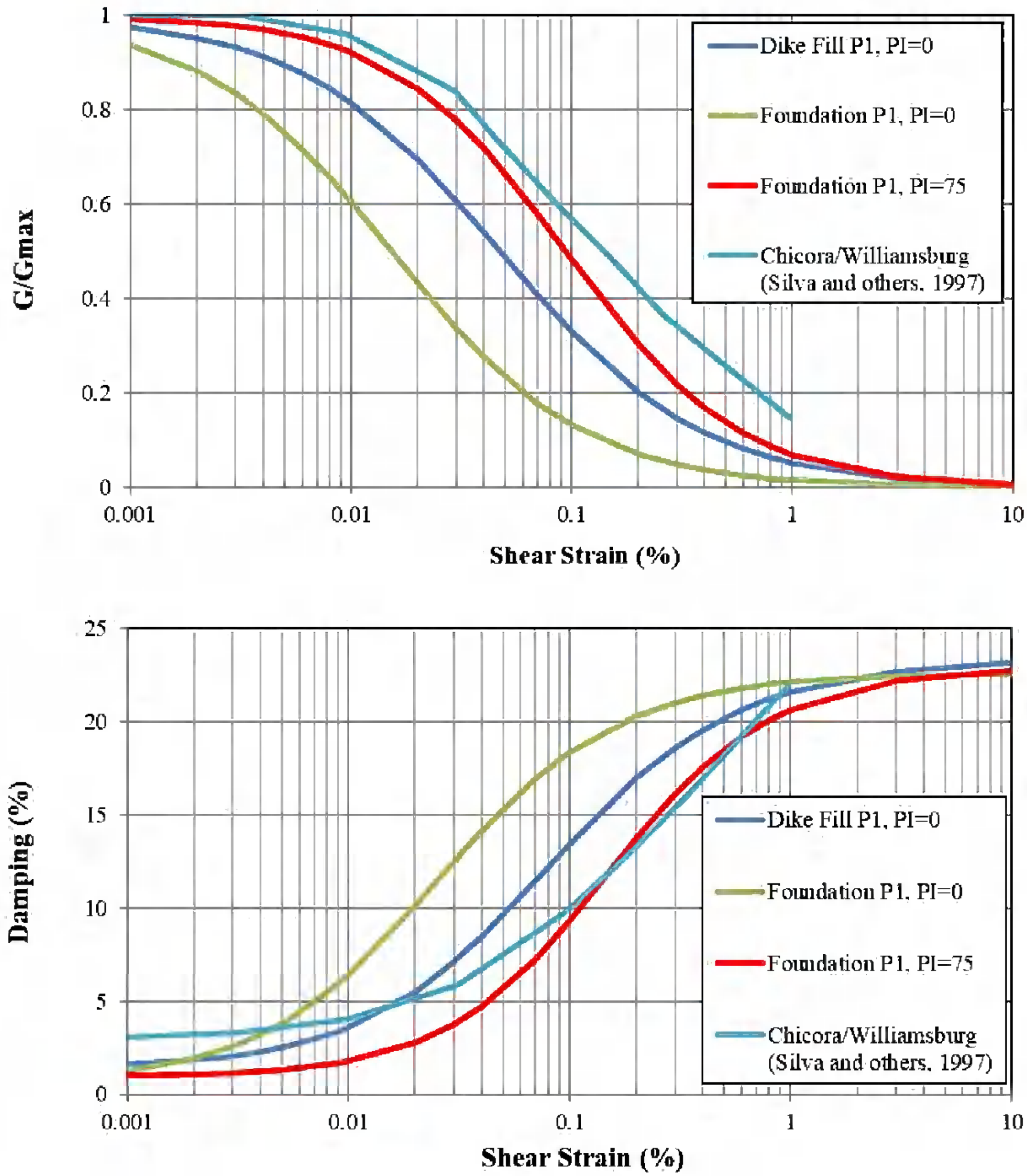


Figure 7. Shear Modulus Reduction (G/G_{max}) and Damping Curves for Soils Used in Site Response Analyses

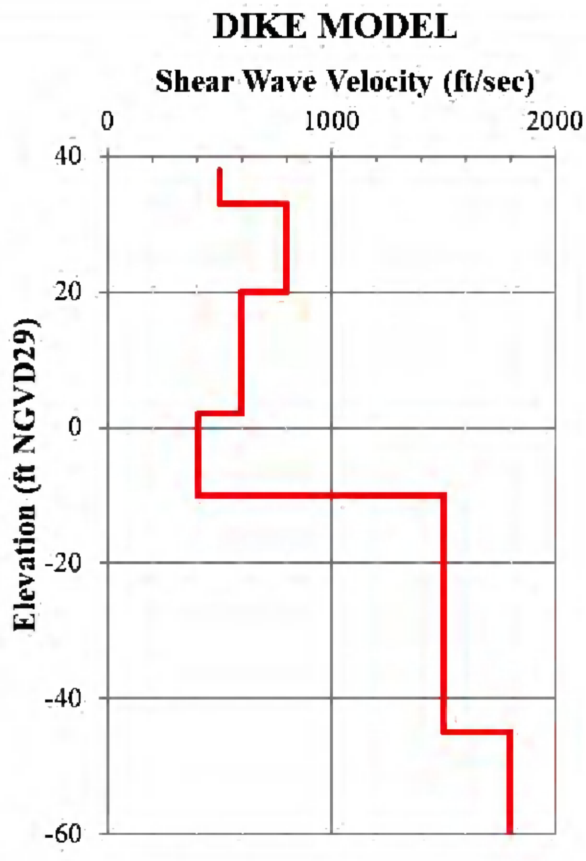


Figure 8. Selected Shear Wave Velocity (V_s) Profile for Site Response Analyses

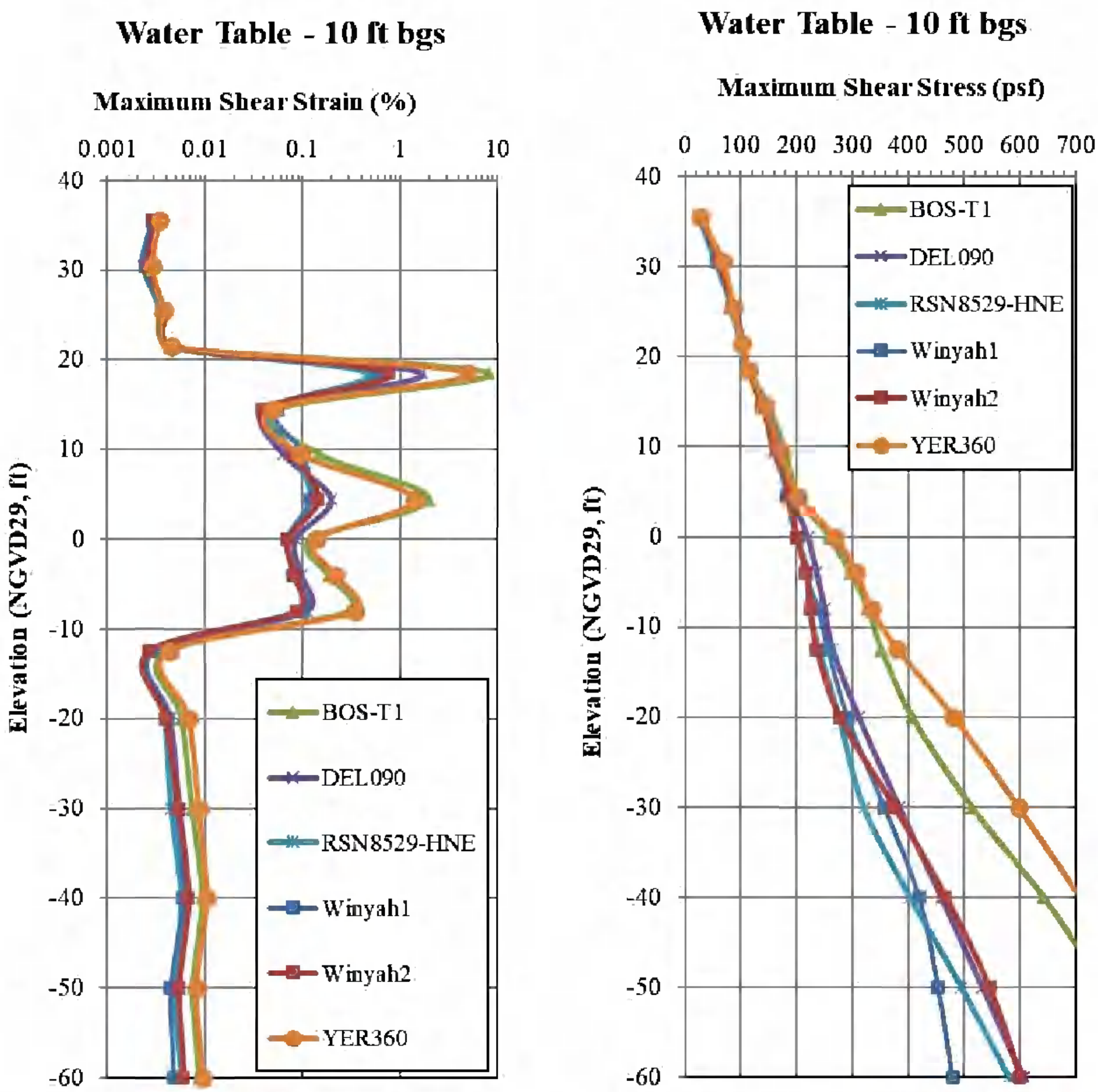


Figure 9. Site Response Analysis Results for Profile 1 with Water Table 10 ft bgs

Note:
1. bgs = below ground surface.

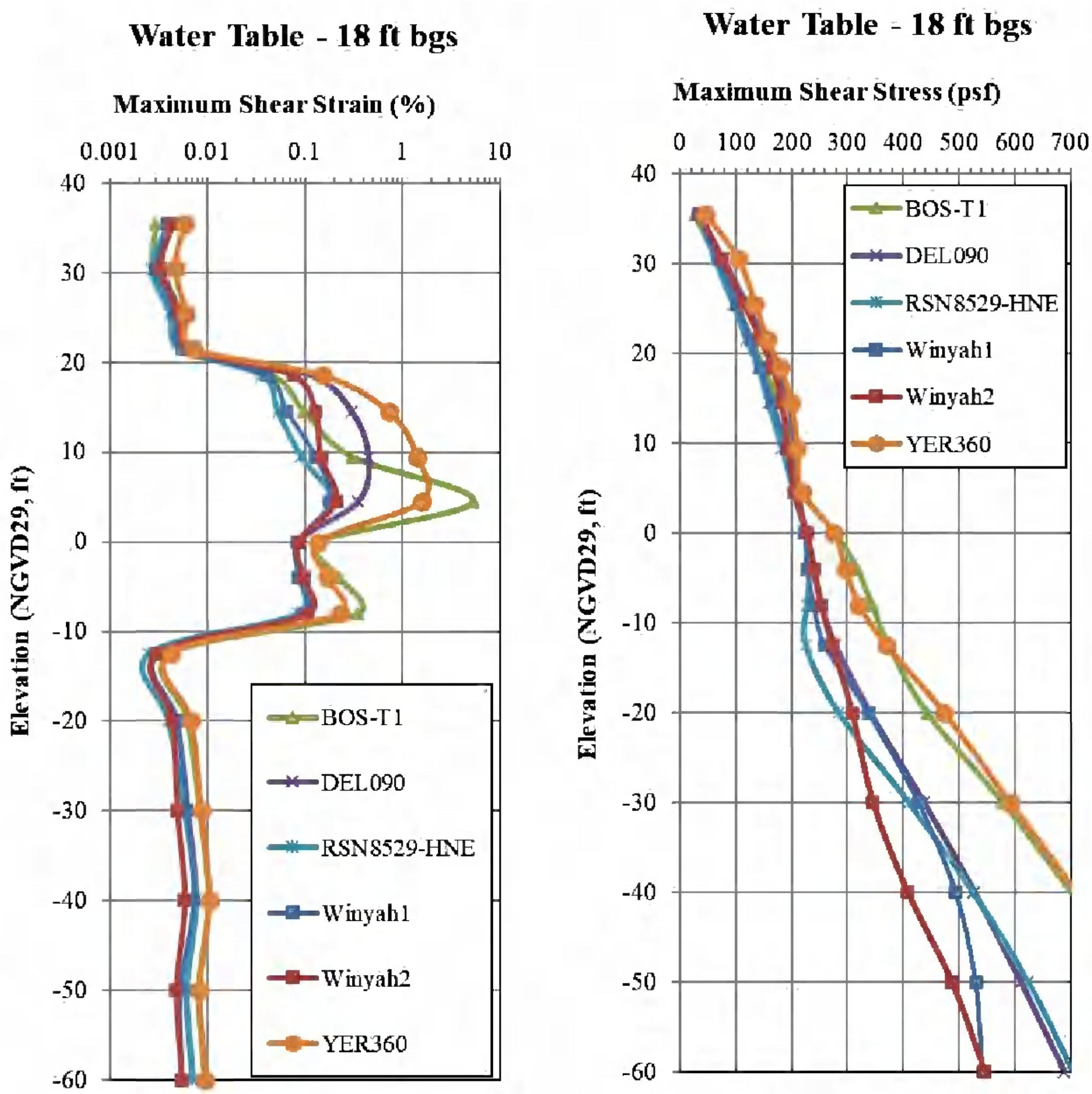


Figure 10. Site Response Analysis Results for Profile 1 with Water Table 18 ft bgs

Note:

1. bgs = below ground surface.

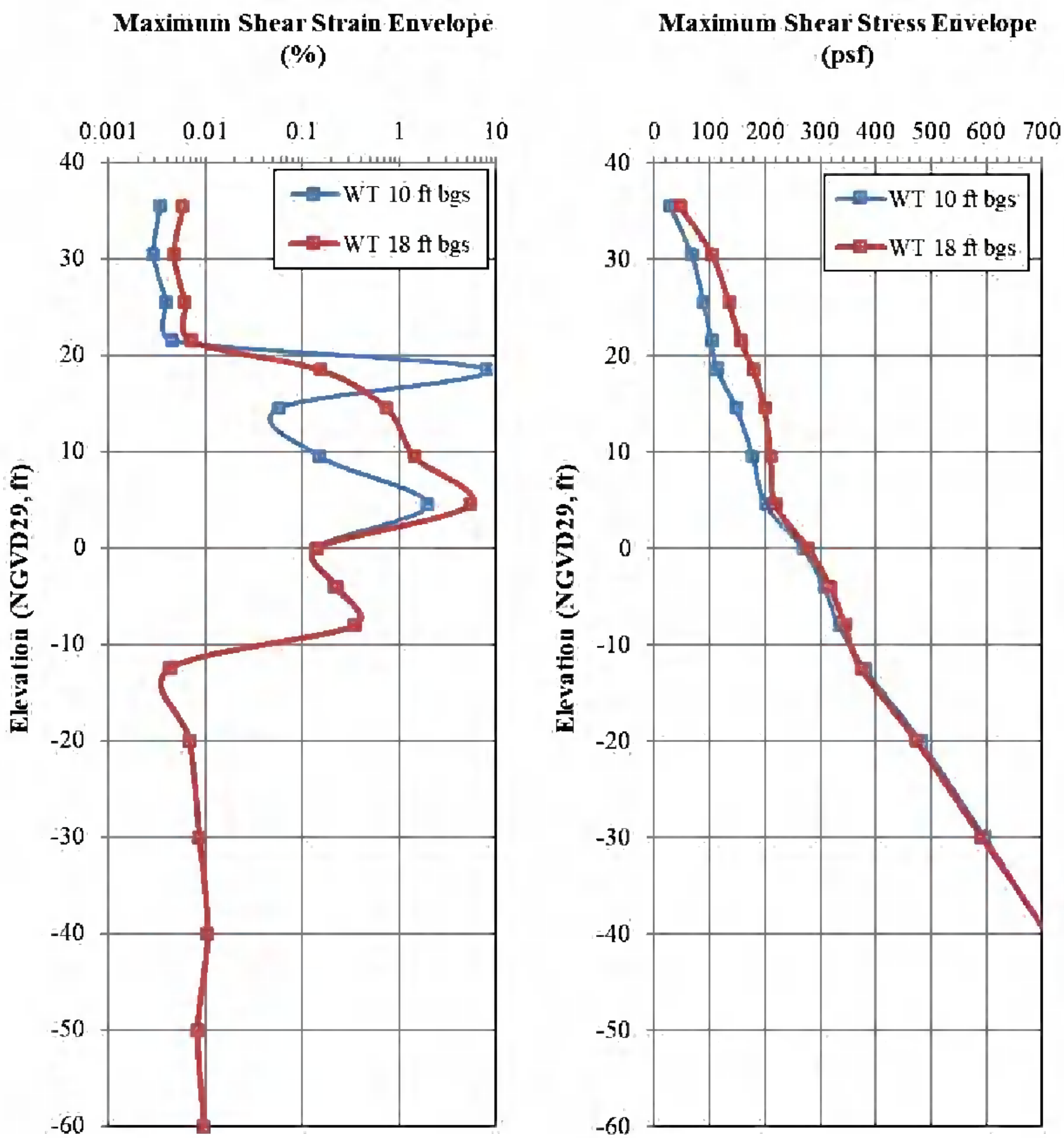


Figure 11. Maximum Shear Strain and Shear Stress Envelopes for Each Water Table Elevation

Note:
1. bgs = below ground surface; and WT = water table.

Appendix 1

Selected Time Histories

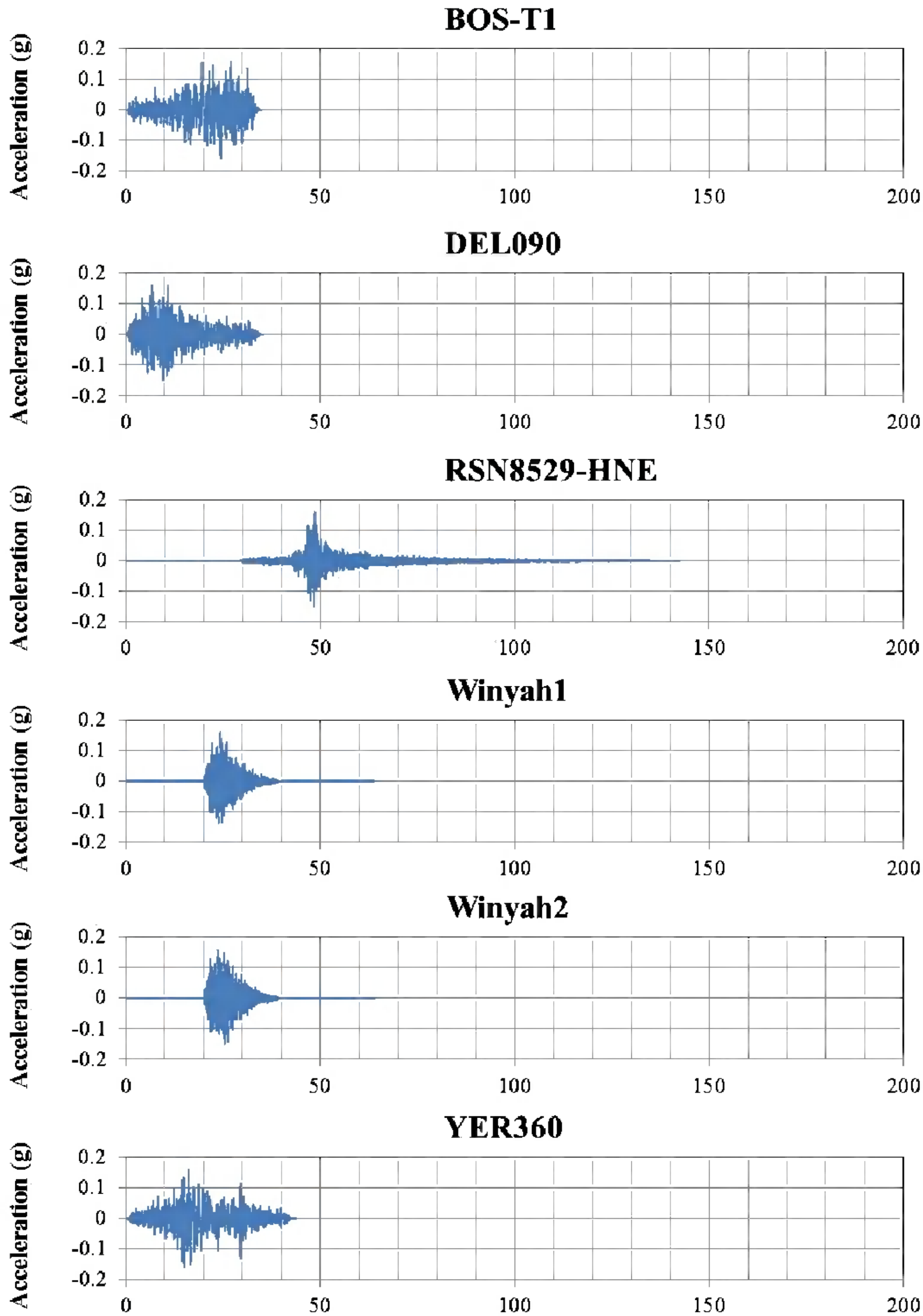


Figure 1-1. Acceleration Time Histories of Selected Earthquake Motions Scaled to PGA of 0.15g

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 (2019). 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 high plasticity foundation soil in Profile 1

(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: clayey soils with PI=75; groundwater table @ 10 ft bgs.

Step 3 – calculate σ_m'@ mid-depth of the layer (42 ft bgs)

$\sigma_v' = \gamma H - \gamma_w H_w = 125\times18 + 115\times18 + 100\times6 - 62.4\times32 = 2923.2$ psf

$\sigma_m' = \sigma_v' (1+2K'_o)/3 = 2923.2\times(1+2\times0.675)/3 = 2289.8$ psf

$(K_o' = 0.6+0.001\times PI = 0.6+0.001\times75 = 0.675)$

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 entirety of the high plasticity foundation soils in Profile 1.

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

$\gamma_{r1} = 0.092\%$, $\alpha = 1.10$, $k = 0.2$

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

$\gamma_r = \gamma_{r1} (\sigma_m'/P_a)^k = 0.092\times(2289.8/2089)^{0.2} = 0.0937\%$

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 $\gamma = 0.001\%$, $G/G_{max} = 1/[1+(0.001/0.0937)] = 0.989$
If $\gamma = 0.01\%$, $G/G_{max} = 1/[1+(0.01/0.0937)] = 0.904$
If $\gamma = 0.1\%$, $G/G_{max} = 1/[1+(0.1/0.0937)] = 0.484$

Damping Curve for the high plasticity foundation soil in Profile 1

(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.

$D_{min1} = 0.96\%$

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

$D_{min} = D_{min1}\ (\sigma_m'/P_a)^{-0.5k} = 0.96\times(2289.8/2089)^{-0.5\times0.2} = 0.951\%$

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 $\gamma = 0.001\%$, $D = 12.2\times(0.989)^2 - 34.2\times(0.989) + 22.0 + 0.951 = 1.06\%$
If $\gamma = 0.01\%$, $D = 12.2\times(0.904)^2 - 34.2\times(0.904) + 22.0 + 0.951 = 2.00\%$
If $\gamma = 0.1\%$, $D = 12.2\times(0.484)^2 - 34.2\times(0.484) + 22.0 + 0.951 = 9.26\%$

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)

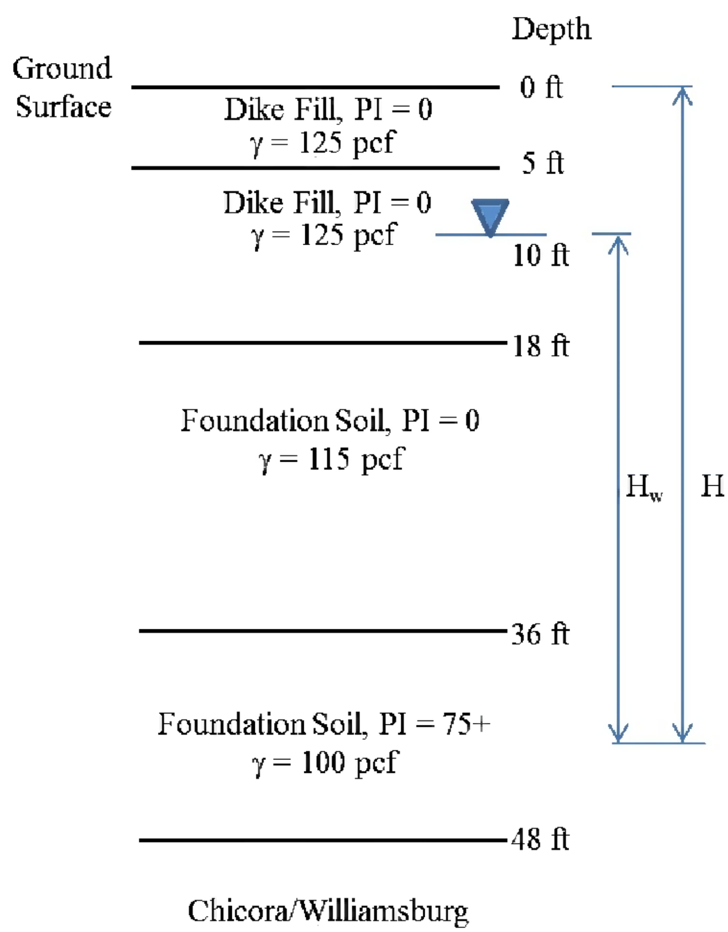


Figure 2-2. Profile 1 for the Example Calculations

$$ {}^{G}/_{G_{max}} = \frac{1}{1+\left(\frac{\gamma_c}{\gamma_{cr}}\right)^{\alpha}} $$ **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)

Note:

1. Values for PI=75 were linearly interpolated between values for PI=50 and PI=100.

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 (SCDOT, 2019)

Note:

1. Values for PI=75 were linearly interpolated between values for PI=50 and PI=100.

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

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

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

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

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

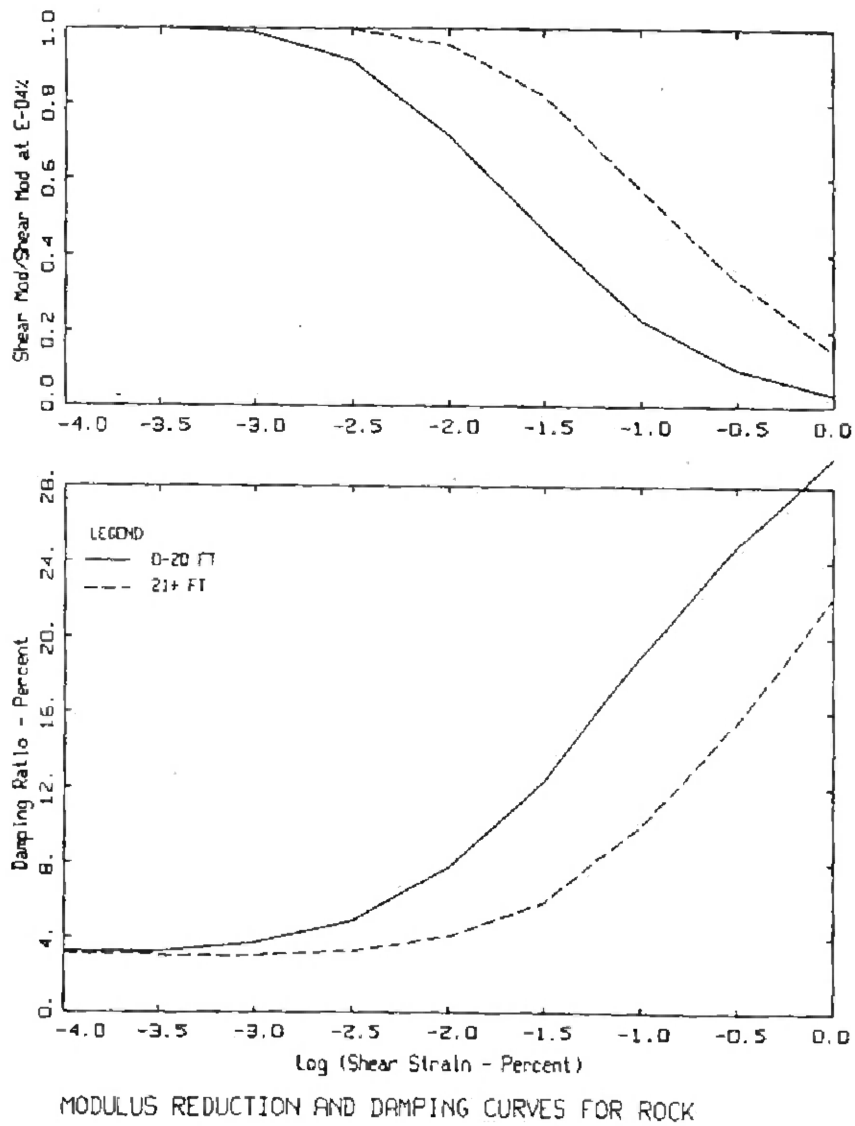


Figure 5b. Generic G/Gmax and hysteretic damping curves for soft rock (Silva et al., 1997).

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

Appendix 3

Shear Wave Velocity Profile Selection

Geosyntec consultants

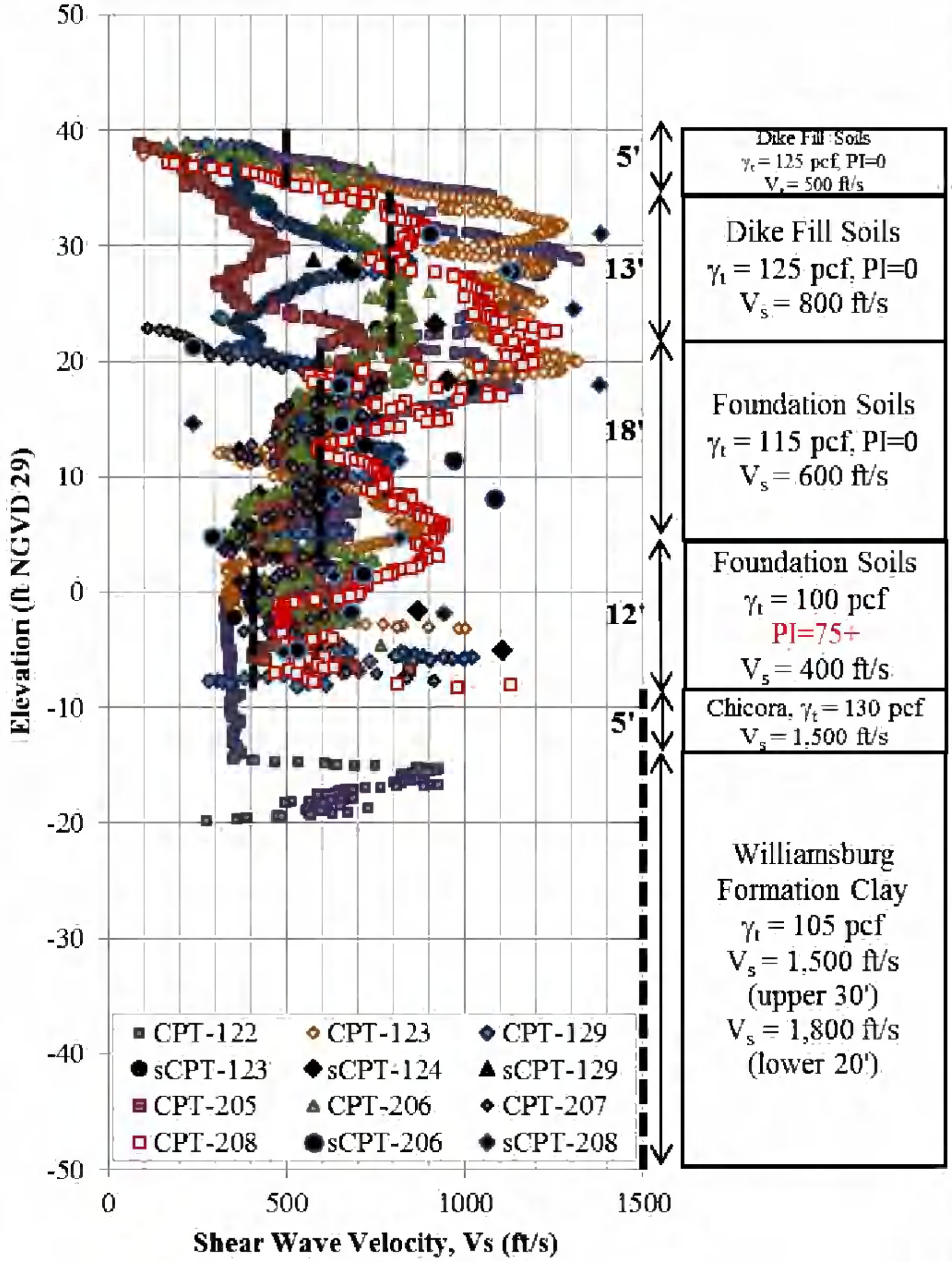


Figure 3-1. Selected V_s Profile for Profile 1

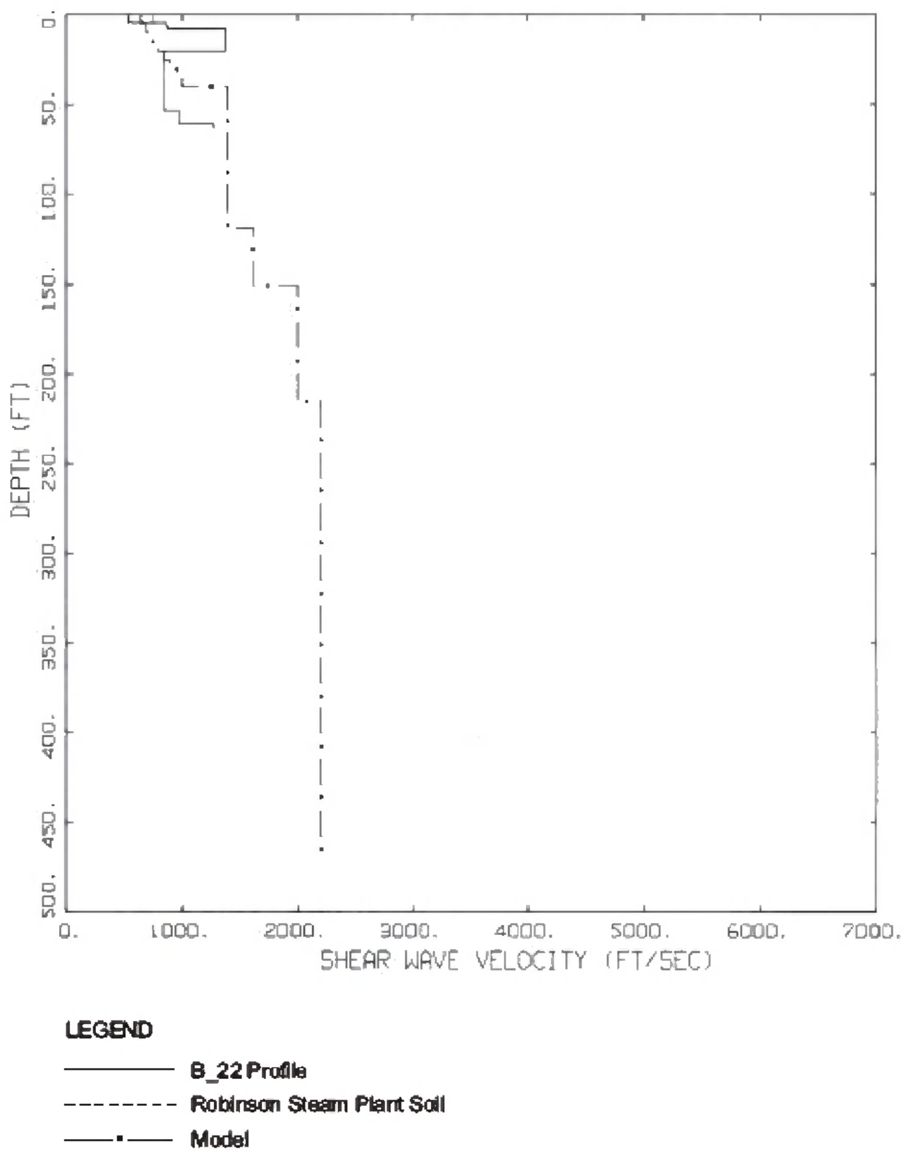


Figure 3-9. Base case shear-wave velocity profile for the Myrtle Beach site response category along with available profiles.

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

Appendix 4

Calculated Acceleration Profile

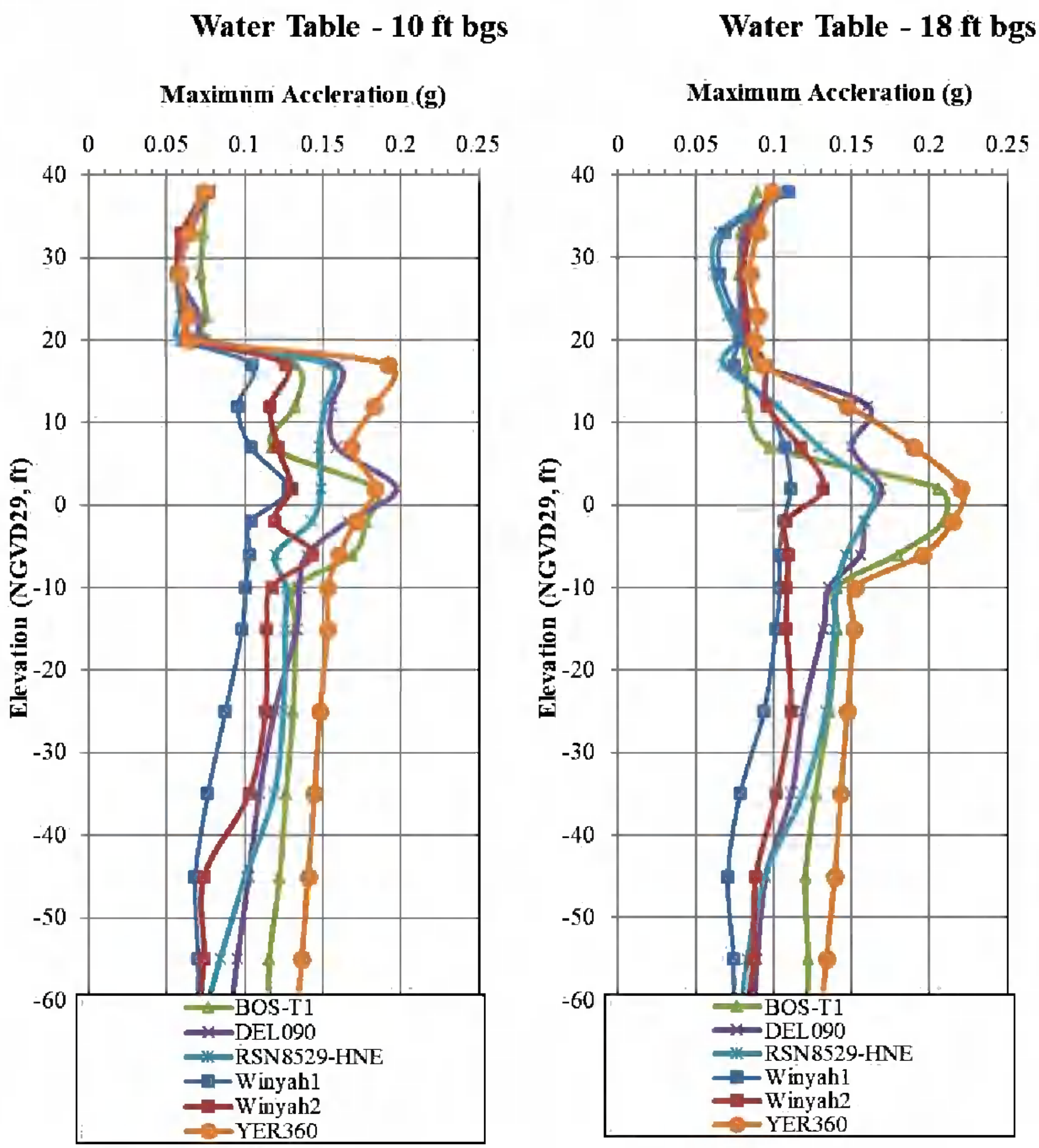


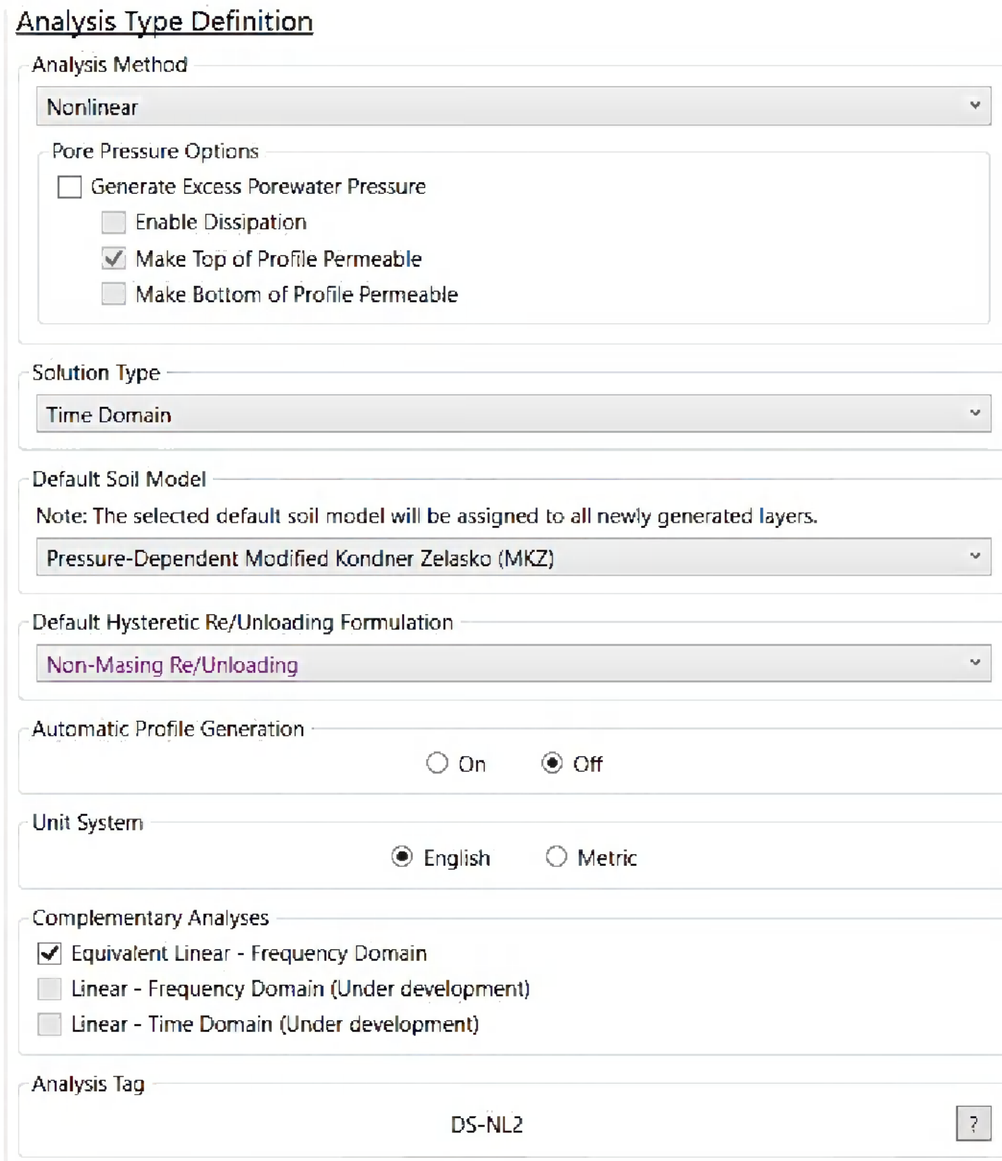
Figure 4-1. Calculated Maximum Accelerations for Profile 1

Note:

1. The input motions were applied as an outcrop motion with a PGA of 0.15g.
2. bgs = below ground surface.

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
?

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
1	Dike	5	125	500		MKZ	1.71909476151	0.0476	0.18	1.605	0.945
2	Dike	5	125	800		MKZ	1.39317391459	0.07120000000	0.18	1.575	0.945
3	Dike	5	125	800		MKZ	1.28911418288	0.08240000000	0.18	1.56	0.945
4	Dike	3	125	800		MKZ	1.24931701456	0.08600000000	0.18	1.53	0.945
5	Foundation Soil	3	115	600		MKZ	0.64382982210	0.0146	0.18	1.605	1.005
6	Foundation Soil	5	115	600		MKZ	0.63772348238	0.0234	0.18	1.605	1.005
7	Foundation Soil	5	115	600		MKZ	0.62110624127	0.0238	0.18	1.545	1.005
8	Foundation Soil	5	115	600		MKZ	0.60643436457	0.025	0.18	1.545	1.005
9	HP Foundation	4	100	400		MKZ	0.95428719296	0.1366	0.18	1.53	1.095
10	HP Foundation	4	100	400		MKZ	0.94927815241	0.1394	0.18	1.545	1.095
11	HP Foundation	4	100	400		MKZ	0.94453259942	0.1382	0.18	1.515	1.095
12	Chicora	5	130	1500		MKZ	3.01770846178	0.234	0.18	1.575	0.96
13	Williamsburg Fc	10	105	1500		MKZ	3.01770846178	0.234	0.18	1.575	0.96
14	Williamsburg Fc	10	105	1500		MKZ	3.01770846178	0.234	0.18	1.575	0.96
15	Williamsburg Fc	10	105	1500		MKZ	3.01770846178	0.234	0.18	1.575	0.96
16	Williamsburg Fc	10	105	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96
17	Williamsburg Fc	10	105	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96
18	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96
19	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96
20	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96
21	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96
22	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96

☑ Water table at top of layer: 3 Add Layer(s) Remove Layer(s)

Layer Properties | Advanced Table View

d	Θ1	Θ2	Θ3	Θ4	Θ5	A	γ1	Reduction Factor Formulation	P1	P2	P3
								MRDF-UIUC	0.644	0.24	3.25
								MRDF-UIUC	0.646	0.24	3.25
								MRDF-UIUC	0.646	0.24	3.25
								MRDF-UIUC	0.646	0.24	3.25
								MRDF-UIUC	0.62	0.264	3.25
								MRDF-UIUC	0.62	0.264	3.25
								MRDF-UIUC	0.618	0.26	3.25
								MRDF-UIUC	0.618	0.26	3.25
								MRDF-UIUC	0.598	0.284	2.9
								MRDF-UIUC	0.598	0.284	2.9
								MRDF-UIUC	0.598	0.284	2.9
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2

☑ Water table at top of layer: 3 | Add Layer(s) | Remove Layer(s)

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
23	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96
24	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96
25	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96
26	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96
27	Williamsburg Fc	10	125	1800		MKZ	3.01770846178	0.234	0.18	1.575	0.96
28	Williamsburg Fc	10	125	2000		MKZ	3.01770846178	0.234	0.18	1.575	0.96
29	Williamsburg Fc	10	125	2000		MKZ	3.01770846178	0.234	0.18	1.575	0.96
30	Williamsburg Fc	10	125	2000		MKZ	3.01770846178	0.234	0.18	1.575	0.96
31	Williamsburg Fc	10	125	2000		MKZ	3.01770846178	0.234	0.18	1.575	0.96
32	Williamsburg Fc	10	125	2000		MKZ	3.01770846178	0.234	0.18	1.575	0.96
33	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
34	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
35	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
36	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
37	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
38	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
39	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
40	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
41	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
42	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
43	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
44	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96

☑ Water table at top of layer: 3 Add Layer(s) Remove Layer(s)

Layer Properties | Advanced Table View

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

☑ Water table at top of layer: 3 | Add Layer(s) | Remove Layer(s)

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
36	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
37	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
38	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
39	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
40	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
41	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
42	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
43	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
44	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
45	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
46	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
47	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
48	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
49	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
50	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
51	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
52	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
53	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
54	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
55	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
56	Williamsburg Fc	10	125	2200		MKZ	3.01770846178	0.234	0.18	1.575	0.96
57	Williamsburg Fc	10	125	2200		MKZ	3.15051070328	0.081200000000	0.18	1.515	0.975

☑ Water table at top of layer: 3 | Add Layer(s) | Remove Layer(s)

Layer Properties | Advanced Table View

d	Θ1	Θ2	Θ3	Θ4	Θ5	A	γ1	Reduction Factor Formulation	P1	P2	P3
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
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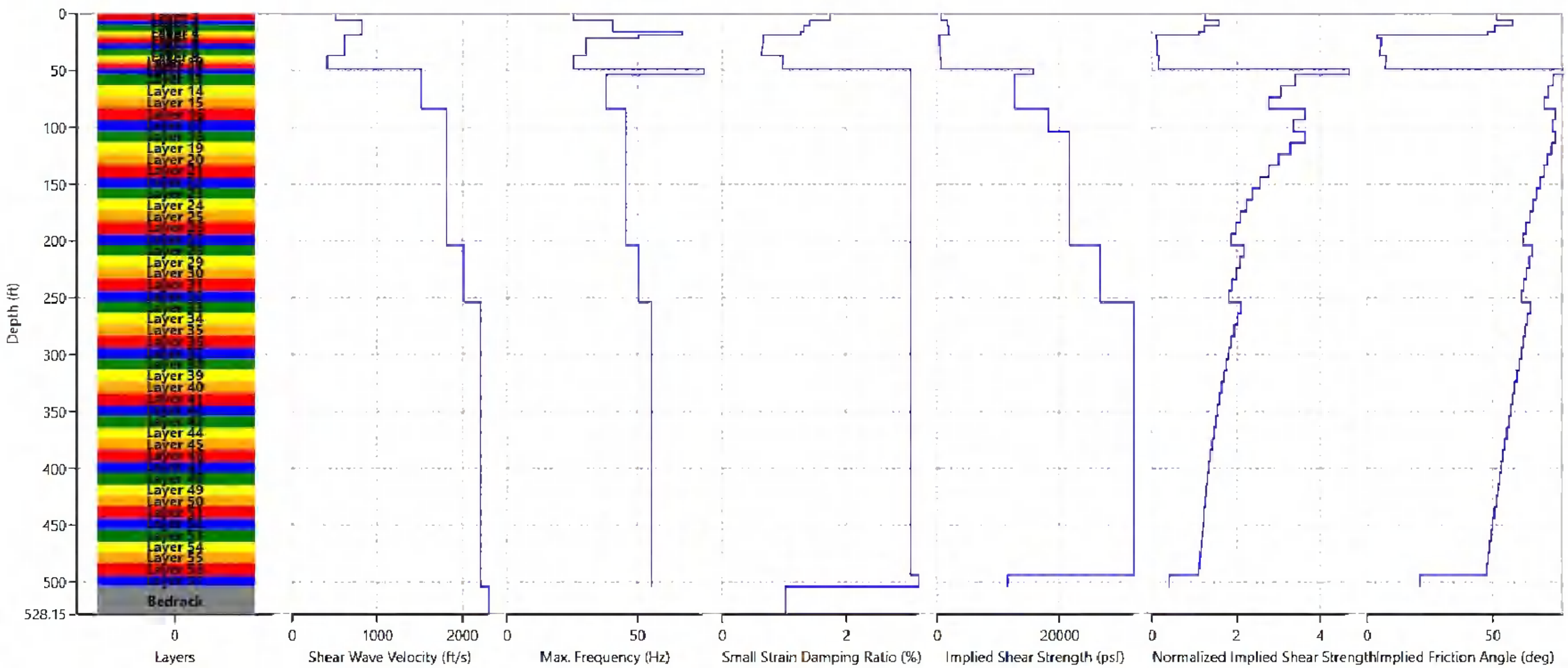
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.676	0.254	2
								MRDF-UIUC	0.81	0.308	0.65

☑ Water table at top of layer: 3 Add Layer(s) Remove Layer(s)

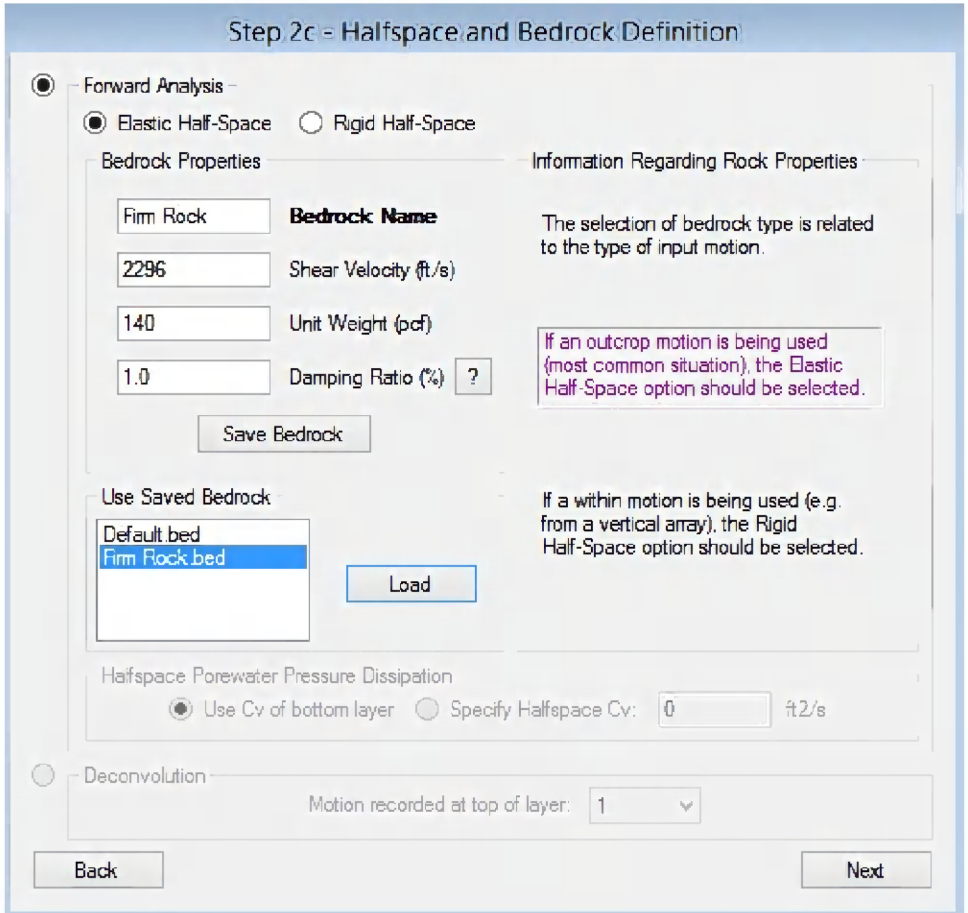
Soil Profile Definition

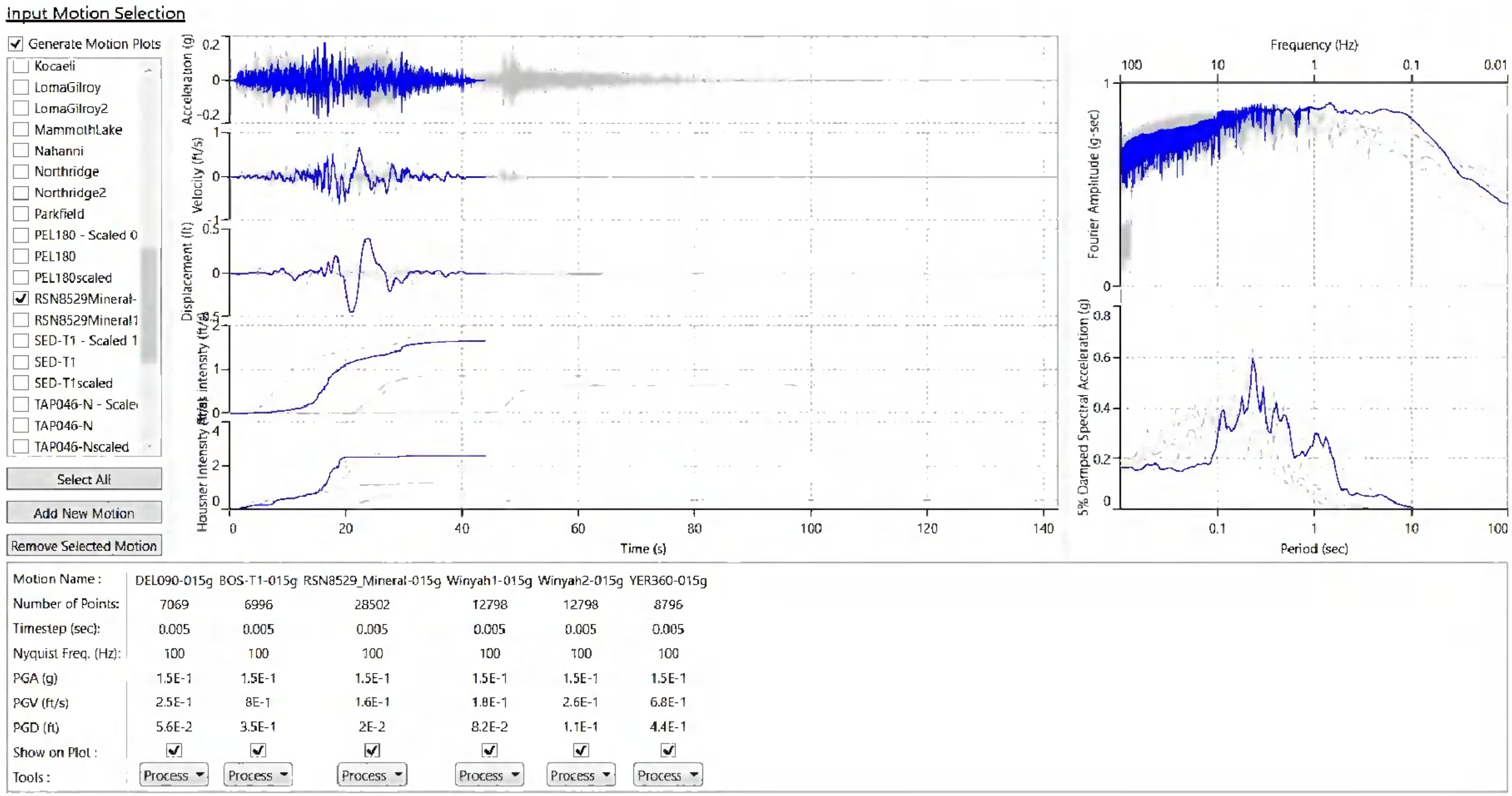
Soil Profile Plot

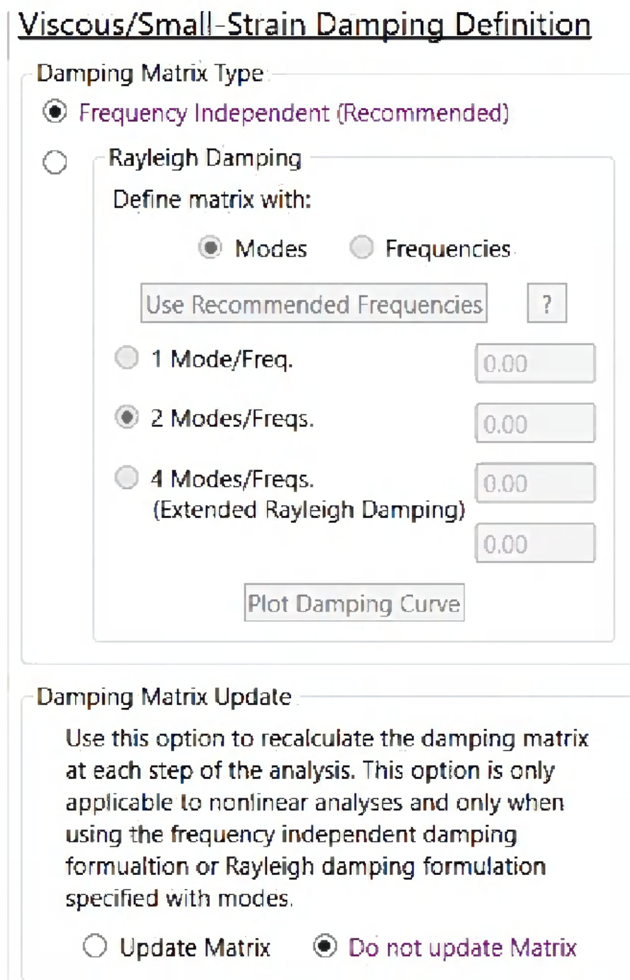


Soil Profile Metrics

Total Profile Depth	503
Profile Natural Frequency (Hz):	0.7939
Profile Natural Period (sec):	1.26


Step 2c - Halfspace and Bedrock Definition
Forward Analysis
Elastic Half-Space
Rigid Half-Space
Bedrock Properties
Firm Rock
Bedrock Name
2296
Shear Velocity (ft/s)
140
Unit Weight (pcf)
1.0
Damping Ratio (%)
?
Save Bedrock
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 Half-Space option should be selected.
If a within motion is being used (e.g. from a vertical array), the Rigid Half-Space option should be selected.
Use Saved Bedrock
Default.bed
Firm Rock.bed
Load
Halfspace Porewater Pressure Dissipation
Use Cv of bottom layer
Specify Halfspace Cv:
0
ft2/s
Deconvolution
Motion recorded at top of layer:
1
Back
Next


Input Motion Selection
Generate Motion Plots
Kocaeli
LomaGilroy
LomaGilroy2
MammothLake
Nahanni
Northridge
Northridge2
Parkfield
PEL180 - Scaled 0
PEL180
PEL180scaled
RSN8529Mineral-
RSN8529Mineral1
SED-T1 - Scaled 1
SED-T1
SED-T1scaled
TAP046-N - Scale
TAP046-N
TAP046-Nscaled
Select All
Add New Motion
Remove Selected Motion
Acceleration (g)
Velocity (ft/s)
Displacement (ft)
Arias Intensity (ft/s)
Housner Intensity (ft)
Time (s)
Frequency (Hz)
Fourier Amplitude (g-sec)
5% Damped Spectral Acceleration (g)
Period (sec)
Motion Name : DEL090-015g BOS-T1-015g RSN8529_Mineral-015g Winyah1-015g Winyah2-015g YER360-015g
Number of Points: 7069 6996 28502 12798 12798 8796
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) 2.5E-1 8E-1 1.6E-1 1.8E-1 2.6E-1 6.8E-1
PGD (ft) 5.6E-2 3.5E-1 2E-2 8.2E-2 1.1E-1 4.4E-1
Show on Plot :
Tools : Process Process Process Process Process Process


Viscous/Small-Strain Damping Definition
Damping Matrix Type
Frequency Independent (Recommended)
Rayleigh Damping
Define matrix with:
Modes
Frequencies
Use Recommended Frequencies
?
1 Mode/Freq.
2 Modes/Freqs.
4 Modes/Freqs.
(Extended Rayleigh Damping)
0.00
0.00
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

Analysis Control Definition

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)

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

( ) Simplified

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

Time Domain

Step Control

(●) Flexible ( ) Fixed

Maximum Strain Increment (%): 0.005

Number of Sub-increments: 1

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	HP 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: SOUTH ASH POND

INTRODUCTION

This liquefaction potential analysis calculation package (Liquefaction Package) was prepared to present the evaluation for liquefaction potential of the perimeter dike soils forming the South Ash Pond and foundation soils beneath the perimeter dike at Winyah Generating Station (WGS or Site). This calculation package is Attachment 4 to *2021 Periodic Safety Factor Assessment: South Ash Pond* (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: South Ash Pond* (Site Response Package) to the 2021 Safety Factor Assessment Report. The liquefaction potential of soils was evaluated using results from soil borings and cone penetration test (CPT) soundings advanced through the South Ash Pond perimeter dikes during Geosyntec's 2013 and 2016 geotechnical subsurface investigations. Details of these investigations are discussed in *2016 Surface Impoundment Periodic Safety Factor Assessment Report: South Ash Pond* (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 analyses 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 at WGS to evaluate the susceptibility of fine-grained soils to cyclic softening. Of the four, fine-grained samples tested within the South Ash Pond area, none were found to be susceptible to cyclic softening under the criteria described in Bray and Sancio (2006).

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 potential of soils to liquefy and the factor of safety against liquefaction are described below.

Cyclic Stress Ratio (CSR)

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

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

where:

$\mathrm{CSR}_{\mathrm{M},\sigma'vo}$ = 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 of the CPT data are reiterated below.

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

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

and,

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

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:

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

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:

$$q_{c1} = C_N q_c \qquad (5)$$

where:

C_N = overburden correction factor = $(P_a/\sigma'_{vo})^{1.338-0.249(q_{c1Ncs})^{0.264}}$; and
q_{c1N} = normalized tip resistance q_{c1}/P_a (dimensionless).

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 implemented 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 used in computing resistance of a soil against triggering of liquefaction, was calculated by the following equation presented by Idriss and Boulanger (2008).

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

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:

$$C_E = \frac{ER}{60} \qquad (7)$$

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 $(N_1)_{60cs}$ values less than 46 blows per foot, as follows:

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

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:

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

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, as follows:

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

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:

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

where:

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

and,

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

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:

$$CRR_{\sigma'_{vo}} = K_\sigma \times CRR_{\sigma'_{vo}=1\,atm} \quad (16)$$

Magnitude Scaling Factor (MSF)

The magnitude scaling factor (MSF) is applied to adjust the CRR to a common earthquake magnitude, M. For cohesionless soils, the MSF is calculated using the equation proposed by Idriss (1999):

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

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:

$$CRR_M = MSF \times CRR_{M=7.5} \quad (18)$$

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 (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 Geotechnical Design Manual (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 Against Triggering of Liquefaction

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

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

where:

$CRR_{M,\sigma'vo,Kdr}$ = cyclic resistance ratio adjusted for earthquake magnitude, effective overburden stress, and deposit age ($CRR_{M=7.5,\sigma'_{vo}=1\,atm} \times K_\sigma \times MSF \times K_{DR}$); 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 from soil borings and CPT soundings performed in 2013 and 2016. 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 foundation soils immediately underlying the perimeter dikes.

Two representative shear wave velocity (V_s) profiles were developed from the dike crest 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 to the underlying Chicora and Williamsburg Formation Clay strata for the site response analyses of the South Ash Pond.

The 2016 Safety Factor Assessment identified zones of subsurface materials expected to undergo triggering of liquefaction along the west end of the South Ash Pond. Therefore, site response analyses were only performed for Profile 1, which contains the identified zone of potential liquefaction, for the 2021 Safety Factor Assessment Report to provide an updated evaluation of liquefaction potential of the subsurface materials in the critical area of the South Ash Pond. Only the investigations along Profile 1 are considered in the liquefaction potential evaluation presented in this calculation package (CPT-122, CPT-129, CPT-205, CPT-206, CPT-208, SPT-109, SPT-302, and SPT-303), as shown on Figure 1.

For Profile 1, 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 Profile 1. The τ_{max} profile was used to compute the CSR at every depth for each soil boring or sounding. The maximum shear stress at each computed depth for Profile 1 is 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 considered for the liquefaction potential analysis.

Total Unit Weight

The total unit weight (γ_T) was used to calculate the total and effective vertical stresses for the soil column for each boring and sounding analyzed. For the purpose 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) and the site-specific laboratory data (Geosyntec, 2016). 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
- 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 of 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 ($N \geq 30$ 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 near 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 South Ash 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 2, soils immediately surrounding the South Ash 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. The bottom of dike fill soils at each investigation point was estimated based on the elevation of the

South Ash Pond toe drains as provided in design drawings (Lockwood-Greene, 1978). The dike fill soils were assumed to extend 1 ft below the toe drain elevation for each investigation point. Information for the investigation points considered in this calculation package are summarized in Table 5. An age correction factor of 1.0 was applied for dike fill soils, as these structures are approximately 30 to 40 years old. As noted previously, "geologic" age differs from "geotechnical" age: geologic age refers to the overall age of the soil since deposition, while 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}.

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 soils is somewhat variable across the South Ash Pond footprint. Physical samples are not collected during CPT soundings, so index test data for each CPT sounding was based on the data collected from the nearest available soil boring with laboratory index testing, as provided in the 2016 Safety Factor Assessment Report. Index testing, when available, for soil borings were utilized for each individual SPT N-value. The source of the select fines content for each investigation point 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 for each individual boring or CPT sounding based on depth to water measurements, porewater pressure (u_o) signatures, and dissipation tests. A review of the topographic survey data from August 2021 (McKim & Creed, 2021) and the topographic survey used in the 2016 Safety Factor Assessment indicated that dewatering lowered the free water level in the east side of the South Ash Pond and ash has been excavated from the east side of the pond (top of ash surface in the west side of the South Ash Pond is similar to that used for the 2016 Safety Factor Assessment). Santee Cooper provided available water level measurements from wells in the South Ash 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 South Ash Pond perimeter dikes, the water level within the perimeter dike may be similar to the water level used for the 2016 Safety Factor Assessment or lower due to dewatering in the east side of the pond. Therefore, as detailed in the Site Response Package, site response analyses were performed with the water table modeled at 10 ft below ground surface, which was used for the 2016 Safety Factor Assessment, and 18 ft below ground surface, which models a lowered water table. Maximum shear stress profiles were calculated for both water table elevations and reported in the Site Response Package.

For the liquefaction potential evaluation presented in this calculation package, the phreatic surface assumptions through the South Ash 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 using the phreatic surface assumed for the 2016 Safety Factor Assessment (Base Water Table) and with the phreatic surface assumed to be 8 ft lower or at the bottom of the dike (whichever is higher) (Lowered Water Table). The elevations of the phreatic surface at the TOB used for the CRR estimation and for the Base Water Table and Lowered Water Table used for the CSR estimation are summarized in Table 5.

Energy Calibration for SPT N-Values

As described in the 2016 Safety Factor Assessment Report (Geosyntec, 2016), the subcontractor during Geosyntec's 2013 subsurface investigation, Soil Consultants, Inc. (SCI), reported that the automatic hammer on the utilized drilling rig had an energy ratio of 89 percent, which was independently evaluated within six months of the investigation. During the 2016 subsurface investigation, Mid-Atlantic Drilling, Inc. (MAD) utilized a drill rig with an energy ratio of 77 percent.

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 the selected soil borings and soundings located at the dike centerline. The factor of safety against liquefaction (FS_{liq}) was computed at every depth interval where data were 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 assuming the phreatic surface representing the Base Water Table for the CSR are shown in Figures 3 through 6. Figure 3 shows CPT-122 and SPT-303 which are located in the southwest corner of the South Ash Pond. Subsequent figures depict calculation results for soil borings and CPT soundings moving in a clockwise direction around the surface impoundment. Figures 7 through 10 show the calculated FS_{liq} for the same soil borings and CPT soundings assuming the phreatic surface representing the Lowered Water Table for the CSR. Example calculations are provided within Appendix 1 of this attachment.

The liquefaction potential calculation results can be generally summarized as follows:

- The computed FS_{liq} typically exceeded 2.0 within dike fill and foundation soils immediately below the South Ash Pond perimeter dikes.

- Dike fill soils adjacent to SPT-302 were computed with FS_{liq} ranging from 1.6 to 1.9 between elevations 20 and 30 ft NGVD29.

- Foundation soils were typically not found to be susceptible to liquefaction beneath the South Ash Pond perimeter dikes. Isolated zones where FS_{liq} between 1.0 and 1.3 were computed for CPT-122, CPT-129, and CPT-205 between elevations of -20.0 and 2.0 ft NGVD29.

- The FS_{liq} computed for the soil borings and CPT soundings were found to be generally consistent between investigation points adjacently located (e.g., SPT-302 and CPT-205).

- Similar observations were made for the phreatic surfaces representing both the Base Water Table and Lowered Water Table. The FS_{liq} computed with the Lowered Water Table were generally larger than the FS_{liq} computed with the Base Water Table.

CONCLUSIONS

Based on the updated liquefaction potential evaluation presented within this calculation package, the calculated FS_{liq} values are greater than 1.0. Therefore, the dike fill and foundation materials underlying the South Ash Pond perimeter dikes are not expected to undergo triggering of liquefaction under the design earthquake.

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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 Envelopes for the South Ash Pond Dike Centerline

Profile 1 – Base WT		Profile 1 – Lowered WT	
Depth (ft)	τ_{max} (psf)	Depth (ft)	τ_{max} (psf)
2.5	29	2.5	47
7.5	67	7.5	104
12.5	90	12.5	135
16.5	103	16.5	156
19.5	114	19.5	179
23.5	149	23.5	199
28.5	178	28.5	210
33.5	202	33.5	219
38.0	269	38.0	278
42.0	307	42.0	318
46.0	334	46.0	344
50.5	380	50.5	374
58.0	480	58.0	473
68.0	597	68.0	591
78.0	708	78.0	710
88.0	816	88.0	824
98.0	929	98.0	934
108.0	1060	108.0	1064

Notes:
1. Calculations are presented in the Site Response Package provided as Attachment 3.
2. For calculation points located in between the depth intervals listed above, the τ_{max} was linearly interpolated.
3. WT = water table.

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 Geotechnical Design Manual (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	Dike Base Elevation	GWT (ft NGVD29)[5]			GWT Source	FC Basis	τ_{max} Profile
-	ft	ft	ft NGVD29	ft NGVD29	TOB	Base	Lowered	-	-	-
CPT-122	546598.0511	2499294.0984	38.82	17.6	30.4	30.4	22.4	Diss. Test	SPT-303/303A	Profile 1
CPT-129	547429.4842	2498965.8353	38.88	20.0	25.8	25.8	20.0	Diss. Test	SPT-302	Profile 1
CPT-205	547422.5833	2499106.9970	38.88	20.3	29.2	29.2	21.2	Diss. Test	SPT-302	Profile 1
CPT-206	547384.8761	2498800.1583	38.88	19.7	29.2	29.2	21.2	u_2 Signature	SPT-302	Profile 1
CPT-208	547121.6171	2498742.0069	37.39	17.0	27.4	27.4	19.4	u_2 Signature	SPT-109	Profile 1
SPT-109	546898.7338	2498876.8972	37.39	16.5	32.3	32.3	24.3	Borehole	SPT-109	Profile 1
SPT-302	547422.0662	2498943.1317	38.88	20.0	29.2	29.2	21.2	Borehole	SPT-302	Profile 1
SPT-303	546607.9500	2499254.4342	38.82	17.6	33.4	33.4	25.4	Borehole	SPT-303	Profile 1

Notes:

1. ft NGVD29 - feet National Geodetic Vertical Datum of 1929; TOB - Time of Boring; GWT - Groundwater Table; FC - Fines Content; N/A = Not Applicable.
2. Dike bottom elevation was estimated based on the design elevation of the nearest toe drain (Lockwood-Greene, 1978) minus 1 ft.
3. FC Basis refers to the source of the fines content profile for each investigation point. Fines content data are provided within the 2016 Safety Factor Assessment (Geosyntec, 2016).
4. The GWT elevation for SPT-302, which was advanced using mud rotary wash techniques, was selected as 9.64 ft bgs (based on CPT-205) as the bentonite slurry prevented the water within the borehole from reaching an equilibrium condition within 24 hours.
5. Cyclic Resistance Ratios (CRR) were calculated using the water table elevation from TOB. Different water table elevations (Base and Lowered) were considered for calculations of the Cyclic Stress Ratio (CSR).

FIGURES

LEGEND

W-SW-SAP	EXISTING STAFF GAUGE		POND BOUNDARY
CPT-52	GEOSYNTEC CONE PENETRATION TEST		PROPERTY BOUNDARY
PPZ-5	PIEZOMETER		EXISTING GAS LINE
SPT-111	GEOSYNTEC SOIL BORING		EXISTING RAILROAD
SC-60	HISTORICAL BORING		
WAP-9	GROUNDWATER MONITORING WELL		

600 300 0 600 Feet

NOTES:
1. Aerial imagery was obtaied from ESRI online database.
2. The position of underground utilities shown should be considered approximate.
3. Historical boring SC-77 is not shown on this map as its location is not known. It is assumed to be a South Ash Pond boring based on its numerical sequence related to other borings.

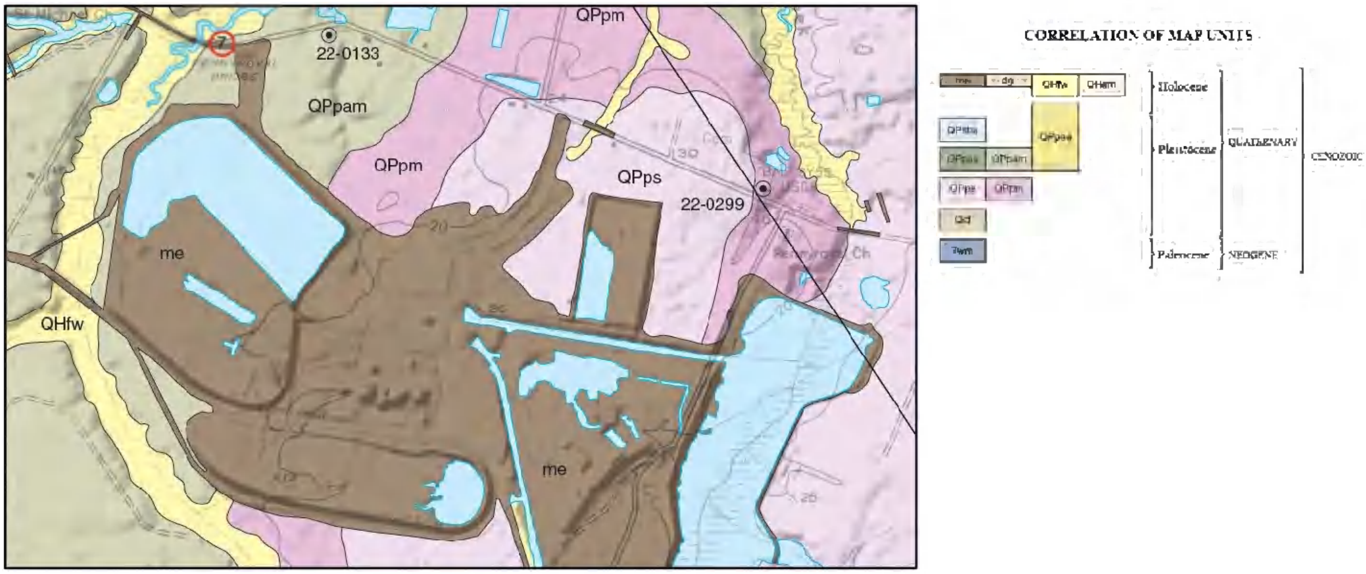
SOUTH ASH POND
WINYAH GENERATING STATION
GEORGETOWN, SOUTH CAROLINA

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_South Ash Pond_No X-Sections.mxd; Abby Wesley; 10/12/2021



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 2. Geologic Map of Areas Surrounding the South Ash Pond
(Map taken from SC Department of Natural Resources: Geological Survey, 2012)

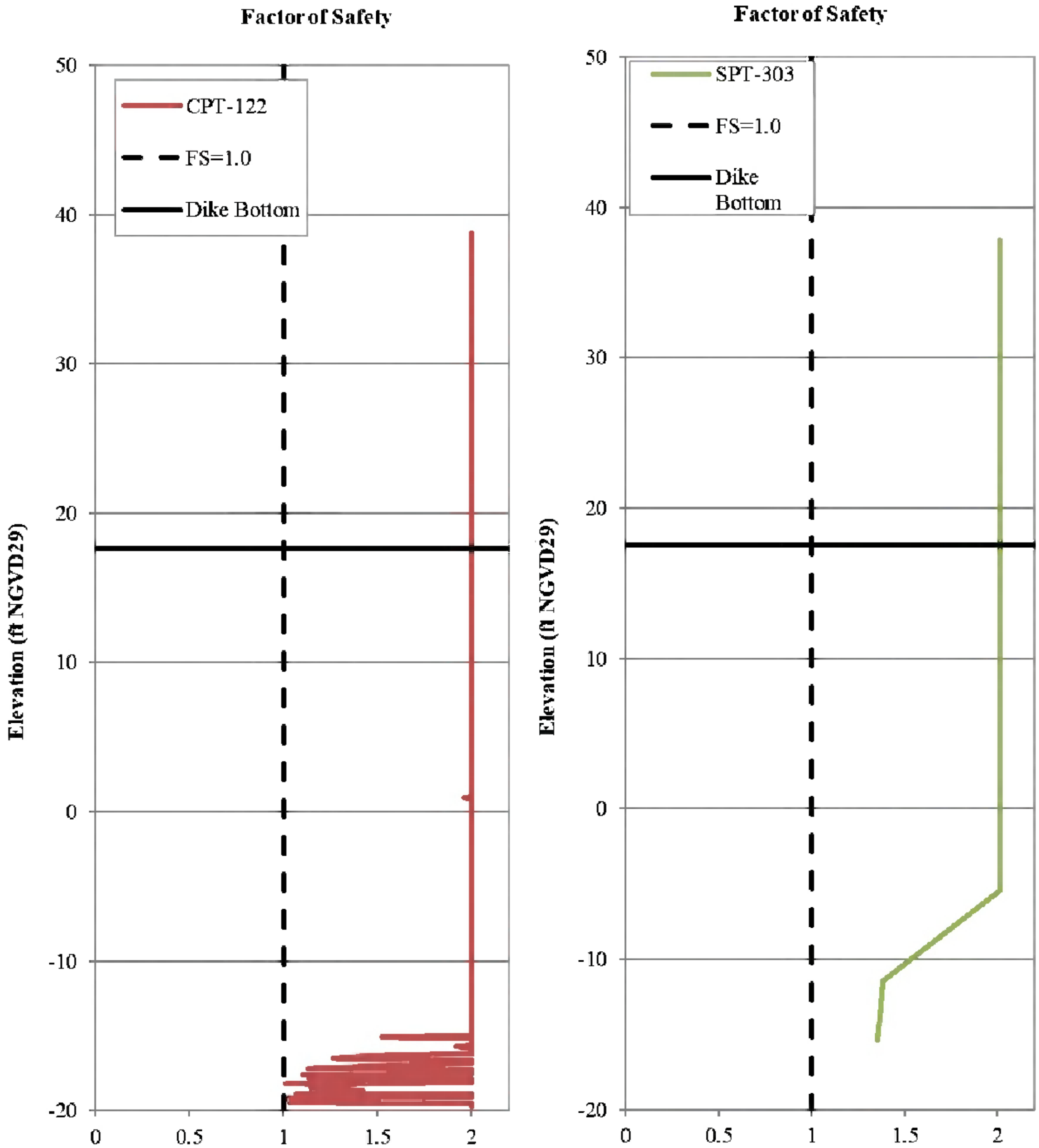


Figure 3. Liquefaction Results for Dike and Foundation Soils for CPT-122 and SPT-303 for Base Water Table

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.

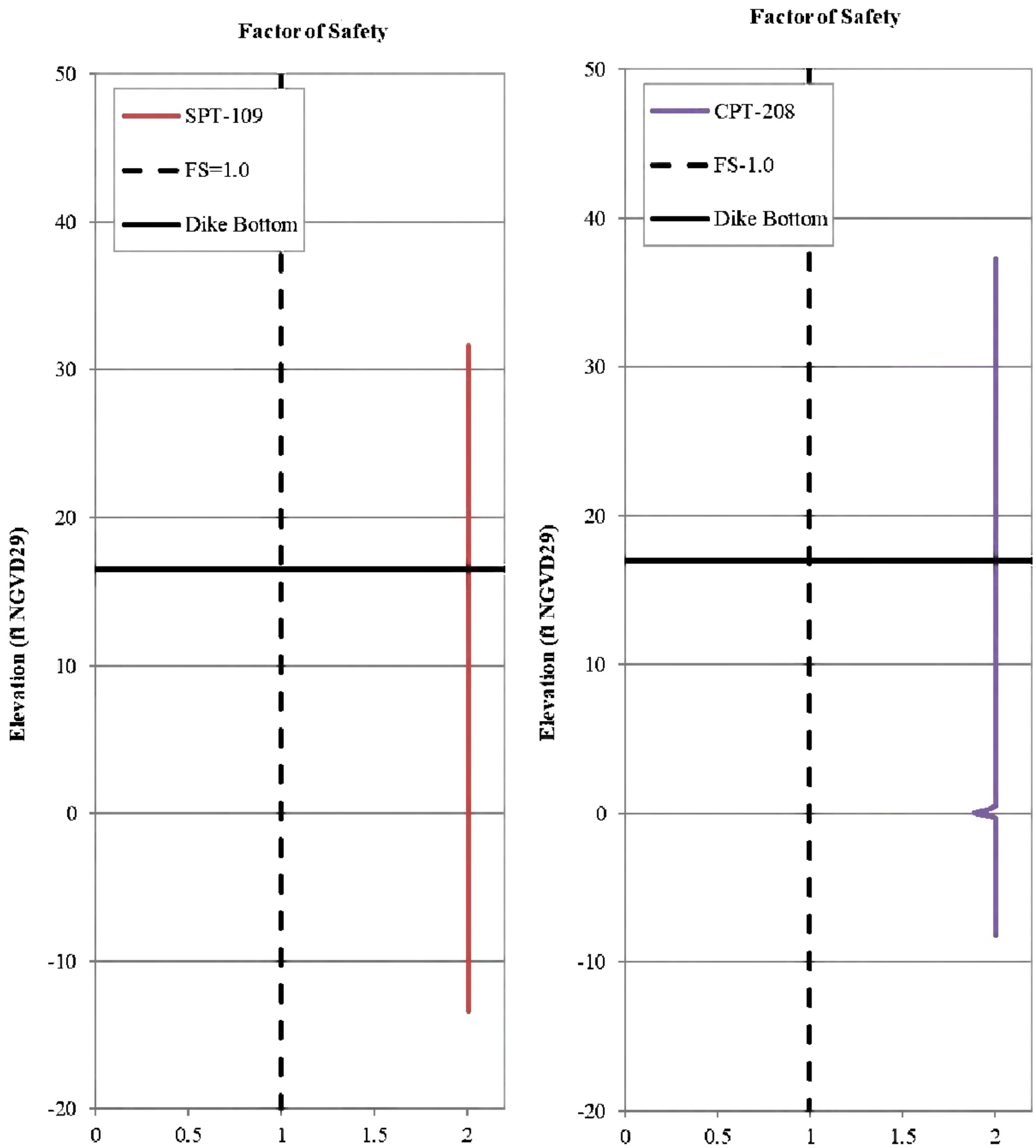
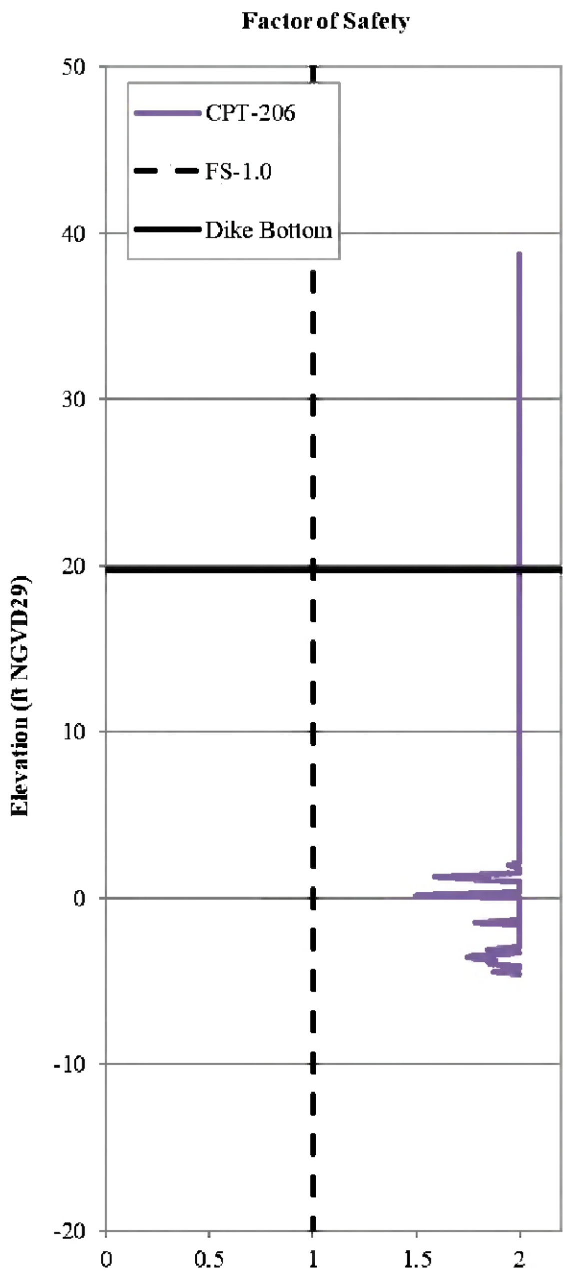


Figure 4. Liquefaction Results for Dike and Foundation Soils for SPT-109 and CPT-208 for Base Water Table

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.



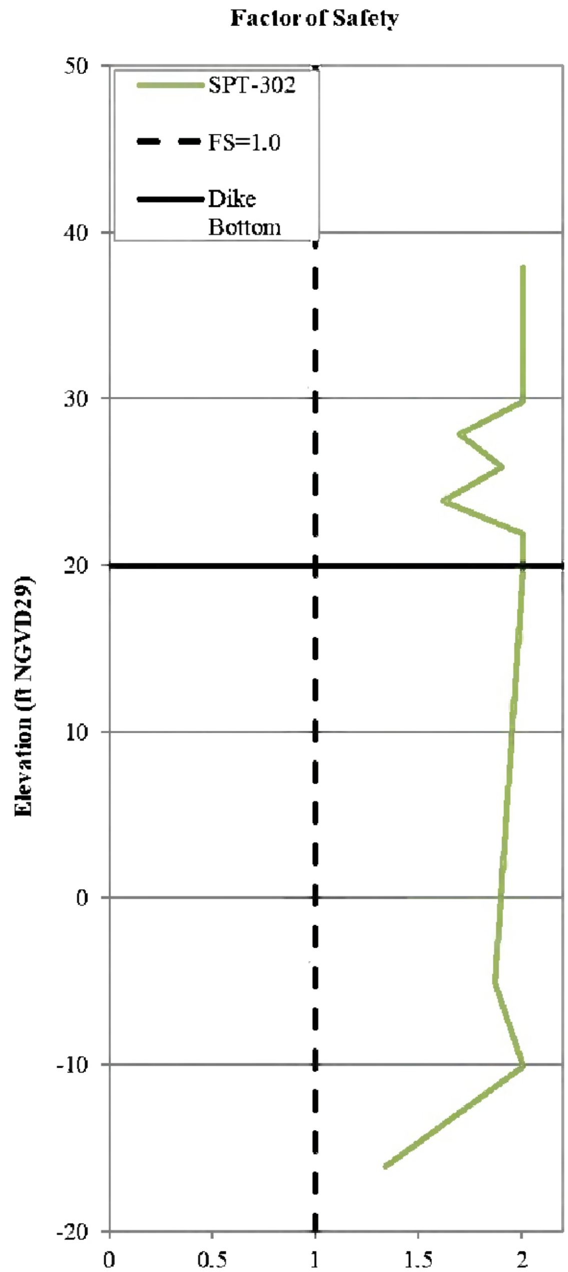


Figure 5. Liquefaction Results for Dike and Foundation Soils for CPT-206 and SPT-302 for Base Water Table

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.

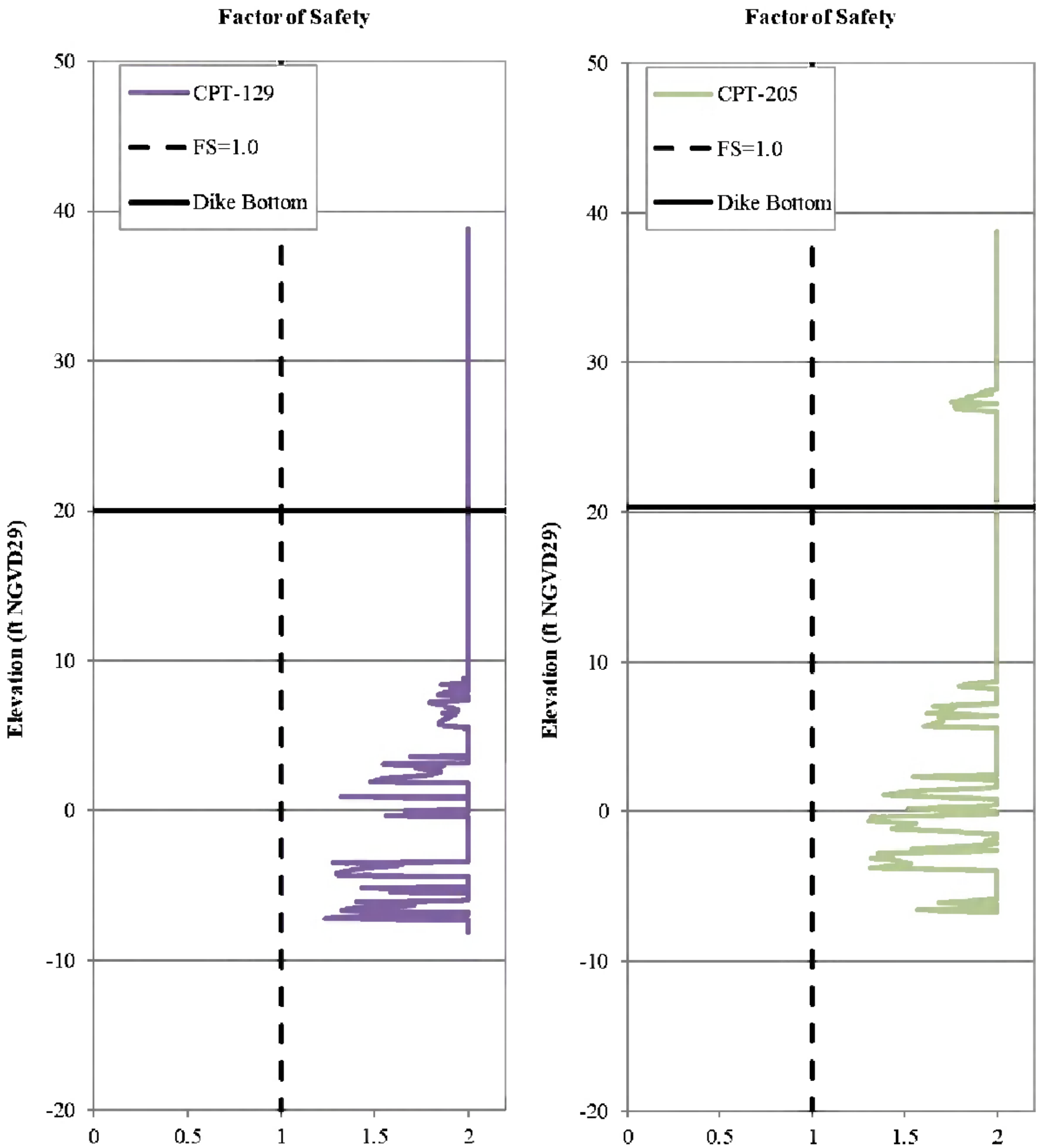


Figure 6. Liquefaction Results for Dike and Foundation Soils for CPT-129 and CPT-205 for Base Water Table

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.

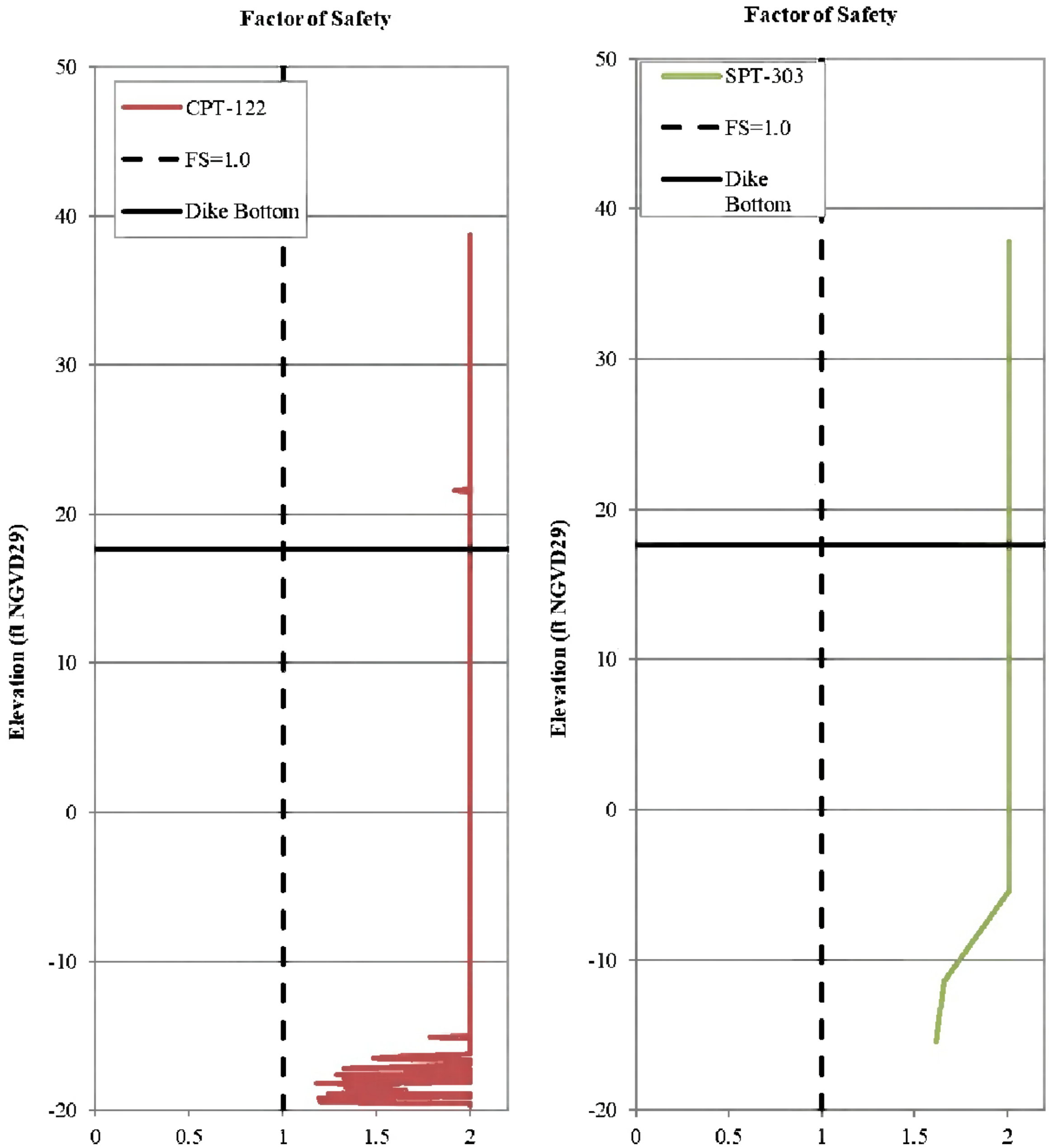


Figure 7. Liquefaction Results for Dike and Foundation Soils for CPT-122 and SPT-303 for Lowered Water Table

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.

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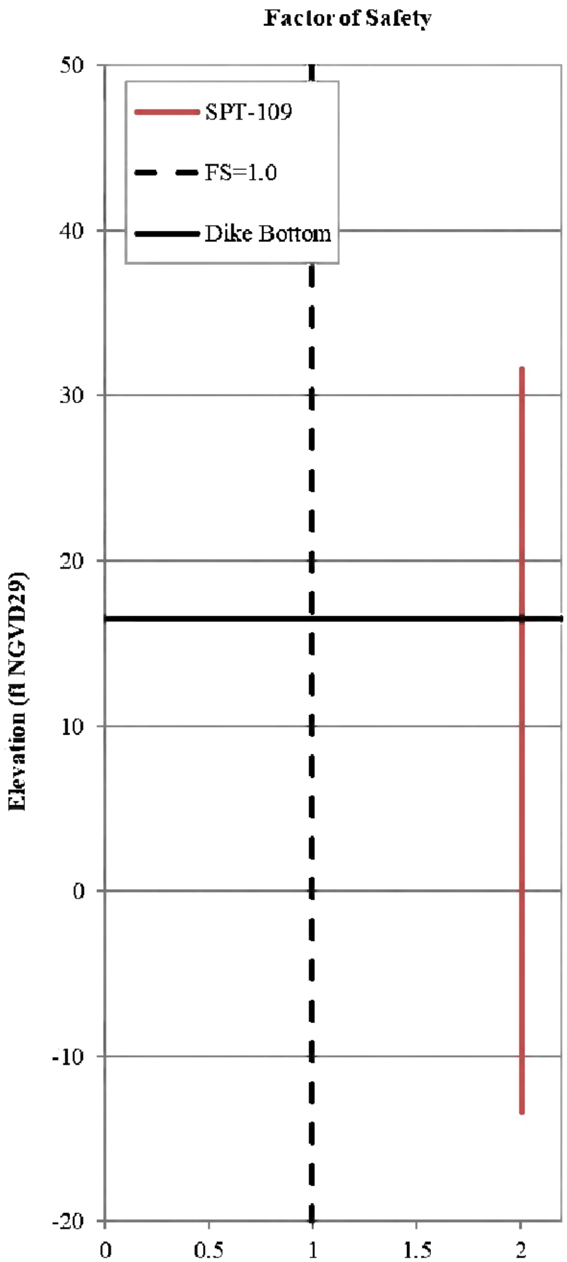

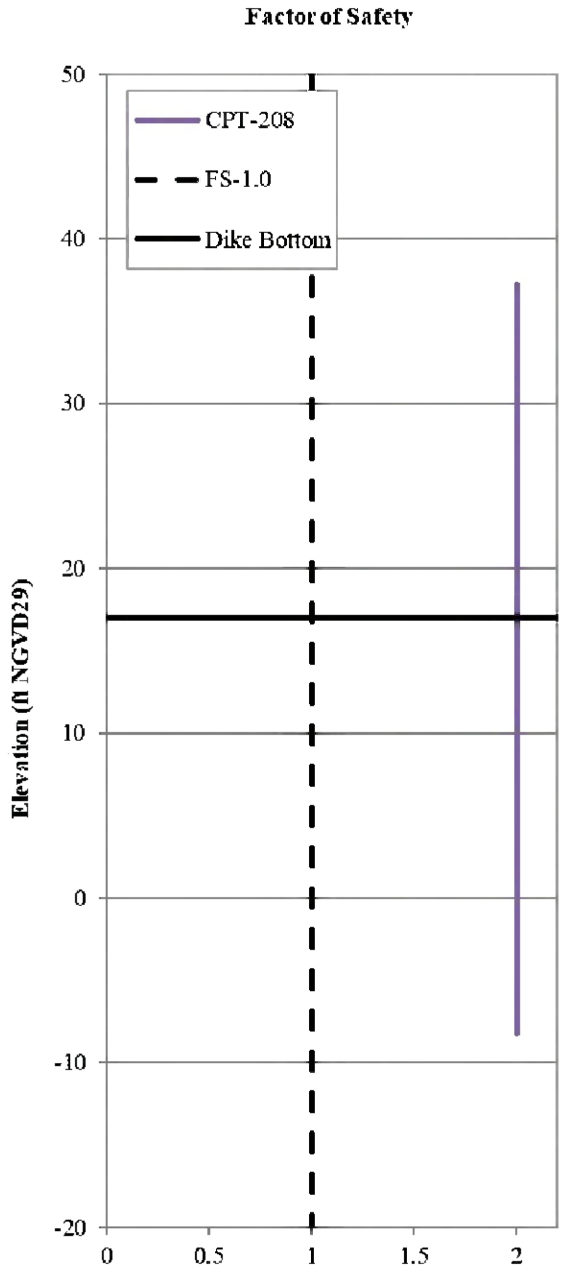

Figure 8. Liquefaction Results for Dike and Foundation Soils for SPT-109 and CPT-208 for Lowered Water Table

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.

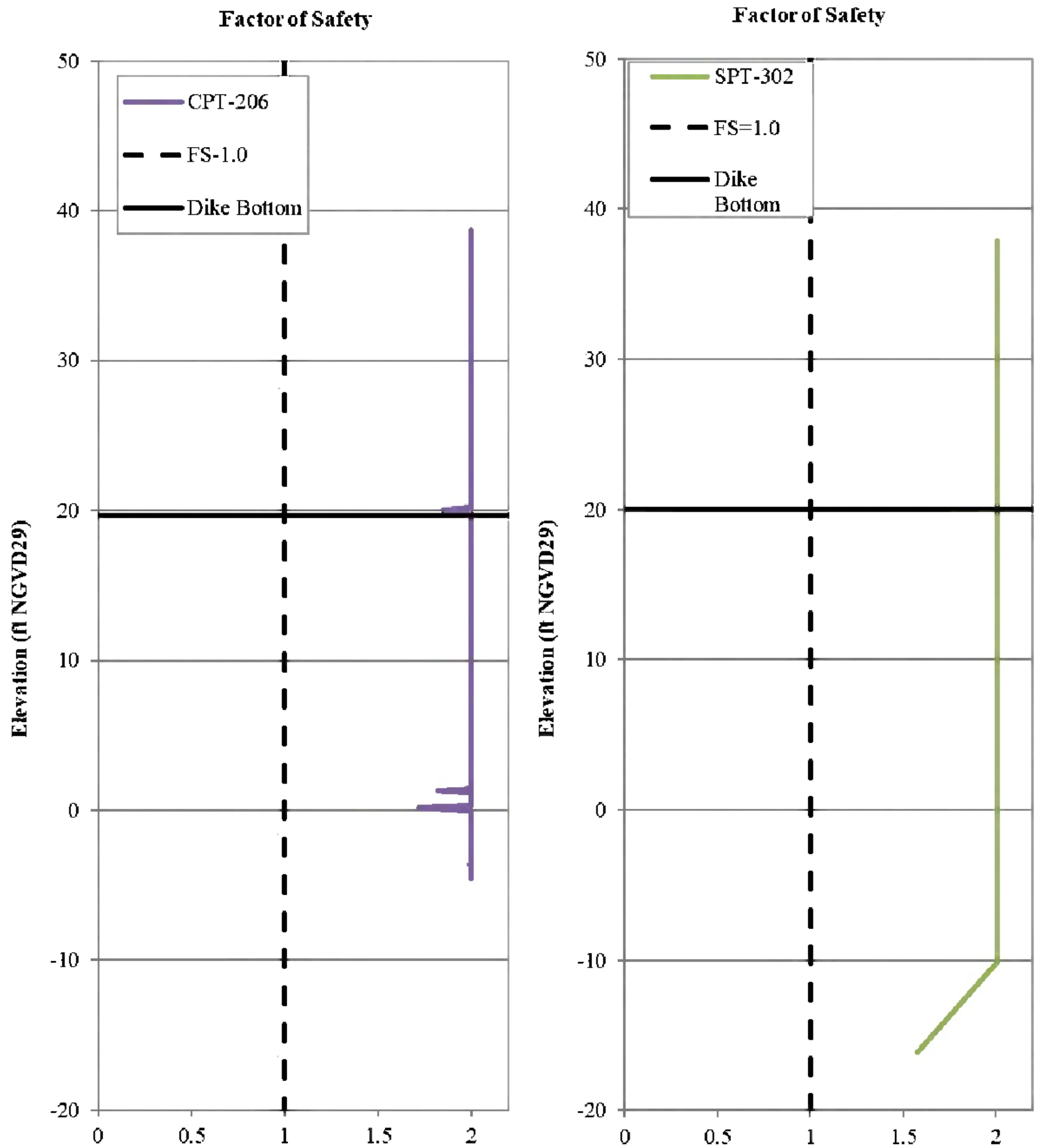

Figure 9. Liquefaction Results for Dike and Foundation Soils for CPT-206 and SPT-302 for Lowered Water Table

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.

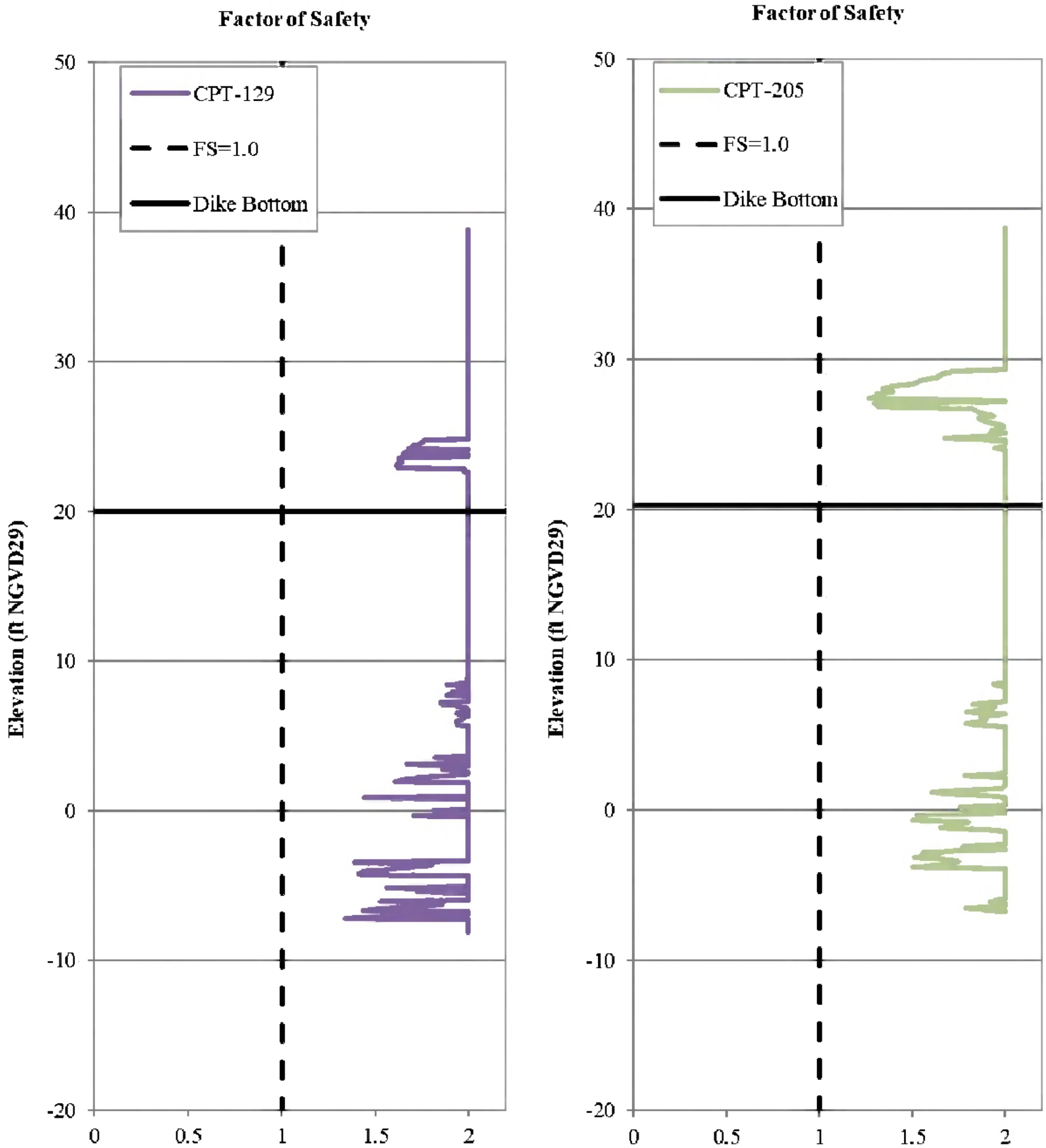

Figure 10. Liquefaction Results for Dike and Foundation Soils for CPT-129 and CPT-205 for Lowered Water Table

Note:

1. Foundation soils were assumed to begin at the dike bottom, which was selected based on the toe drain elevation less 1-ft (Lockwood-Greene, 1978), as provided in Table 5.

Appendix 1

MathCAD® Example Calculations

SPT - Based Liquefaction Analysis

$BoringID := \text{"SPT\_109"}$ $\qquad tsf := \frac{tonf}{ft^2}$ $\qquad kPa := \frac{1}{95.760518} \cdot tsf$

Site Parameters:

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

Earthquake Magnitude: $M := 7.3$

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

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

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: $Elevation := 37.39 \cdot ft$ NGVD29

Groundwater Depth at Time of Boring (TOB): $GWT_b := 5.1 \cdot ft$ bgs

Groundwater Depth at Time of Analysis (TOA): $GWT := 5.1 \cdot ft$ bgs

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

Bottom of Holocene Elevation / Bottom of Dike Fill Soils: $Elev_h := 16.50 \cdot ft$ NGVD29

Energy Calibration: $ER := 88$ % (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:

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

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

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

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:

Adjust according to specific site conditions

1. Coal Combustion Residuals $\gamma_1 := 100 \cdot pcf$
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

$Elev := (Elevation - depth)$ $WMElev := -8 \cdot ft$ (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)

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

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

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]:

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

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

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]:

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

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

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

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

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

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:

$$FS_i := \left\| \begin{array}{l} \text{if } Class_i = \text{“CHICORA”} \\ \quad \| \, 2.01 \\ \text{also if } Class_i = \text{“CLAY”} \\ \quad \| \, 2.01 \\ \text{also if } depth_i < GWT \\ \quad \| \, 2.01 \\ \text{else} \\ \quad \left\| \, min\left(\dfrac{CRR_{final_i}}{CSR_i}, 2.01\right) \right. \end{array} \right.$$

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

$BoringID := \text{"CPT\_122"}$ $\qquad$ $tsf := 2116 \cdot psf$

Site Parameters:

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

Earthquake Magnitude: $M := 7.3$

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

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

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: $Elevation := 38.82 \cdot ft$ NGVD 29

Groundwater Depth at Time of Boring (TOB): $GWT_b := 8.46 \cdot ft$ bgs

Groundwater Depth at Time of Analysis (TOA): $GWT := 8.46 \cdot ft$ bgs

Bottom of Holocene Elevation / Bottom of Dike Fill Soils: $Elev_h := 17.6 \cdot ft$ 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:

$$class_{1983} := \begin{array}{l} \text{for } i \in 0..\text{rows}(qt)-1 \\ \quad class_i \leftarrow 5 \\ \quad \text{if } \dfrac{qt_i}{100 \cdot kPa} \geq s04\left(Rf_i\right) \\ \quad\quad class_i \leftarrow 4 \\ \quad \text{if } \dfrac{qt_i}{100 \cdot kPa} \geq s03\left(Rf_i\right) \\ \quad\quad class_i \leftarrow 3 \\ \quad \text{if } \dfrac{qt_i}{100 \cdot kPa} \geq s02\left(Rf_i\right) \\ \quad\quad class_i \leftarrow 2 \\ \quad \text{if } \dfrac{qt_i}{100 \cdot kPa} \geq s01\left(Rf_i\right) \\ \quad\quad class_i \leftarrow 1 \\ class \end{array}$$

$$\gamma 1 := \begin{array}{l} \text{for } i \in 0..\text{rows}(qt)-1 \\ \quad \text{for } m \in 1..5 \\ \quad\quad \text{if } class_{1983_i} = m \\ \quad\quad\quad \gamma 1_i \leftarrow \gamma_m \\ \gamma 1 \end{array}$$

Refined Soil Classification Using Robertson and Cabal 2010:

Static Pore Pressures at time of Sounding: $$u_{0b_i} := \begin{array}{l} \text{if } depth_i > GWT_b \\ \quad \left(depth_i - GWT_b\right) \cdot \gamma_{water} \\ \text{else} \\ \quad 0 \end{array}$$

Total and Effective Overburden Pressure:

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

$$\sigma_{v0b_i} := \begin{array}{l} \text{if } i > 0 \\ \quad \left(depth_i - depth_{i-1}\right) \cdot \left(\dfrac{\gamma 1_i + \gamma 1_{i-1}}{2}\right) + \sigma_{v0b_{i-1}} \\ \text{else} \\ \quad depth_i \cdot \gamma 1_0 \end{array}$$

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

Normalized Parameters:

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

Unit Weight Values to be Assigned to Robertson (1990) Classification:

Adjust according to specific site conditions

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:

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

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

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:

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

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

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: $q_{c1Ncs_i} := q_{c1N_i} + \Delta q_{c1N_i}$

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

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

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

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

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

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

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

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

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)

$$CRR_{final_i} := \left\| \begin{array}{l} \text{if } I_{c_i} \le 2.60 \wedge Elev_i < Elev_h \\ \quad \left\| CRR2_i \cdot K_{dr} \right. \\ \text{else} \\ \quad \left\| CRR2_i \right. \end{array} \right.$$

Compute the CSR for the Soil Profile:

Final Static Pore Pressure Calculation at Time of Analysis:

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

Final Total and Effective Overburden Pressure at Time of Analysis:

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

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

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

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

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

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

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

Compute Factor of Safety:

$$FS_i := \begin{cases} 2.00 & \text{if } depth_i < GWT_b \\ min\left(\dfrac{CRR_{final_i}}{CSR_i}, 2.00\right) & \text{else} \end{cases}$$

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**	Date:	**10/14/2021**	Reviewed by:	**C. Carlson/B. Gin**	Date:	**10/14/2021**
Client:	**Santee Cooper**	Project:	**Winyah Generating Station**	Project/ Proposal No.:	**GC8100**	Task No.:	**03**

SAFETY FACTOR ASSESSMENT: SOUTH ASH POND

INTRODUCTION

This calculation package was prepared as Attachment 5 to the *2021 Periodic Safety Factor Assessment: South Ash Pond* (2021 Safety Factor Assessment Report) and presents the slope stability analyses for the critical portion of the South Ash Pond perimeter dikes at Winyah Generating Station (WGS), Georgetown County, South Carolina. On 17 April 2015, the United States Environmental Protection Agency (USEPA) published 40 Code of Federal Regulations [CFR] Parts 257 and 261 (CCR Rule). Under the CCR Rule, the South Ash 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 South Ash 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: South Ash Pond* (Liquefaction Package) of the 2021 Safety Factor Assessment Report did not indicate that the South Ash 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 South Ash 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 and the non-rotational mode 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: South Ash Pond* (Site Response Package) of 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 sections evaluated as part of this safety factor assessment.

External Geometry

The South Ash Pond perimeter dikes are approximately 24 feet (ft) in height, with a crest elevation of approximately 38.0 ft National Geodetic Vertical Datum of 1929 (NGVD29) and toe elevation of approximately 24.0 ft NGVD29. The upstream and downstream side slopes range from 3 horizontal to 1 vertical (3H:1V) in the east to 4H:1V in the west; the dike crest is typically 12 to 15 ft wide (Thomas and Hutton, 2012). To the north, east, and south of the perimeter dikes, a shallow drainage swale has been excavated inside the railroad loop and drains to the sump located to the west of the South Ash Pond.

Five cross sections were developed and evaluated as part of the 2016 Safety Factor Assessment Report (Geosyntec, 2016). These cross sections were selected based on the critical slope geometry, engineering parameters of subsurface materials, and phreatic conditions. The external geometry of each cross section was based on a topographic survey prepared by Thomas and Hutton (2012). The locations of the five cross sections analyzed in the 2016 Safety Factor Assessment are shown in Figure 1.

For the 2021 Safety Factor Assessment Report, only the critical cross sections identified in the 2016 Safety Factor Assessment were analyzed. In the 2016 Safety Factor Assessment for the South Ash Pond, Cross Section A had the lowest calculated FS for the static slope stability analyses (both maximum normal storage pool and maximum surcharge pool loading conditions) and Cross Section B had the lowest calculated FS for the seismic slope stability analyses. Therefore, updated slope stability analyses were performed for these two cross sections as part of the 2021 Safety Factor

Assessment Report. Updated topographic survey data from August 2021 were also incorporated into these two 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 South Ash 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. Cross Section A also includes riprap buttress material placed against the outer dike slope and across the adjacent shallow drainage swale. 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 CCR 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 South Ash Pond was maintained at an elevation of 28.7 ft NGVD29 by a concrete riser structure with a top stop log (Thomas and Hutton, 2016). An operating level of 28.7 ft NGVD29 in the South Ash Pond was used as the "Maximum Normal Storage Pool" for the South Ash Pond in the 2016 static and seismic slope stability analyses. A review of the topographic survey data from August 2021 and the topographic survey used in the 2016 Safety Factor Assessment indicated that dewatering lowered the free water level in the east side of the South Ash Pond and ash has been excavated from the east side of the pond (top of ash surface in the west side of the South Ash Pond is similar to the observed surface in 2016). Santee Cooper provided water level measurements from wells located outside the downstream toe of the South Ash 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 South Ash Pond perimeter dikes, the water level within the perimeter dike may be similar to the water level used for the 2016 Safety Factor Assessment or lower due to dewatering in the east side of the pond. To model a potentially lowered water table, Cross Section A and Cross Section B were also modeled with the water table approximately at the bottom of the perimeter dike (20.0 to 21.0 ft NGVD29). A higher water table is generally more conservative for static slope stability analyses (i.e., results in a lower calculated FS). However, for seismic slope stability analyses, a lower water table may result in a larger seismic horizontal seismic coefficient and a lower calculated FS. Therefore, for the 2021 Safety Factor Assessment Report, static slope stability analyses were performed using a water table elevation of 28.7 ft NGVD29 (Base Water Table) in the retained ash while seismic slope stability analyses were performed using water table elevations of both 28.7 ft NGVD29 (Base Water Table) and 20.0 to 21.0 ft NGVD29 (Lowered Water Table) in the retained ash and the corresponding seismic horizontal seismic coefficients. 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 South Ash Pond was classified as a "Low Hazard Potential" surface impoundment (Geosyntec, 2021), the 100-year rainfall event with a rainfall duration of 72 hours was selected as the Inflow Design Flood (IDF), as required by §257.73(d)(1)(v)(B). The "maximum surcharge pool" elevation within the South Ash Pond was established based on the maximum surface water elevation within the South Ash Pond computed from the hydrologic and hydraulic (H&H) analysis with the IDF and selected as a more conservative water level (30.7 ft NGVD29) than the maximum surface water level (28.1 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: South Ash 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 A and B for the Base Water Table are provided in Figures 2 and 3, respectively. Figures 4 and 5 respectively show the geometric models for Cross Sections A and B for the Lowered Water Table.

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 South Ash Pond. Table 1 provides a summary of the material properties selected for each evaluated cross section as part of the 2021 Safety Factor Assessment. The interpretation and selection of properties for Cross Sections A and B are shown in Figures 6 and 7, respectively.

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.) for each section.

It was assumed that seismic waves generated from the design seismic earthquake may load dike fill and foundation soils rapidly enough to develop elevated pore pressures and induce an undrained loading condition. In accordance with recommendations made by Hynes-Griffin and Franklin (1984), the selected undrained shear strength values for the clayey soils 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

The seismic hazard evaluation for WGS and the site response analysis for the South Ash Pond perimeter dikes are presented in the Site Response Package of the 2021 Safety Factor Assessment Report. Within that package, maximum shear stress profiles for the six ground motions were computed for the critical soil column of the South Ash Pond. The maximum shear stress profiles were used to compute the MHEA profiles in general accordance with Bray et al. (1995). Preliminary seismic slope stability analyses of the South Ash Pond perimeter dikes for the Base Water Table elevations indicated that a typical depth of the critical slip surface is located less than 30 ft below the dike crest. The MHEA at the anticipated critical slip surface was selected assuming the critical slip surface is located at 10 and 27 ft below the dike crest for Cross Sections A and B, respectively. These critical slip surface depths were not significantly different for the Lowered Water Table elevations. 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 both water table elevations to an approximate depth of 100 ft below ground surface (bgs) are provided in Table 2. MHEA values of 0.064g and 0.052g were selected for Cross Sections A and B for the Base Water Table elevation, respectively, and MHEA values of 0.098g and 0.064g were selected for Cross Sections A and B for the Lowered Water Table elevation, 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 South Ash Pond perimeter dike structures. This is a conservative allowable deformation typically used for seismic analyses of large waste disposal structures (e.g., landfills) (Kavazanjian, 1999). Using the Hynes-Griffin and Franklin (1984) chart and assuming the "Upper Bound" displacement, the ratio of N/A (or k_h/MHEA) was conservatively selected as 0.5, as shown in Figure 8. Thus, k_h values of 0.032 and 0.026 were computed for Cross Sections A and B, respectively, for the Base Water Table elevation, and k_h values of 0.049 and 0.032 were computed for Cross Sections A and B, respectively, for the Lowered Water Table elevation.

RESULTS

The safety factor evaluation for Cross Sections A and B was performed according to the methodology and parameters discussed above, and the results are summarized within Table 3. Computed FS were found to exceed the minimum safety factors required by §257.73(e)(1) of the CCR Rule. The critical cross sections (i.e., the sections with the lowest computed factors) were found to be Cross Section A for static slope stability with maximum normal storage pool and maximum surcharge loading conditions, and Cross Section B for seismic slope stability for the Base Water Table and Lowered Water Table elevations. Figures 9 through 16 depict the calculated safety factors for Cross Sections A and B. While both non-circular and block-type slip surfaces were considered in the analyses, non-circular slip surfaces were consistently more critical for the failure modes of concern and are the critical slip surfaces as presented in Figures 9 through 16.

CONCLUSIONS

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

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TABLES

Table 1. Selected Material Parameters for Analysis

Material	Total Unit Weight (pcf)	Drained Parameters		Undrained Parameters[1]	
		ϕ' (°)	c' (psf)	S_u/σ'_{vo}	$S_{u,min}$ (psf)
Dike Fill	120[2]	27 to 36[3]	0	-	-
Foundation Soils (Clayey)	94[2]	15	300	Varies[4]	300
Foundation Soils (Clayey Sands)	123[2]	32[3]	0	-	-
Chicora	130[2]	50[2]	0	-	-
Williamsburg Formation Clay	105[2]	50[2]	0	-	-
Fly Ash	100[2]	34[2]	0	-	-
Riprap Buttress	150	45	0	-	-

Notes:

1. Undrained strength parameters for clayey foundation soils were applied for the seismic slope stability case only.
2. The selection of shear strength parameters for Chicora, Williamsburg Formation Clay, and Fly Ash, as well as total unit weights for all materials, is explained in the 2016 Safety Factor Assessment Report (Geosyntec, 2016).
3. These drained shear strengths (ϕ') vary by location. Interpretation of in-situ results applied in the selection is provided in Figures 6 and 7.
4. The selected undrained strength ratio (S_u/σ'_{vo}) varies between locations and ranges from 0.25 to 0.70 for the selected cross sections. Interpretation of in-situ results applied in the selection is provided in Figures 6 and 7. A more detailed explanation of the undrained strength ratio for clayey foundation soils is provided in the 2016 Safety Factor Assessment Report (Geosyntec, 2016).

Table 2. Maximum Equivalent Horizontal Acceleration (MHEA) from Site Response Analysis for the South Ash Pond Perimeter Dikes and Varying Water Table Elevation

Profile 1 – Base Water Table		Profile 1 – Lowered Water Table	
Depth (ft)	MHEA	Depth (ft)	MHEA
2.5	0.094	2.5	0.149
7.5	0.072	7.5	0.111
12.5	0.057	12.5	0.086
16.5	0.050	16.5	0.076
18.0	0.048	18.0	0.077
19.5	0.047	19.5	0.074
23.5	0.052	23.5	0.069
28.5	0.052	28.5	0.061
33.5	0.050	33.5	0.054
36.0	0.060	36.0	0.063
38.0	0.059	38.0	0.062
42.0	0.062	42.0	0.065
46.0	0.063	46.0	0.065
48.0	0.065	48.0	0.065
50.5	0.065	50.5	0.064
58.0	0.072	58.0	0.071
68.0	0.077	68.0	0.076
78.0	0.080	78.0	0.081
88.0	0.083	88.0	0.084
98.0	0.085	98.0	0.086
108.0	0.088	108.0	0.088

Notes:

1. Cross Sections A and B (similar in subsurface stratigraphy and location to Profile 1) were found to have depths to the critical slip surface of approximately 10 ft and 27 ft, respectively.
2. For the seismic slope stability with the Base Water Table, MHEA values of 0.064g and 0.052g were selected for Cross Section A and Cross Section B, respectively.
3. For the seismic slope stability with the Lowered Water Table, MHEA values of 0.098g and 0.064g were selected for Cross Section A and Cross Section B, respectively.

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

Safety Factor Case	Target FS	Cross Section A	Cross Section B
Static - Maximum Normal Storage Pool (Base Water Table)	1.50	*1.69*	1.79
Static FS- Maximum Surcharge Pool (Base Water Table)	1.40	*1.67*	1.73
Seismic - Maximum Normal Storage Pool (Base Water Table)	1.00	1.24	*1.09*
Seismic – Lowered Water Table	1.00	1.19	*1.12*
Liquefaction Slope Stability[1]	1.20	N/A	N/A

Notes:

1. The liquefaction safety factors for Cross Sections A and B were not evaluated as embankment soils are not expected to undergo triggering of liquefaction (Liquefaction Package).
2. The lowest computed FS for each analysis case is *italicized*. Cross Sections A and B were observed to have the lowest computed FS and are shown in Figures 9 through 16.
3. Only critical failure surfaces passing through the perimeter dikes were considered.

FIGURES

LEGEND

W-SW-SAP	EXISTING STAFF GAUGE		POND BOUNDARY
CPT-52	GEOSYNTEC CONE PENETRATION TEST		PROPERTY BOUNDARY
PPZ-5	PIEZOMETER		EXISTING GAS LINE
SPT-111	GEOSYNTEC SOIL BORING		EXISTING RAILROAD
SC-60	HISTORICAL BORING		
WAP-9	GROUNDWATER MONITORING WELL		

600 300 0 600 Feet

NOTES:
1. Aerial imagery was obtaied from ESRI online database.
2. The position of underground utilities shown should be considered approximate.
3. Historical boring SC-77 is not shown on this map as its location is not known. It is assumed to be a South Ash Pond boring based on its numerical sequence related to other borings.

SOUTH ASH 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_South Ash Pond.mxd, Abby.Wesley, 10/12/2021

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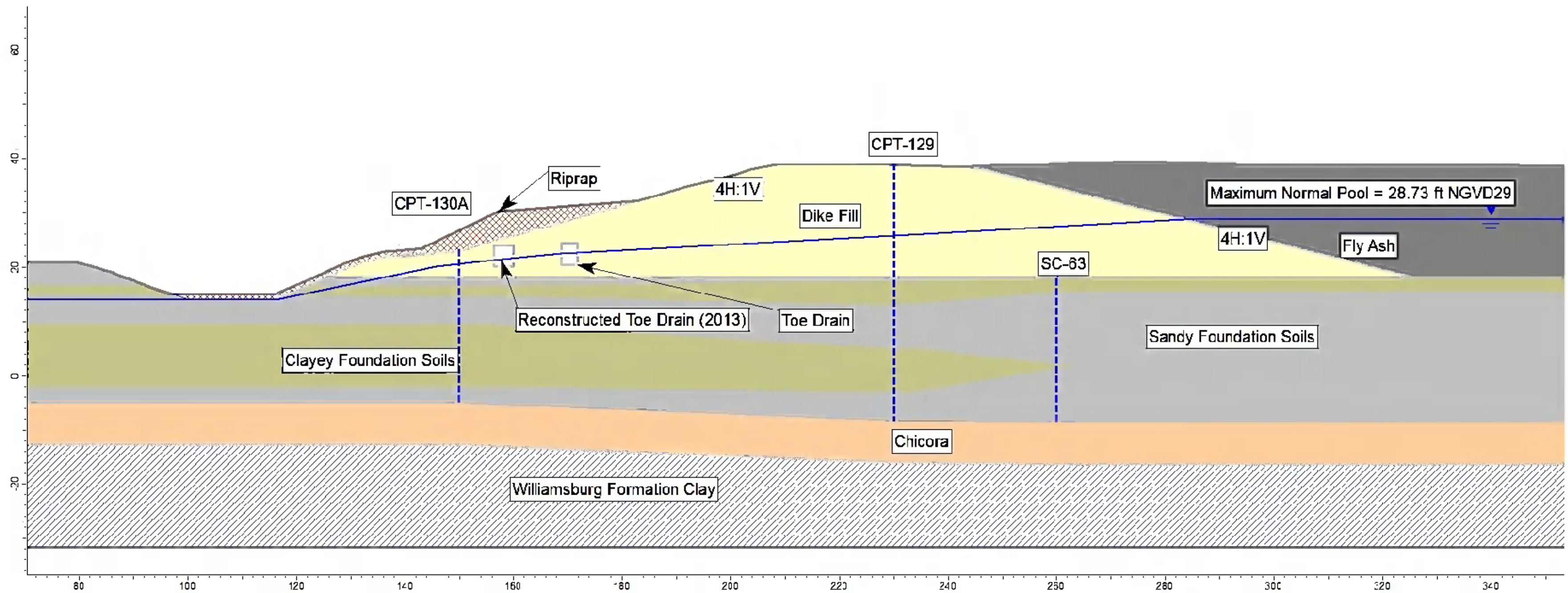

Figure 2. Cross Section A Geometry for Maximum Normal Storage Pool Conditions (Base Water Table)

Note:

1. A riprap buttress was constructed against the outer dike slope and across the adjacent drainage swale.

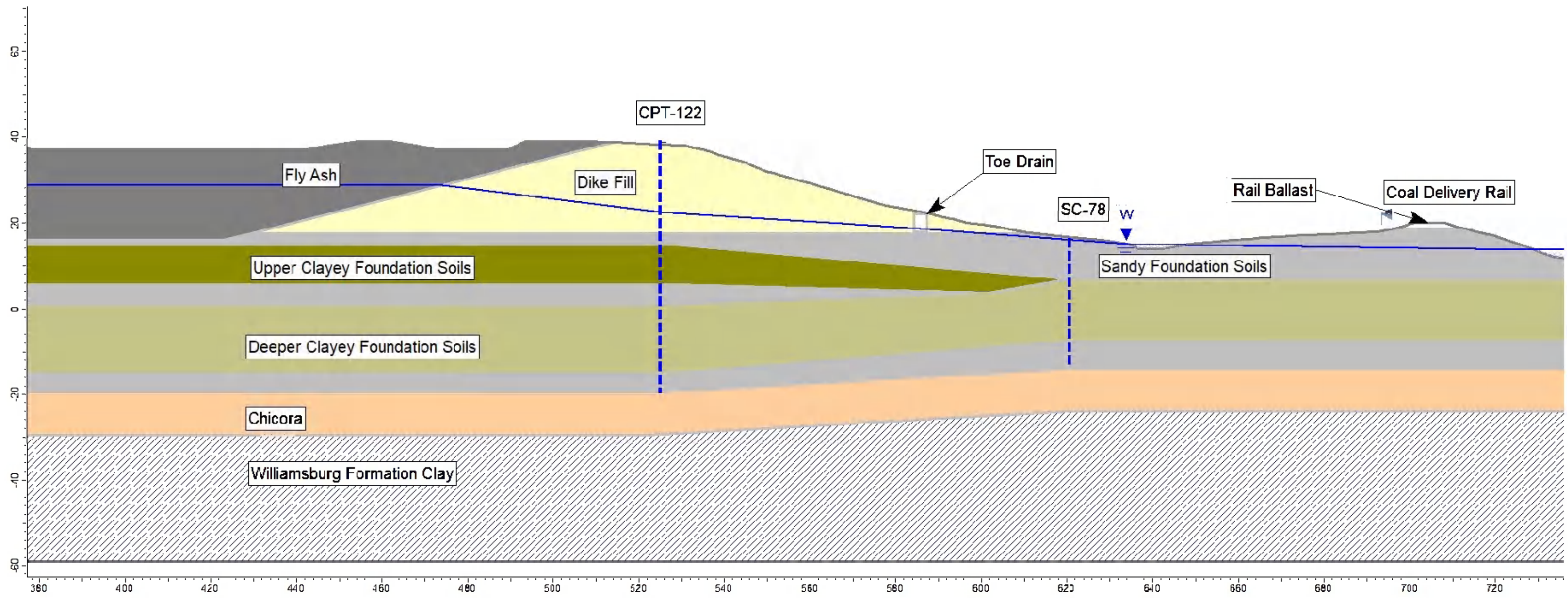

Figure 3. Cross Section B Geometry for Maximum Normal Storage Pool Conditions (Base Water Table)

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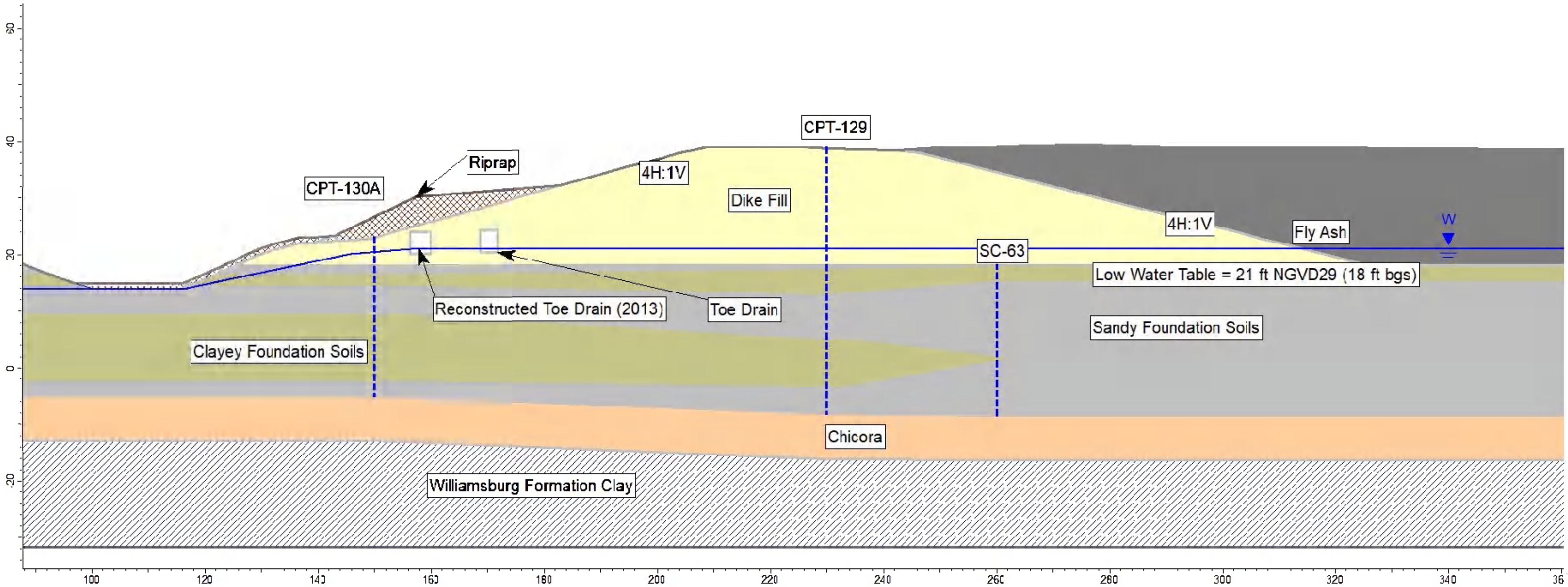


Figure 4. Cross Section A Geometry for Maximum Normal Storage Pool Conditions with Lowered Water Table

Note:

1. A riprap buttress was constructed against the outer dike slope and across the adjacent drainage swale.

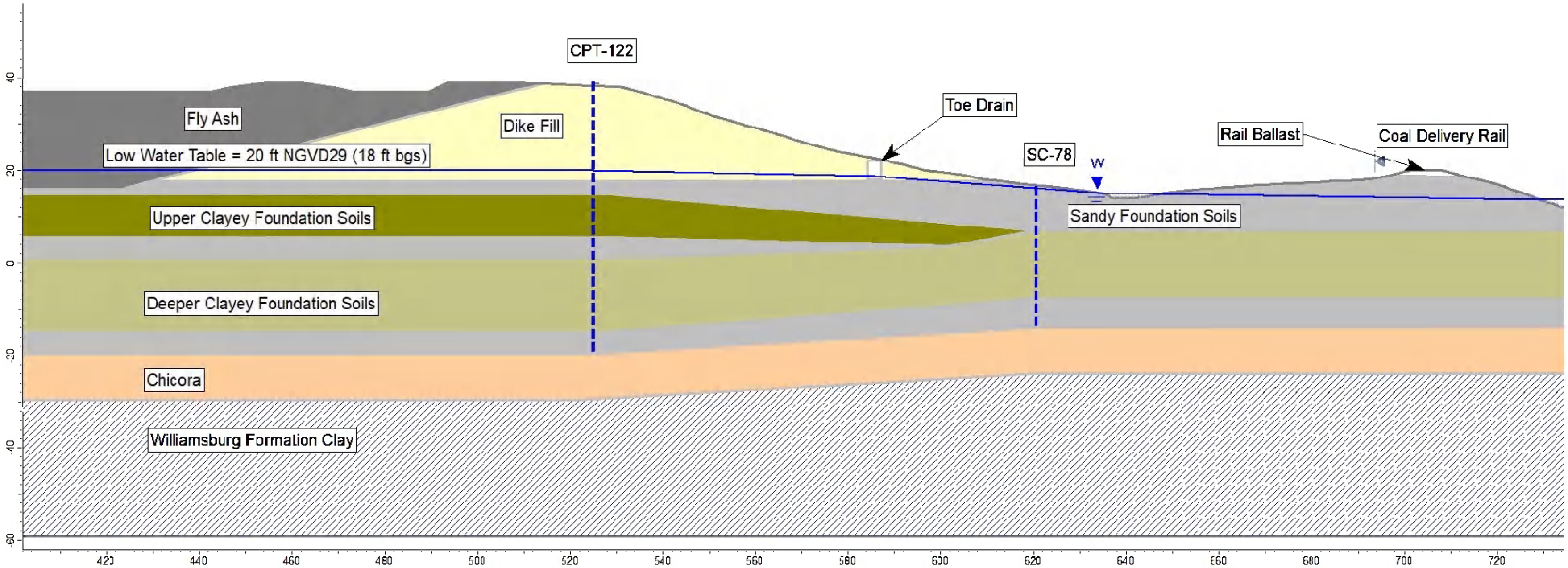


Figure 5. Cross Section B Geometry for Maximum Normal Storage Pool Conditions with Lowered Water Table

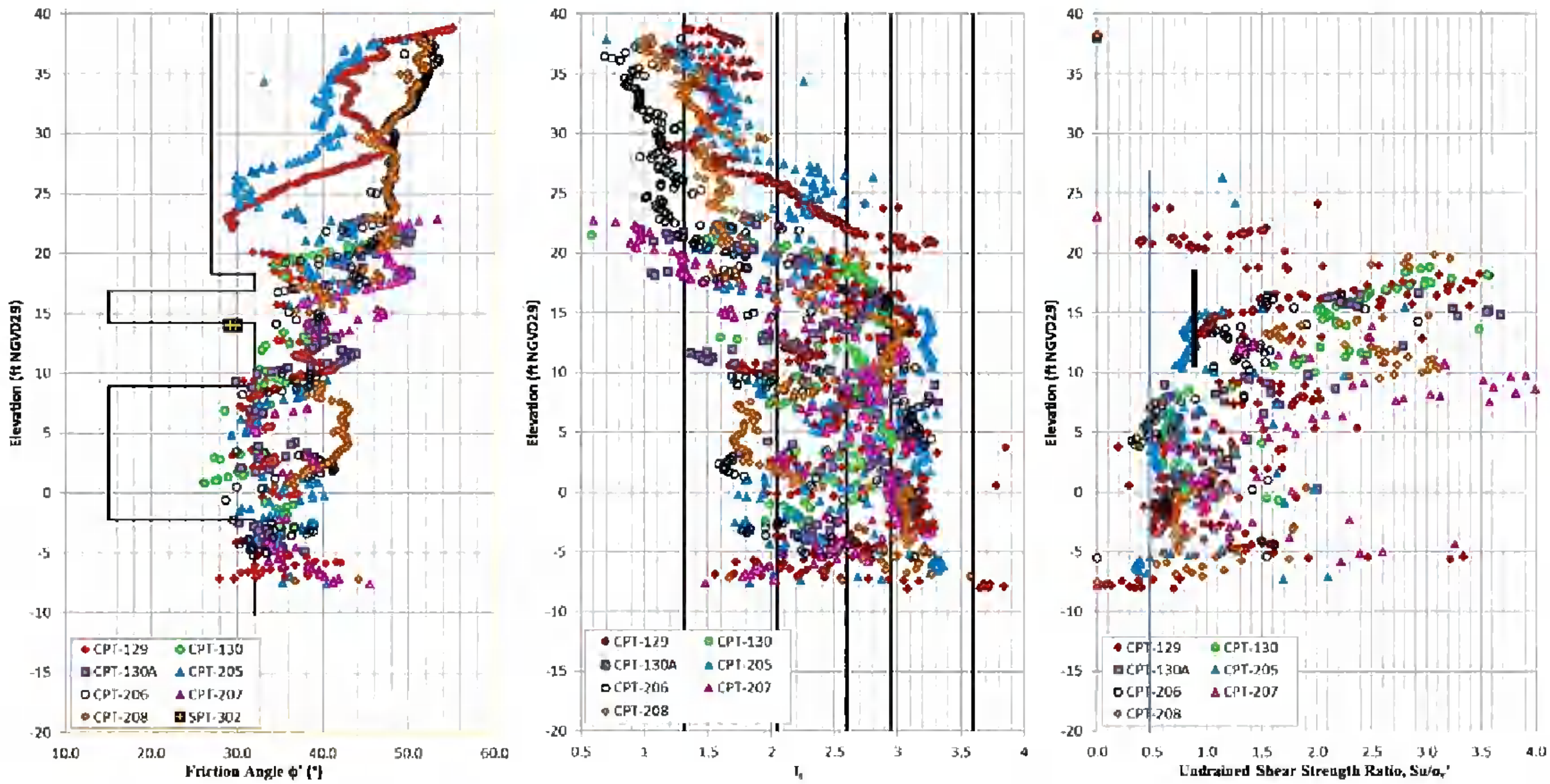


Figure 6. Subsurface Stratigraphy and Shear Strength Model for Cross Section A

Notes:

1. Clayey foundation soils were modeled with a ϕ'=15° and a c'=300 psf during static slope stability and with 80% of the $S_u/\sigma'_v = 0.5$ (i.e., $S_u/\sigma'_v = 0.4$) and 80% of the $S_{u,min}$ = 300 psf (i.e. $S_{u,min}$ = 240 psf) during seismic slope stability analysis.
2. The dike fill soils were conservatively modeled as a single material with ϕ' = 27°.
3. The sandy foundation soils were modeled with ϕ' = 32°.

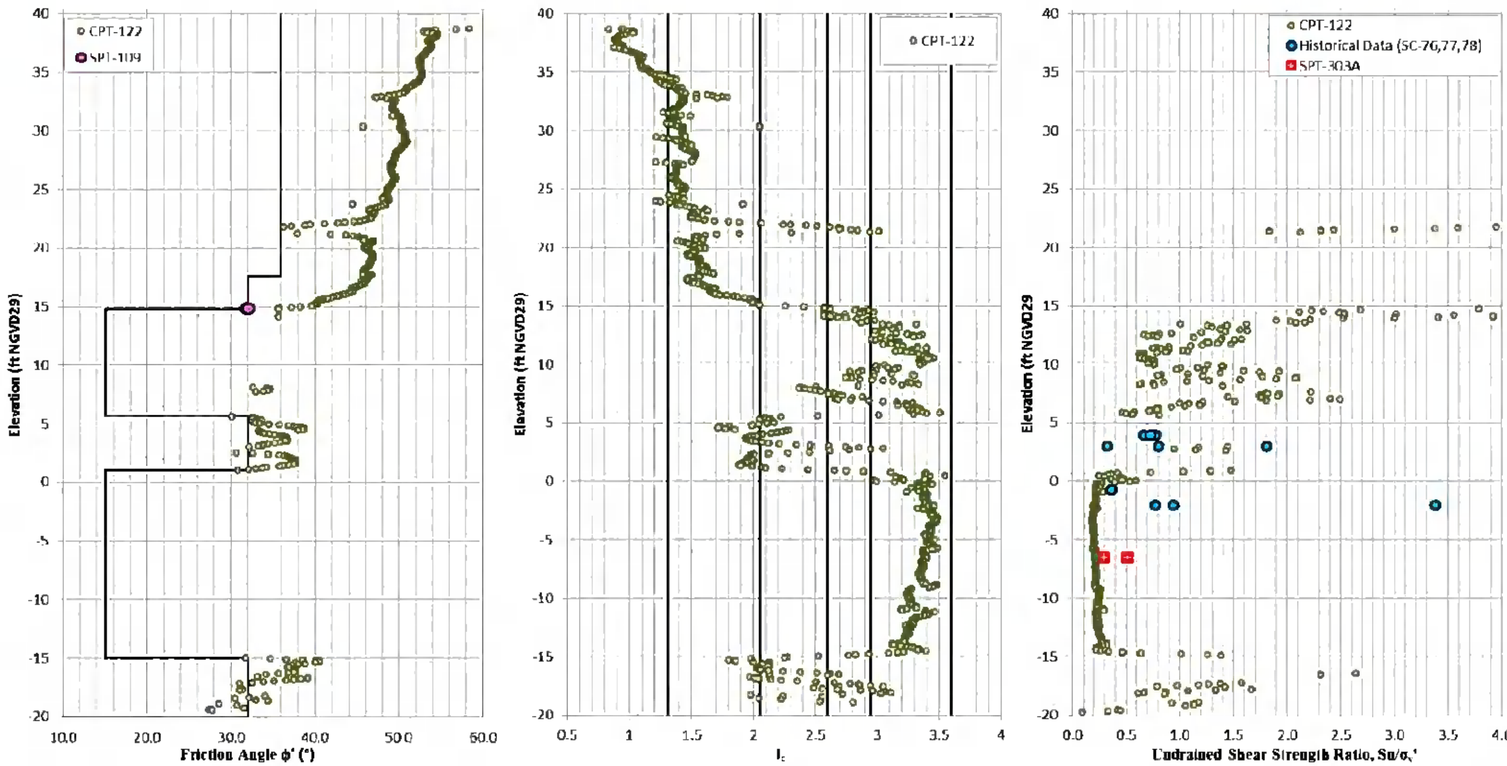


Figure 7. Subsurface Stratigraphy and Shear Strength Model for Cross Section B

Notes:

1. Clayey foundation soils (both upper clayey foundation soils and deeper clayey foundation soils) were modeled with a ϕ'=15° and a c'=300 psf during static slope stability. For pseudo-static stability analysis (i.e., seismic safety factor), the upper clayey foundation soils from 14.8 ft to 5.6 ft NGVD29 were modeled with 80% of the $S_u/\sigma'_v = 0.70$ (i.e. $S_u/\sigma'_v = 0.56$) and 80% of the $S_{u,min}$=300 psf (i.e. $S_{u,min}$=240 psf), and the deeper clayey foundation soils from 1.0 ft to -15.0 ft NGVD29 were modeled with 80% of the $S_u/\sigma'_v = 0.25$ (i.e. $S_u/\sigma'_v = 0.20$) and 80% of the $S_{u,min}$=300 psf (i.e. $S_{u,min}$=240 psf).
2. The sandy foundation soils were modeled with $\phi' = 32°$, and the dike fill soils were modeled with $\phi' = 36°$.

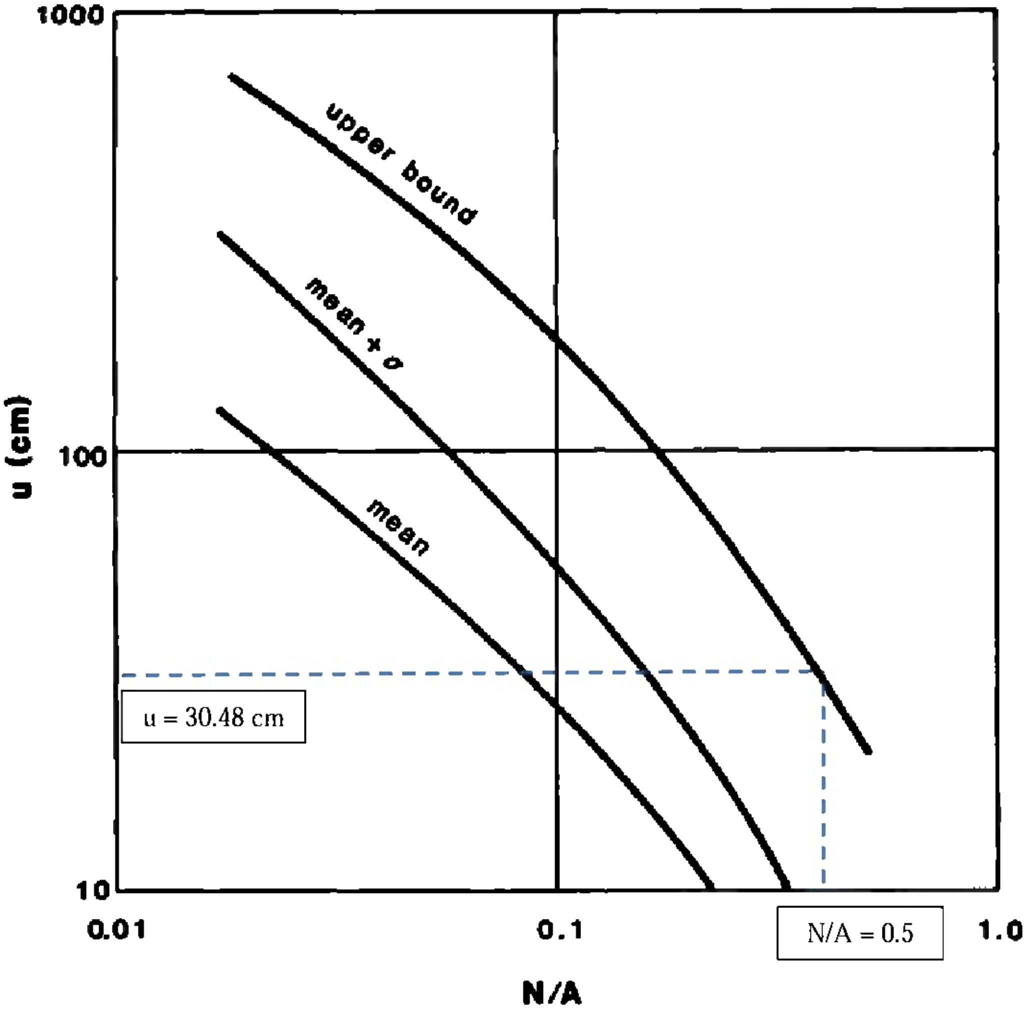
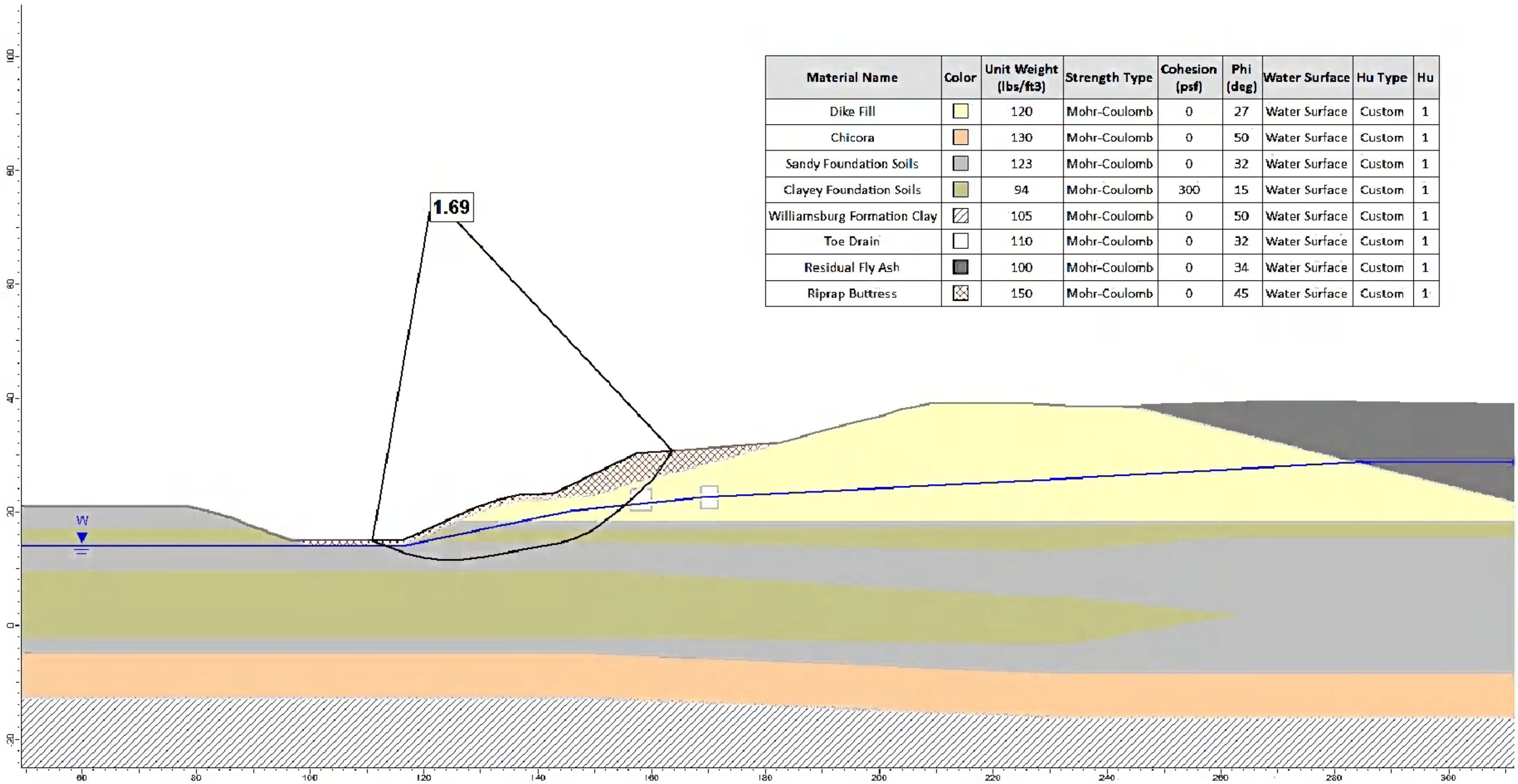


Figure 8. Allowable Deformation (u) vs. N/A
(from Figure 7 of Hynes-Griffin and Franklin, 1984)

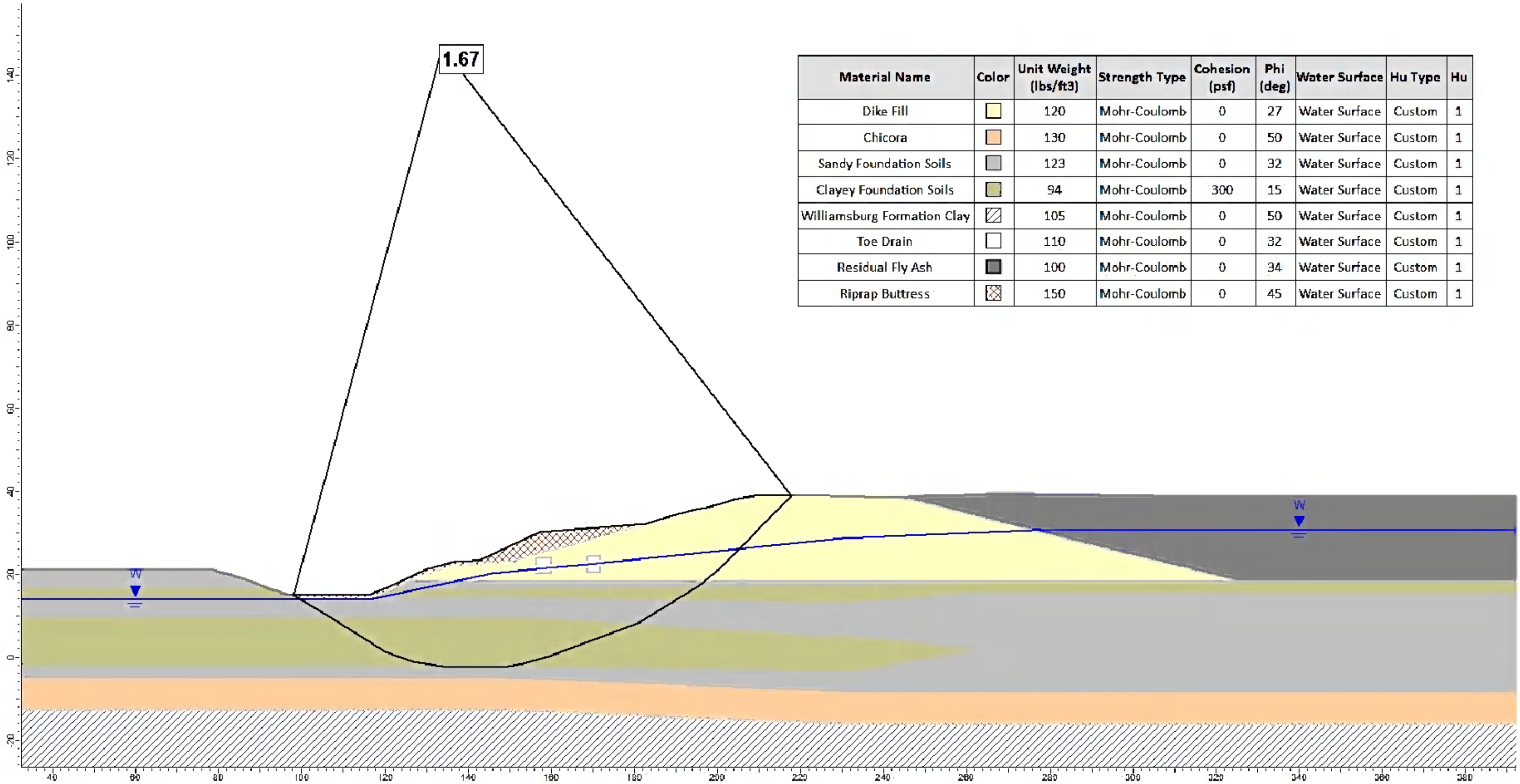
Notes:

1. An allowable deformation (u) of 12 in. (30.48 cm) and the "Upper Bound" curve were selected for the 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)	Water Surface	Hu Type	Hu
Dike Fill		120	Mohr-Coulomb	0	27	Water Surface	Custom	1
Chicora		130	Mohr-Coulomb	0	50	Water Surface	Custom	1
Sandy Foundation Soils		123	Mohr-Coulomb	0	32	Water Surface	Custom	1
Clayey Foundation Soils		94	Mohr-Coulomb	300	15	Water Surface	Custom	1
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50	Water Surface	Custom	1
Toe Drain		110	Mohr-Coulomb	0	32	Water Surface	Custom	1
Residual Fly Ash		100	Mohr-Coulomb	0	34	Water Surface	Custom	1
Riprap Buttress		150	Mohr-Coulomb	0	45	Water Surface	Custom	1

Figure 9. Calculated Factor of Safety for Cross Section A: Static Factor of Safety – Maximum Normal Storage Pool (Base Water Table)

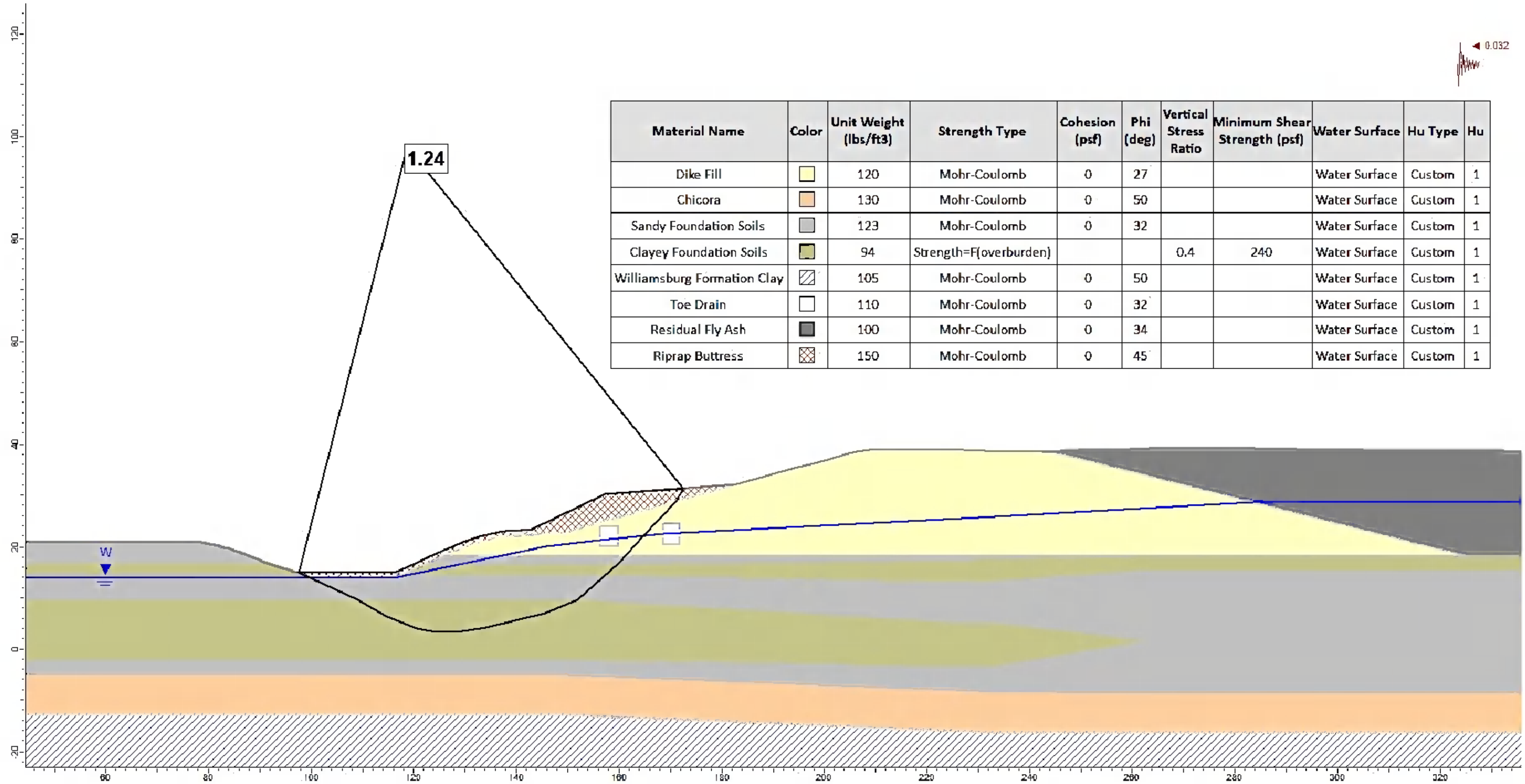


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Dike Fill		120	Mohr-Coulomb	0	27	Water Surface	Custom	1
Chicora		130	Mohr-Coulomb	0	50	Water Surface	Custom	1
Sandy Foundation Soils		123	Mohr-Coulomb	0	32	Water Surface	Custom	1
Clayey Foundation Soils		94	Mohr-Coulomb	300	15	Water Surface	Custom	1
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50	Water Surface	Custom	1
Toe Drain		110	Mohr-Coulomb	0	32	Water Surface	Custom	1
Residual Fly Ash		100	Mohr-Coulomb	0	34	Water Surface	Custom	1
Riprap Buttress		150	Mohr-Coulomb	0	45	Water Surface	Custom	1

Figure 10. Calculated Factor of Safety for Cross Section A: Static Factor of Safety - Maximum Surcharge Pool (Base Water Table)

Note:

[1] The maximum surcharge pool within the South Ash Pond was conservatively selected at 30.7 ft NGVD29.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Dike Fill		120	Mohr-Coulomb	0	27			Water Surface	Custom	1
Chicora		130	Mohr-Coulomb	0	50			Water Surface	Custom	1
Sandy Foundation Soils		123	Mohr-Coulomb	0	32			Water Surface	Custom	1
Clayey Foundation Soils		94	Strength=F(overburden)			0.4	240	Water Surface	Custom	1
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom	1
Toe Drain		110	Mohr-Coulomb	0	32			Water Surface	Custom	1
Residual Fly Ash		100	Mohr-Coulomb	0	34			Water Surface	Custom	1
Riprap Buttress		150	Mohr-Coulomb	0	45			Water Surface	Custom	1

Figure 11. Calculated Factor of Safety for Cross Section A: Seismic Factor of Safety – Maximum Normal Storage Pool (Base Water Table)

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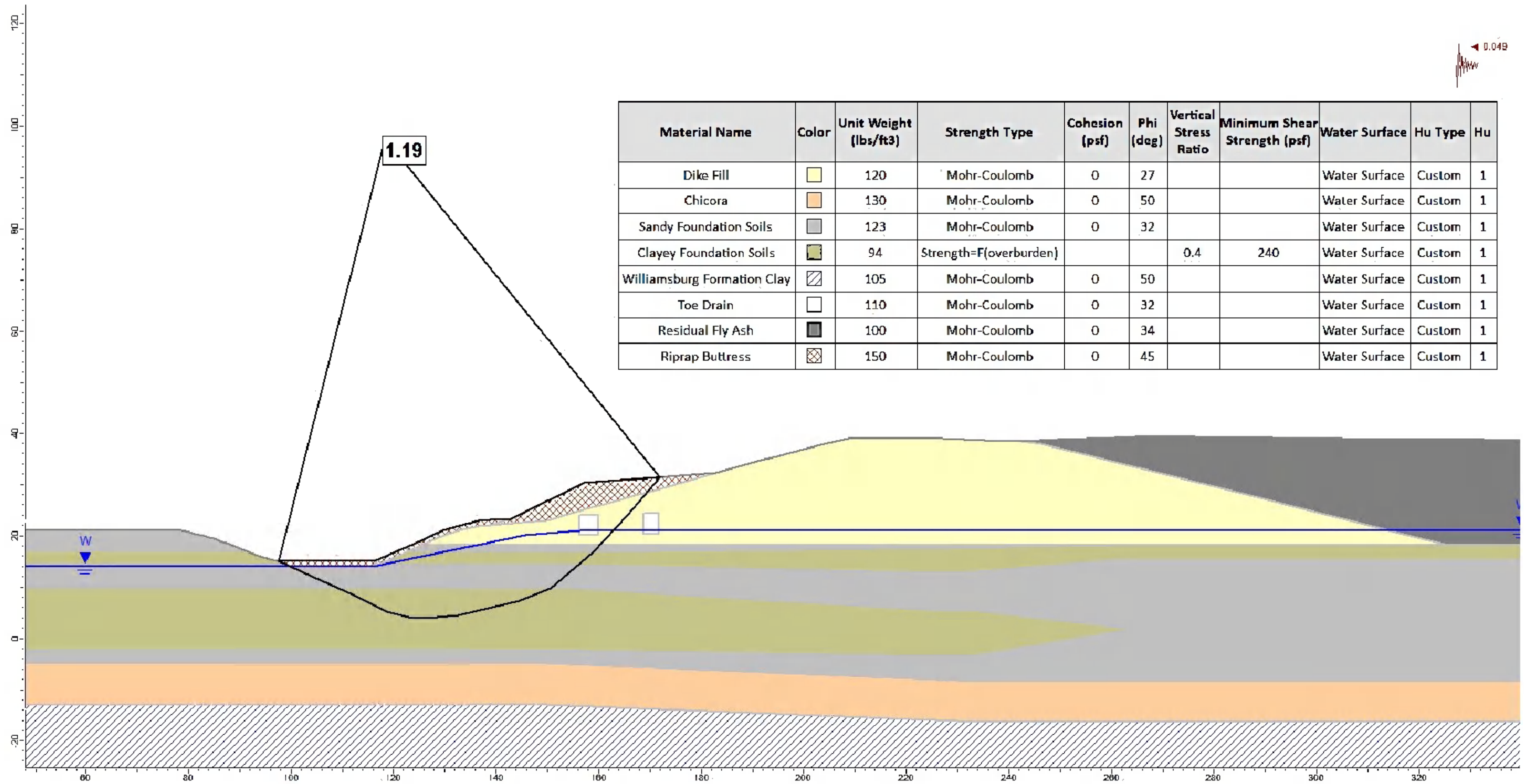


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Dike Fill		120	Mohr-Coulomb	0	27			Water Surface	Custom	1
Chicora		130	Mohr-Coulomb	0	50			Water Surface	Custom	1
Sandy Foundation Soils		123	Mohr-Coulomb	0	32			Water Surface	Custom	1
Clayey Foundation Soils		94	Strength=F(overburden)			0.4	240	Water Surface	Custom	1
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom	1
Toe Drain		110	Mohr-Coulomb	0	32			Water Surface	Custom	1
Residual Fly Ash		100	Mohr-Coulomb	0	34			Water Surface	Custom	1
Riprap Buttress		150	Mohr-Coulomb	0	45			Water Surface	Custom	1

Figure 12. Calculated Factor of Safety for Cross Section A: Seismic Factor of Safety – Maximum Normal Storage Pool (Lowered Water Table)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Dike Fill		120	Mohr-Coulomb	0	36	Water Surface	Custom	1
Chicora		130	Mohr-Coulomb	0	50	Water Surface	Custom	1
Sandy Foundation Soils		123	Mohr-Coulomb	0	32	Water Surface	Custom	1
Upper Clayey Foundation Soils		94	Mohr-Coulomb	300	15	Water Surface	Custom	1
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50	Water Surface	Custom	1
Rail Ballast		130	Mohr-Coulomb	0	35	Water Surface	Custom	1
Toe Drain		110	Mohr-Coulomb	0	32	Water Surface	Custom	1
Residual Fly Ash		100	Mohr-Coulomb	0	34	Water Surface	Custom	1
Lower Clayey Foundation Soils		94	Mohr-Coulomb	300	15	Water Surface	Custom	1

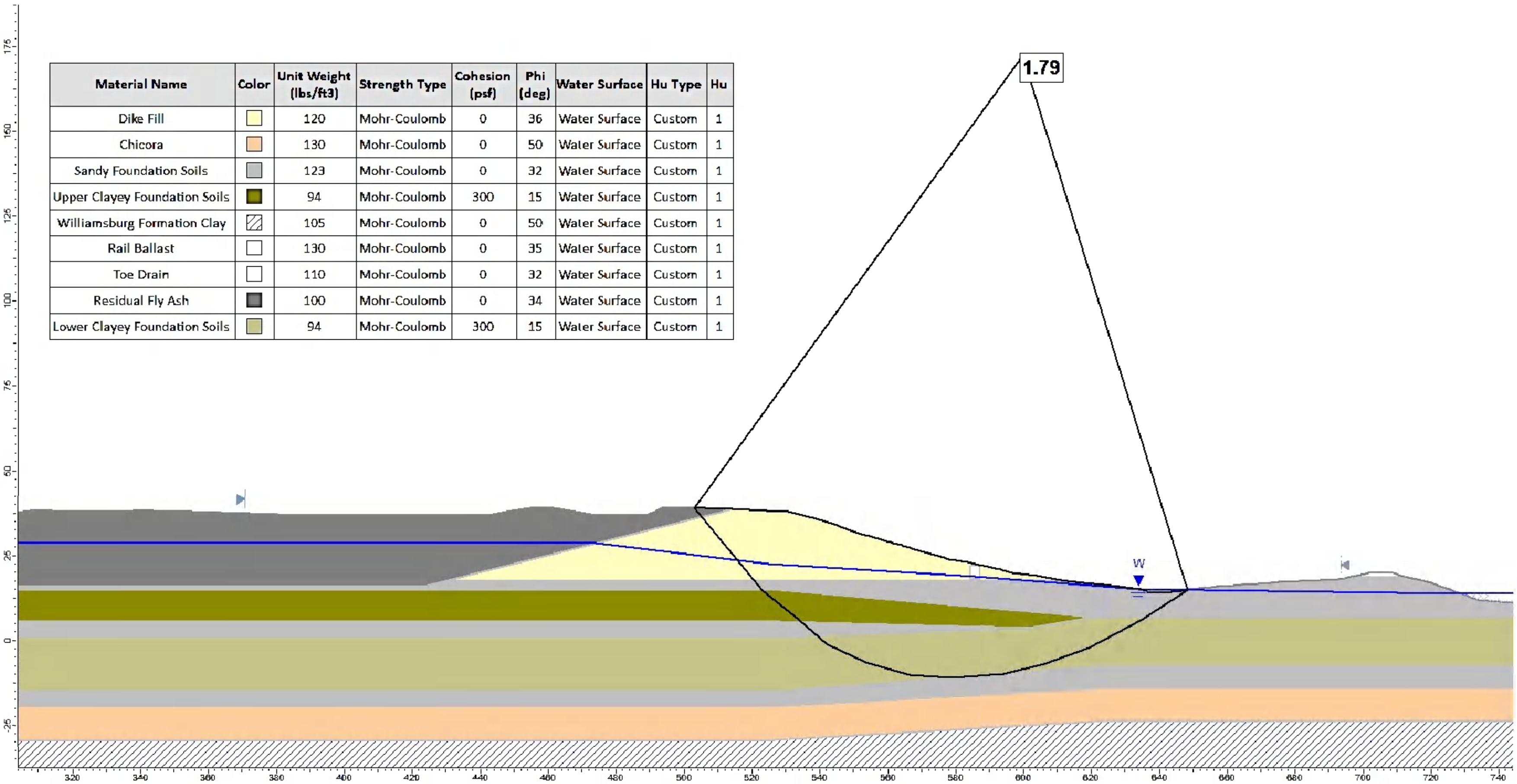


Figure 13. Calculated Factor of Safety for Cross Section B: Static Factor of Safety – Maximum Normal Storage Pool (Base Water Table)

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
Dike Fill		120	Mohr-Coulomb	0	36	Water Surface	Custom	1
Chicora		130	Mohr-Coulomb	0	50	Water Surface	Custom	1
Sandy Foundation Soils		123	Mohr-Coulomb	0	32	Water Surface	Custom	1
Upper Clayey Foundation Soils		94	Mohr-Coulomb	300	15	Water Surface	Custom	1
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50	Water Surface	Custom	1
Rail Ballast		130	Mohr-Coulomb	0	35	Water Surface	Custom	1
Toe Drain		110	Mohr-Coulomb	0	32	Water Surface	Custom	1
Residual Fly Ash		100	Mohr-Coulomb	0	34	Water Surface	Custom	1
Lower Clayey Foundation Soils		94	Mohr-Coulomb	300	15	Water Surface	Custom	1

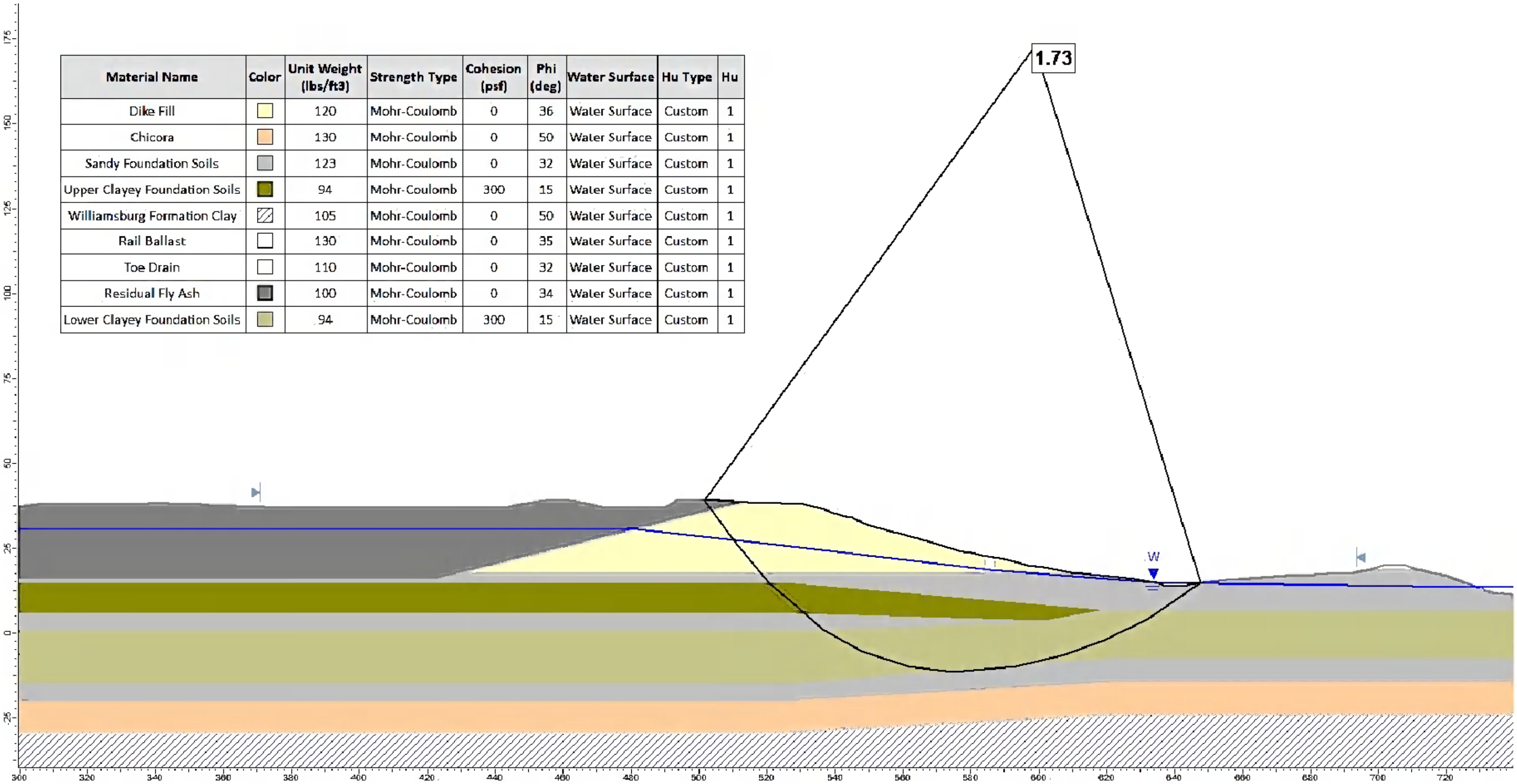
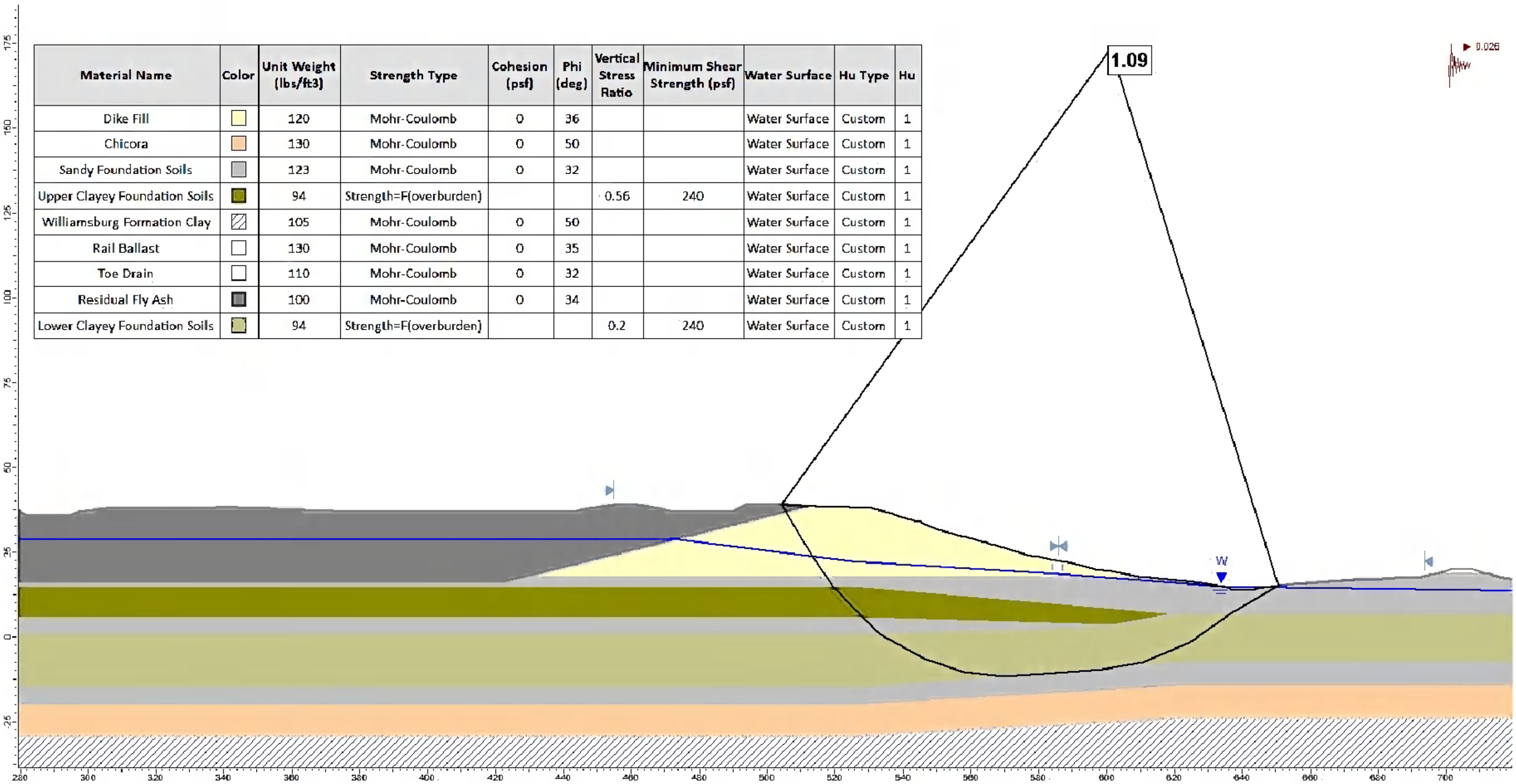


Figure 14. Calculated Factor of Safety for Cross Section B: Static Factor of Safety - Maximum Surcharge Pool (Base Water Table)

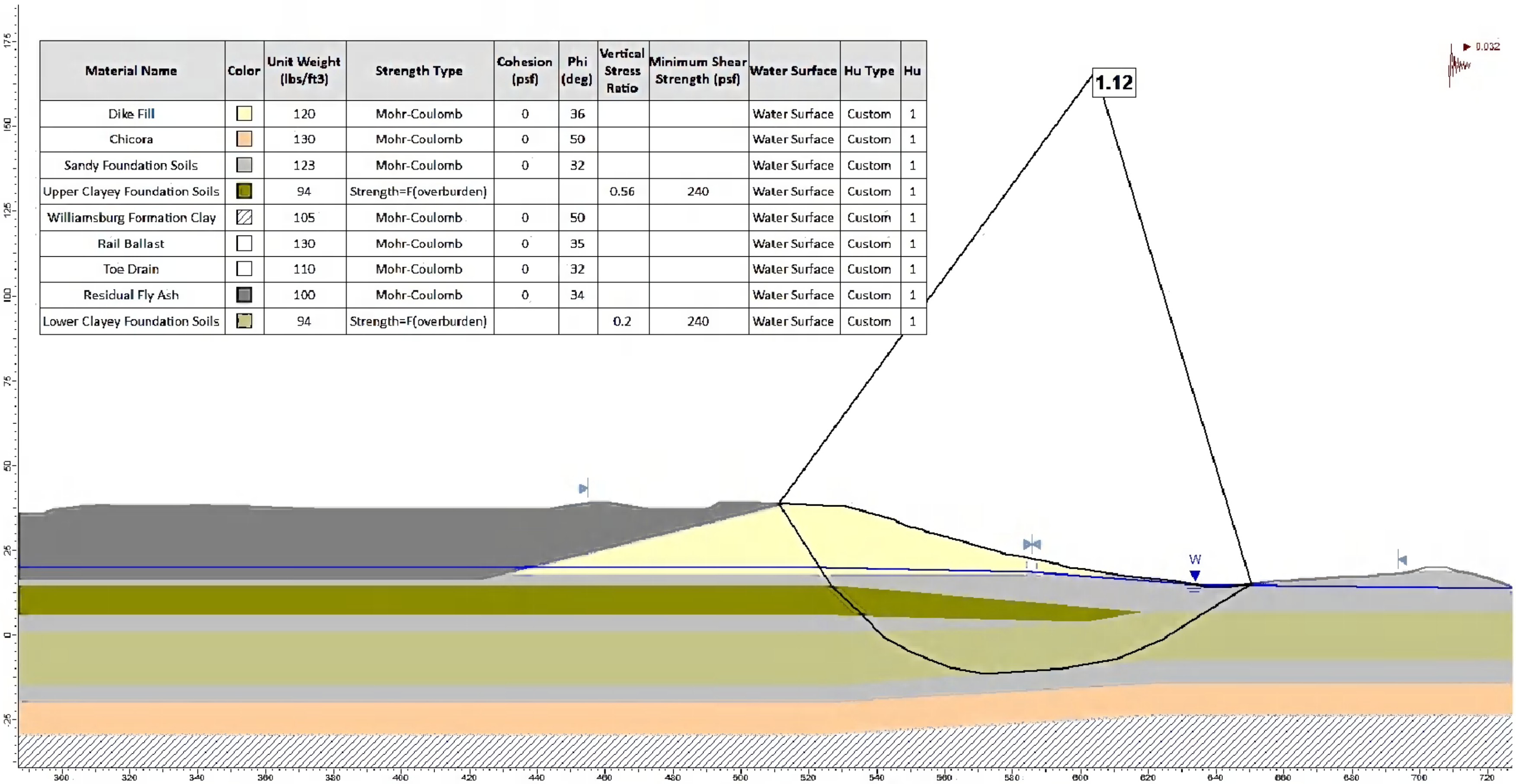
Note:

[1] The maximum surcharge pool within the South Ash Pond was conservatively selected at 30.7 ft NGVD29.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Dike Fill		120	Mohr-Coulomb	0	36			Water Surface	Custom	1
Chicora		130	Mohr-Coulomb	0	50			Water Surface	Custom	1
Sandy Foundation Soils		123	Mohr-Coulomb	0	32			Water Surface	Custom	1
Upper Clayey Foundation Soils		94	Strength=F(overburden)			0.56	240	Water Surface	Custom	1
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom	1
Rail Ballast		130	Mohr-Coulomb	0	35			Water Surface	Custom	1
Toe Drain		110	Mohr-Coulomb	0	32			Water Surface	Custom	1
Residual Fly Ash		100	Mohr-Coulomb	0	34			Water Surface	Custom	1
Lower Clayey Foundation Soils		94	Strength=F(overburden)			0.2	240	Water Surface	Custom	1

Figure 15. Calculated Factor of Safety for Cross Section B: Seismic Factor of Safety – Maximum Normal Storage Pool (Base Water Table)



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Dike Fill		120	Mohr-Coulomb	0	36			Water Surface	Custom	1
Chicora		130	Mohr-Coulomb	0	50			Water Surface	Custom	1
Sandy Foundation Soils		123	Mohr-Coulomb	0	32			Water Surface	Custom	1
Upper Clayey Foundation Soils		94	Strength=F(overburden)			0.56	240	Water Surface	Custom	1
Williamsburg Formation Clay		105	Mohr-Coulomb	0	50			Water Surface	Custom	1
Rail Ballast		130	Mohr-Coulomb	0	35			Water Surface	Custom	1
Toe Drain		110	Mohr-Coulomb	0	32			Water Surface	Custom	1
Residual Fly Ash		100	Mohr-Coulomb	0	34			Water Surface	Custom	1
Lower Clayey Foundation Soils		94	Strength=F(overburden)			0.2	240	Water Surface	Custom	1

Figure 16. Calculated Factor of Safety for Cross Section B: Seismic Factor of Safety – Maximum Normal Storage Pool (Lowered Water Table)