Prepared for

Santee Cooper Power
1 Riverwood Drive
Moncks Corner, South Carolina 29461

LOCATION RESTRICTIONS COMPLIANCE DEMONSTRATION

BOTTOM ASH POND
CROSS GENERATING STATION
PINEVILLE, SOUTH CAROLINA

Prepared by

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Certification Statement – Demonstration of Compliance with Location Restrictions

Federal CCR Rule: 40 CFR §257.60-64

CCR Unit: Bottom Ash Pond at Cross Generating Station

Certification:

I, **Majdi A. Othman**, a qualified professional engineer registered in the state of **South Carolina**, am the engineer-of-record for demonstration of compliance with location restrictions for the above-referenced coal combustion residual (CCR) Unit. Based on the evaluations presented in this Location Restrictions Compliance Demonstration Report, the Bottom Ash Pond does not meet the requirements of 40 CFR §257.60 for placement 5 feet above the uppermost aquifer. Therefore, the above-referenced CCR Unit is not, in my professional opinion, demonstrated to be in compliance with the United States Environmental Protection Agency (USEPA) minimum location restriction requirements for the siting criteria of 40 CFR §257.60-64 for existing CCR surface impoundments.

SOUTH CAROLINA REGISTERED PROFESSIONAL ENGINEER NO. 18188 MAJDI A. OTHMAN

SOUTH CAROLINA GEOSYNTEC CONSULTANTS, INC. No. 773 CERTIFICATE OF AUTHORIZATION

Firm Seal

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1. INTRODUCTION

Geosyntec Consultants (Geosyntec) prepared this *Location Restrictions Compliance Demonstration* (Report) on behalf of the South Carolina Public Service Authority doing business as (d.b.a.) Santee Cooper (Santee Cooper). The compliance demonstration pertains to the coal combustion residuals (CCR) unit referred to as the Bottom Ash Pond at the Cross Generating Station (CGS) located in Pineville, South Carolina (SC).

On 17 April 2015, the United States Environmental Protection Agency (USEPA) promulgated the federal CCR Rule that establishes national minimum criteria for existing and new CCR landfills and surface impoundments. The Bottom Ash Pond is subject to the CCR Rule as an existing surface impoundment as defined in 40 Code of Federal Regulations (CFR) §257.53, and as such the owner or operator is to demonstrate whether the CCR unit complies with the location restriction requirements under 40 CFR §257.60 through §257.64 and place appropriate documentation within the site's Operating Record. This Report serves as the location restrictions demonstration for the Bottom Ash Pond at CGS.

1.1 Facility Location

CGS is a coal-fired electric generating facility with four generation units and is located at 533 Cross Station Road, Pineville, SC 29468. CGS is owned and operated by Santee Cooper. CGS is located approximately ten miles southwest of the town of Pineville, Berkeley County, SC, and is accessed via SC Hwy 45 to Viper Road. CGS is located along a diversion canal that connects Lake Marion to Lake Moultrie from northwest to the southeast adjacent to the property boundary. A general site vicinity map is presented on Figure 1. CGS includes an approximately 513-acre parcel utilized for station operations and an adjacent approximately 1,720-acre parcel which contains CCR ponds, two CCR landfills, and undeveloped forest land.

The Bottom Ash Pond was constructed in two phases. The original Bottom Ash Pond 1 was constructed in 1982 with a bentonite liner; while Bottom Ash Pond 2 was constructed in 1993 and lined with a geosynthetic clay liner (GCL). The two surface impoundments were connected via a 10-ft wide trapezoidal spillway with 3 horizontal to 1 vertical (3H:1V) side slopes through the northern embankment of Bottom Ash Pond 1. In 2015, all CCR were removed from Bottom Ash Pond 1 and it was repurposed to manage wastewater. The surface impoundment was renamed the Wastewater Decant Pond (WorleyParsons, 2016a). The ponds are currently connected via a trapezoidal spillway through the northern embankment of the Wastewater Decant Pond. The spillway is 10 feet wide with 3H:1V side slopes and is covered with concrete revetment. The former Bottom Ash Pond 2 is now commonly referred to as the Bottom Ash Pond, is shown in Figure 2, and is the subject of the demonstration presented herein. The Bottom Ash Pond contains a total storage capacity of approximately 1,868,240 cubic yards (cy). As of the 2017 annual inspection, the volume of CCR stored within the Bottom Ash Pond was estimated as 879,000 cy (Santee Cooper, 2017).

The Bottom Ash Pond is primarily used for storage of bottom ash and boiler slag from the four generating units at CGS. Units 1 and 2 each sluice bottom ash through a dedicated pipeline to the Bottom Ash Pond; while, Units 3 and 4 sluice bottom ash through a single pipeline for disposal. A separate pipeline conveys pyrites, economizer ash, and FGD by-products from Units 3 and 4. The Bottom Ash Pond also receives water from several sources: (i) the Coal Pile Runoff Pond; (ii) the Landfill Leachate Collection Pond; (iii) the Unit 1 and 2 Stormwater Pond; (iv) the Unit 3 and 4 Stormwater Pond; and (v) numerous station drainage sumps. The Bottom Ash Pond formerly received decant water from the Gypsum Pond, which was closed by removal on 11 March 2017 (WorleyParsons, 2016b).

The *Existing Bottom Ash Pond Liner Certification* (WorleyParsons, 2016c), located within the operating record, concluded the GCL does not meet the requirements of §257.71 of the CCR Rule. The Bottom Ash Pond is considered an existing unlined CCR surface impoundment and is subject to the requirements of §257.101.

1.2 Previous Investigations and Reports

For numerous projects, Santee Cooper implemented subsurface investigations at the CGS to collect geologic, hydrogeologic, and geotechnical data, several of which collected data within and directly adjacent to the Bottom Ash Pond footprint. This Report was prepared and is supported by the detailed information contained within the following reports:

- *Final Report Cross Generating Station,* Law Engineering Testing Company, 9 February 1978;

- *Final Report Unit 1 Generating Station*, Woodward-Clyde Consultants, 26 January 1981;
- *Site Hydrogeologic Characterization Report*, Cross Generating Station Proposed Class Three Landfill, October 2011, prepared by Garrett & Moore; and
- *Landfill Siting Study*, Cross Generating Station, Pinewood, South Carolina, April 2016, prepared by Garrett & Moore.

1.3 Site Geology and Hydrogeology

The lithostratigraphic units at the CGS, in descending order, include Holocene sediments, the Wicomico Formation, the Raysor Formation, Santee Limestone and the Black Mingo Group.

Holocene sediments are sparsely distributed at CGS and consist typically of loose, silty or clayey fine sand with abundant organic material. "Wicomico sediments encountered at the CGS are predominantly soft, clayey sands and sandy clays varying in texture from fine to coarse and range in thickness from approximately 12 to 39 feet. The sandy clay and clayey sand are interbedded with silty fine to coarse sand and localized clay and relatively clean sand. The sandy clay, silty sand and clay beds are of variable thickness and discontinuous and appear to transition laterally and vertically into clayey sand" (Garrett & Moore, 2011). The Raysor Formation sediments are discontinuous at CGS and are generally, unconsolidated to partially indurated, shelly, fine to medium sand (calcarenite). In general, the Raysor Formation sediments are relatively dense; however, some soft zones were encountered during drilling and vary in thickness from approximately 5 to 17 feet.

The Santee Limestone encountered at the CGS consists of a variably weathered crystalline, soft to hard, medium to light gray, shelly to muddy limestone. Rock Quality Designation (RQD) values of recovered rock cores varied considerably from 0 to 96 percent, with most values falling in the range of approximately 0 to 60 percent. In general, lower RQD values were observed near the top and bottom of the geologic unit, while basal gray to greenish gray, shelly, silty to clayey, fine to medium sand layer was observed within many soil borings. The thickness of the Santee Limestone ranges from approximately 10 to 60 feet. The basal sand layer was likely reworked from the underlying greenish gray, silty sand of the upper Black Mingo Group sediments during initial deposition of the Santee Limestone.

The surficial aquifer at CGS is unconfined and includes the saturated sediments of the Wicomico Formation and the underlying Raysor Formation. Groundwater recharge to the surficial aquifer occurs via direct precipitation infiltration. Hydrogeological characterization at CGS did not provide evidence of a laterally continuous, definable confining unit that separates the surficial aquifer from the underlying Santee Limestone. "Consequently, the surficial aquifer is directly hydraulically connected to the underlying regional Santee Limestone aquifer" (Garrett & Moore, 2011).

Sinkholes (karst) are natural geologic features that occur in areas underlain by limestone and other types of soluble rock. Limestone is susceptible to dissolution from the percolation of slightly acidic groundwater. Limestone composition also controls dissolution and cavity development. Pure limestone is more easily dissolved by natural waters; however, the presence of impurities (such as quartz sand and clay) within the rock will reduce and limit the rate of dissolution.

The type of sinkhole that may develop in a given area is largely controlled by the geology and hydrogeology at a specific site. Limestone, like most bedrock, generally lies beneath unconsolidated material such as sand and clay. The variable thickness and composition of the overlying soil is important in sinkhole development. There are three general types of sinkholes: solution sinkholes, cover-subsidence sinkholes, and cover-collapse sinkholes. Conditions at CGS are most conducive to the formation of cover-subsidence sinkholes.

Cover-subsidence sinkholes occur where the overlying soil is relatively incohesive and permeable and individual grains of sand move downward in sequence to replace grains that have themselves moved downward to occupy space formerly held by the dissolved limestone. In areas where the overlying sand soils are 50 to 100 feet thick, subsidence sinkholes generally are only a few feet in diameter and depth. Where the limestone is buried beneath a sufficient thickness of unconsolidated material, few sinkholes generally occur. Spalling of sand into solution cavities that have developed along joints in the limestone may cause subsidence due to upward migration of the cavities (a process known as piping) to form cylindrical holes at the land surface. If the overburden is non-cohesive sand, the upward-migrating cavity is dissipated by a general lessening of density over a large area, and the result will be a relatively broad and extensive subsidence of the land surface that occurs over a period of time.

2. LOCATION RESTRICTIONS EVALUATION

The location restrictions under §257.60 through §257.64 include: (1) placement above the uppermost aquifer; (2) wetlands; (3) fault areas; (4) seismic impact zones; and (5) unstable areas. The following sections describe the assessments conducted within this Report to demonstrate compliance of the Bottom Ash Pond with the above location restrictions.

2.1 Placement Above the Uppermost Aquifer

40 CFR §257.60(a) states that existing surface impoundments "*must be constructed with a base that is located no less than 1.52 meters (five feet) above the upper limit of the uppermost aquifer, or must demonstrate that there will not be an intermittent, recurring, or sustained hydraulic connection between any portion of the base of the CCR unit and the uppermost aquifer due to normal fluctuations in groundwater elevations (including the seasonal high water table).*" The "uppermost aquifer" is defined by §257.40 as the geologic formation nearest the natural ground surface that is an aquifer, as well as lower aquifers that are hydraulically interconnected with the upper aquifer within a facility's property boundary. The definition includes a shallow, deep, perched, confined or unconfined aquifer, that yields usable water.

The uppermost aquifer at the CGS is the surficial aquifer, which is an unconfined aquifer that contains predominantly sand with minor amounts of silt and clay. The seasonal high water table is typically interpreted by twelve months of groundwater elevation data obtained from a representative number of monitoring wells. Data collected from monitoring wells which surround the Bottom Ash Pond between January 2016 and February 2018 was reviewed to evaluate separation of CCR from the uppermost aquifer. The seasonal high groundwater table elevation in the vicinity of the Bottom Ash Pond is approximately 77.5 feet (ft) based on the North American Vertical Datum of 1988 (NAVD88). The base of the Bottom Ash Pond is approximately 73 ft NAVD88. As such, the Bottom Ash Pond does not meet the requirements of 40 CFR §257.60 for placement 5 feet above the uppermost aquifer. Further investigation of the potential intermittent, recurring or sustained hydraulic connection was not performed.

2.2 Wetlands

40 CFR §257.61(a) states that existing surface impoundments "*must not be located in wetlands, as defined in §232.2 of this chapter, unless the owner or operator demonstrates… that the CCR unit meets the requirements of paragraph (a)(1) through (5) of this section.*" Wetlands, as defined in 40 CFR §232.2, means "*those areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas.*"

Wastewater treatment systems, which include wastewater treatment ponds designed to meet the requirements of the Clean Water Act (CWA), are not waters of the United States and are exempt from permit requirements under Section 404 of the CWA. Wetlands that may exist within these boundaries are exempt from CWA permits because the CCR surface impoundments are considered wastewater treatment system component, which is permitted and operated under National Pollutant Discharge Elimination System (NPDES) Permit No. SC0037401. A demonstration to show that the Bottom Ash Pond meets the requirements of paragraphs (a)(1) through (a)(5) of 40 CFR §257.61 is not necessary since the CCR unit is not located within areas delineated or defined as wetlands. The Bottom Ash Pond is judged to be in compliance with the requirements of 40 CFR §257.61 for wetlands.

2.3 Fault Areas

40 CFR §257.62(a) states that existing surface impoundments "*must not be located within 60 meters (200 feet) of the outermost damage zone of a fault that has had displacement in Holocene time unless the owner or operator demonstrates by the dates specified in paragraph (c) of this section that an alternative setback distance of less than 60 meters (200 feet) will prevent damage to the structural integrity of the CCR unit.*"

A summary of the known structural features in the state of South Carolina is provided in Maybin et al. (1998). Based on Maybin (1998), no structural features indicative of recent (Holocene-age) fault movements have been identified within ten miles of CGS. However, an inferred fault south of Lake Moultrie was identified, but this inferred fault is located significantly beyond the 200-foot location restriction required by 40 CFR §257.62(a).

As such, the Bottom Ash Pond is judged to be in compliance with the requirements of 40 CFR §257.62 for fault areas.

2.4 Seismic Impact Zones

40 CFR §257.63(a) states that existing surface impoundments must not be located in seismic impact zones unless the owner or operator makes certain demonstrations. A seismic impact zone is defined as *"an area having a 2% or greater probability that the maximum expected horizontal acceleration, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10 g in 50 years."* Seismic zones, which represent areas of the United States with the greatest seismic risk, are identified on United States Geological Survey (USGS) national seismic hazard maps as well as regional seismic hazard maps developed by local experts that consider the regional geologic setting and seismicity.

As identified in the *Seismic Impact Zone Hazard Analysis (*Appendix D of the *Cross Station Proposed Class Three Landfill Permit Application* (Garrett & Moore, 2011), CGS is located in a seismic impact zone. Accordingly, 40 CFR §257.63(a) requires a demonstration 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."* Garrett & Moore evaluated three sources to select the peak ground acceleration (PGA) associated with the design seismic event.

1. The USGS Interactive Seismic Hazard Map (2008) estimates the PGA at the hypothetical bedrock outcrop as 1.0348g. Since bedrock is located 1900 ft below ground surface and the USGS hazard map is heavily influenced by conditions within Charleston, SC, additional evaluation or consideration of the PGA at the ground surface was warranted to account for local site conditions. Garrett & Moore (2011) considered more regional resources to estimate the PGA at CGS.

2. The South Carolina Emergency Preparedness Division (SCEPD)'s report titled "Comprehensive Seismic Risk and Vulnerability Study for the State of South Carolina" (URS Corporation, 2001) predicted ground motions for a Magnitude 7.3 earthquake in Charleston for the 2500-yr return period (2% in 50 years ground motion). SCEPD identifies that the estimated PGA at the ground surface is approximately 0.35g.

3. The South Carolina Department of Transportation (SCDOT) (2008) presents hazard maps developed by Chapman and Talwani (2006) to estimate the rock outcrop and geologically realistic PGA in South Carolina. The SCDOT seismic hazard map indicates that the estimated geologically realistic PGA at CGS is approximately 0.55g for the 2,500-year return period.

The SCEPD and SCDOT hazard maps indicate the geologically realistic PGA at CGS is less than that published within the USGS National Seismic Hazard Maps. The SCDOT value of 0.55g provides the more conservative estimate and was recommended and selected by Garrett & Moore (2011) to design the onsite CCR landfill. The Bottom Ash Pond evaluation used the same PGA to determine if it was designed to resist the maximum horizontal acceleration in lithified earth material.

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

1. Estimate the maximum horizontal earthquake acceleration for the potential critical slip surfaces of the perimeter dike system.

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 an allowable displacement (u) in 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 geologically realistic PGA was selected as the peak earthquake acceleration, A.

3. Perform slope stability analysis applying the seismic horizontal force coefficient to compute a horizontal force ($F = k_h \times W$), for each slice based on slice weight (W), and evaluate the resulting Factor of Safety (FS). If the calculated FS meets or exceeds the target FS (i.e., FS $\geq$ 1.0), the slope is considered to be stable and to meet the requirements of the CCR Rule.

During pseudo-static slope stability analyses, undrained shear strengths should be reduced by 20 percent to account for potential strength degradation during cyclic loading of the earthquake. The k_h must be computed under the assumption that an allowable displacement (u) is acceptable. An allowable displacement of 12 inches (30.48 centimeters) was selected for the Bottom Ash Pond perimeter dike structures. Using the Hynes-Griffin and Franklin (1984) chart and assuming the "Upper Bound" displacement, the ratio of N /A (or k_h / PGA) was conservatively selected as 0.50, as shown in Figure 3.

The critical cross section presented within the *Bottom Ash Pond Initial Safety Factor Assessment* (WorleyParsons, 2016a) was evaluated as a part of this Report. Peak undrained shear strengths presented in Figure 2 of the *Bottom Ash Pond History of Construction* (WorleyParsons, 2016b) were selected for embankment, soft embankment, and clay strata as 1,440 pounds per square (psf), 500 psf, and 1200 psf, respectively. The selected strengths were reduced by 20 percent and the critical cross section was evaluated by Spencer's Method (Spencer, 1967), as implemented within the computer program SLIDE® (Rocscience, 2018), with a seismic coefficient (k_h) of 0.28. As shown in Figure 4, a FS of 1.11, which exceeds the minimum FS of 1.0, was computed under these conditions and the perimeter dikes were considered to be designed to resist the maximum horizontal acceleration.

As such, Bottom Ash Pond is considered to be in compliance with the requirements of 40 CFR §257.63 for seismic impact zones.

2.5 Unstable Areas

40 CFR §257.64(a) indicates that existing surface impoundments "*must not be located in an unstable area unless the owner or operator demonstrates… that recognized and generally accepted good engineering practices have been incorporated into the design of the CCR unit to ensure that the integrity of the structural components of the CCR unit will not be disrupted.*" An unstable area is defined as "*a location that is susceptible to natural or human-induced events or forces capable of impairing the integrity, including structural components of some or all of the CCR unit that are responsible for preventing releases from such unit. Unstable areas can include poor foundation conditions, areas susceptible to mass movements, and karst terrains.*" Historical subsurface investigations and reports indicate the following relevant information with respect to unstable areas in the vicinity of the Bottom Ash Pond.

- CGS is not situated in an area with geologic features or the potential for geomorphically-induced phenomena that could be indicators of susceptibility to mass movements (i.e., landslides, avalanches, debris slides and flows, soil flocculation, block sliding, rock falls, or excessive surface erosion).

- CGS is not situated in an area that is subject to excessive coastal or river erosion.

- CGS is not situated in an area of known subsurface mines, or in an area experiencing significant water or mineral withdrawal, nor do there appear to be evidence of other human-made features or man-induced events that could result in the downslope transport of soil and rock material that would make the CCR unit susceptible to mass movements or otherwise impair the integrity of the unit.

- CGS is not situated (as previously discussed) in an area where active faults have been observed.

- CGS is known to be situated in an area that may be classified as karst terrain.

- CGS and more specifically the Bottom Ash Pond may be underlain by weaker soil strata or soils that may experience loss in shear strength.

To assess whether the Bottom Ash Pond is situated within an unstable area, following conditions were evaluated:

- On-site or local soil conditions that may result in differential settlements;

- On-site or local soil conditions that may constitute poor foundation conditions;

- On-site or local geologic or geomorphologic features (i.e., potential karst terrain); and

- On-site or local human-made features or events (both surface and subsurface).

2.5.1 Human-Made Features

An underdrain system was installed to dewater the foundation soils and facilitate installation of the bentonite liner in the original Bottom Ash Pond 1. Five inch diameter perforated pipes bedded in sand drained the foundation soils into 12-inch diameter header pipes and ultimately to a junction box within the center of Bottom Ash Pond 1. The junction box conveyed water northward through two (2) 12-inch diameter high density polyethylene (HDPE) pipes underneath the perimeter dike structure to a concrete manhole immediately outside of the footprint. Upon completion of pond construction, the manhole and pipes were grouted and abandoned. In 2014, Santee Cooper excavated the soils adjacent to the manhole and confirmed that the manhole and at least 10 ft of the adjacent pipes were grouted. The excavated area was backfilled with flowable fill and a rock drain was installed to minimize piping erosion (Worley Parsons, 2016b). As such, the dewatering pipes under the perimeter dikes and manhole appear to be properly abandoned and do not appear to be a potential unstable area.

2.5.2 Differential Settlement

The Bottom Ash Pond was evaluated to assess the potential of differential settlements on the CCR unit structural integrity as the structure is underlain by loose to medium dense silty and clayey sands and soft to stiff silts and clays, with variable compressibility. The Bottom Ash Pond was designed and constructed with a crest elevation of approximately 91 ft NAVD88 (Worley Parsons, 2016b), 3H:1V slopes, and heights between 14 ft to 18 ft (Dewberry and Davis, 2011) nearly 30 years ago. The Bottom Ash Pond was also constructed with a GCL along the upstream side slopes and pond interior. Since the CCR unit was constructed nearly 30 years ago, primary settlements are complete or nearly complete within the compressible foundation soils. In addition, the thickness and compressibility of the relatively thin compressible foundation soils is relatively uniform underneath the perimeter dike structure. Significant ground surface manifestations of differential settlements were not observed during operations and maintenance activities. As such, future differential settlements along the Bottom Ash Pond perimeter dikes are not anticipated and will not affect the integrity of the structural components (i.e., perimeter dikes and GCL) of the Bottom Ash Pond. In addition, as an unlined CCR unit an engineered liner is not present to be damaged if differential settlements occur.

2.5.3 Poor Foundation Conditions

The presence of low shear strength, liquefiable, and potentially sensitive soils onsite was evaluated adjacent to the Bottom Ash Pond. Extensive, continuous, or thick zones of sensitive or liquefiable soils were not identified. However, several historical and recent soil borings identified isolated and discontinuous areas of soft, lower shear strength clay soils typically immediately above the Santee Limestone. One thicker zone of low shear strength clay was identified, and a critical cross section was evaluated for slope stability within the *Bottom Ash Pond Initial Safety Assessment* (WorleyParsons, 2016a). The slope stability analysis modeled the soft clay foundation soils with an undrained shear strength (S_u) equal to 200 psf for the seismic and lower bound seismic condition. The vertical extent of the soft clay foundation soils extended from the base of the embankment to the model limits and conservatively did not consider the dense Santee Limestone stratum. A minimum factor of safety equal to 1.22 was computed, which exceeds the required factor of safety of 1.0. In addition to the conservative thickness of low or reduced strength clay, further inspection of critical slip surfaces reveals that these surfaces pass below elevations where the Santee Limestone was typically encountered and at depths traditionally not evaluated for slope stability (i.e., depths below the embankment that exceed the embankment height). The analysis indicates that the perimeter dike foundation soils are stable if founded on extensive zones or pockets of low or residual strength material with a S_u = 200 psf. However, subsurface investigations indicate that these conditions are isolated and are at depths that are not influenced by the driving forces applied by the dike structure. As such, low strength clays, sensitive soils, or liquefiable soils with low residual strengths after a seismic event are not extensive or are not located in critical areas based on available information. Thus, the Bottom Ash Pond is not founded on soils anticipated to result in a mass movement that would impair its integrity.

2.5.4 Occurrence of Karst Features

The Santee Limestone formation at CGS was evaluated extensively during construction of the generating units and byproduct management facilities. Law Engineering and Testing (1978) and Woodward-Clyde Consultants (1981) reported the presence or indication of voids within the Santee Limestone and identified that aerial photography supported the presence of voids within the stratum. In addition, Law Engineering and Testing reported the "overall site topography has a hummocky appearance with numerous circular and elongate shallow depressions throughout the site." Ponded water was not initially observed in these depressions; however, Law Engineering and Testing noted the water presence subsequent to increased rainfall. Circular and elongate shallow depressions with a saucer-shape were observed at CGS and direct evidence of soil raveling into voids in the underlying sediments was noted in several steep-sided, conical to elongate depressions observed.

Voids were commonly encountered in the Santee Limestone during extensive geotechnical drilling conducted at the site during initial plant design and construction. "Over 1,000 borings were drilled at the site for the geotechnical drilling program to support plant design and construction. Approximately 450 voids were encountered at approximately 400 boring locations, ranging from approximately 0.1 to 14 feet in thickness, with an average void thickness of approximately 2 feet (Garrett & Moore, 2011)."

2.5.5 Potential Development of Karst (Sinkhole) Features

The generalized lithology for CGS includes a varying thickness (up to 50 feet) of unconsolidated sediments (sands, silts and clays) overlying the Santee Limestone. This type of geologic setting is more likely to produce subsidence sinkholes through the mechanism where the cover material is relatively incohesive and permeable (unconsolidated Wicomico and Raysor Formation sediments) and individual grains of sand, silt and clay move downward in sequence to replace grains that have themselves moved downward to occupy space formerly held by the dissolved limestone.

Aquifer test data reported by Garrett & Moore (2011) indicate that the average (geometric mean) hydraulic conductivity from monitoring wells installed within Wicomico and Raysor Formations is 8.93×10^{-3} centimeters per second (cm/s) and 2.03×10^{-2} cm/s, respectively. The average hydraulic conductivity in monitoring wells installed in the Santee Limestone was reported as 1.16×10^{-3} cm/s. Hydrogeologic data also indicate that the surficial aquifer and the Santee Limestone are hydraulically connected with varying vertical hydraulic gradients (both upward and downward) that indicates a limited vertical flow component between the two aquifers. Limited vertical groundwater flow from the surficial aquifer will minimize the amount of dissolution. The horizontal gradients (0.001 to 0.002 feet/foot) and groundwater flow velocity (29.2 feet/year) reported by Garrett & Moore (2011) in the surficial aquifer (including the Santee Limestone) are also not conducive to the development of significant karst features in the Santee Limestone.

The lithology at the CGS indicates that a continuous low permeability layer (clay) is not present above the Santee Limestone and cohesion and strength of the overlying sediments controls whether the cover material subsides slowly or collapses. Therefore, cover-collapse karst features are not likely to occur at the CGS due to the lack of a continuous clay overlying the limestone and the observed hydraulic connection and similarity between the hydraulic conductivity in the unconsolidated sediments and the Santee Limestone.

Additionally, the composition of the Santee Limestone (Campbell and Coes, 2010), includes a significant amount (up to 25 percent) quartz sand and clay minerals that are not susceptible to dissolution. The presence of these non-soluble minerals will also limit the magnitude and extent of potential voids that could develop in the Santee Limestone.

Based on a review of the geologic and hydrogeologic data, the primary type of karst features that have occurred or are likely to occur at the CGS are cover-subsidence sinkholes. Although cover collapse sinkholes are possible, they are not likely based on the reviewed information and the current and historical observations of karst features at CGS.

However, to further evaluate the risk due to subsidence, due to potential subsurface void collapse, a general three-dimensional (3-D) Mine Subsidence Model solution was utilized. This model was developed by Geosyntec by extending the technical basis used in conventional two-dimensional (2-D) mine subsidence models (i.e., Attewell, 1977; Drumm et al. 1990). The analysis results are used to evaluate deformations at the ground surface caused by the collapse of the void below the ground surface.

Based on the conservative assumptions of void diameter (D) of 7.2 ft, bridging layer thickness of 18 ft, and no soil bulking, the maximum calculated subsidence at the base of the bottom ash pond is 0.8 ft and the maximum calculated strain is 0.6%.

It is noted that the assumption of no soil bulking is very conservative and that assuming a bulking factor will result in lower calculated subsidence and strain at the base of the pond. The calculated deformations and strains are relatively small and are not anticipated to have a negative impact on the performance of the bottom ash pond.

Based on the demonstration above, the Bottom Ash Pond is considered to be in compliance with the requirements of §257.64 for unstable areas.

3. CONCLUSIONS

Geosyntec has evaluated the relevant and available data associated with the Bottom Ash Pond for the purpose of determining compliance with location restrictions per 40 CFR §257.60 through §257.64. A compliance summary of the CCR Rule location restrictions and performance standard addressed in this document are provided in Table 1 below.

Table 1 Location Restriction Compliance Summary

Cross Bottom Ash Pond		Compliant?	
Regulation	**CCR Location Restriction**	**YES**	**NO**
§257.60	Placement Above Uppermost Aquifer		X
§257.61	Wetlands	X	
§257.62	Fault Areas	X	
§257.63	Seismic Impact Zones	X	
§257.64	Unstable Areas	X	

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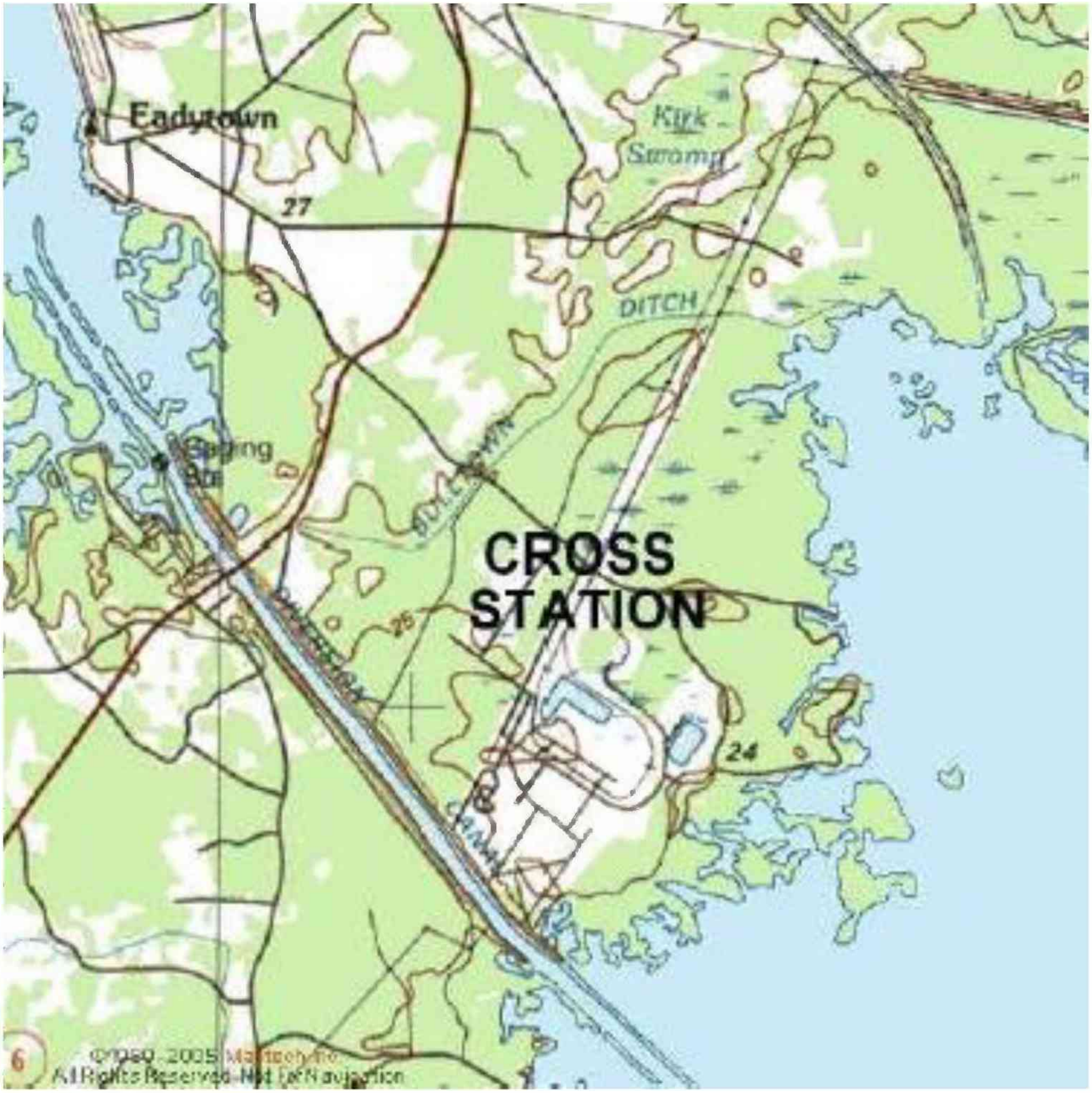
FIGURES



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Vicinity Map
Santee Cooper Cross Generating Station
Pinewood, South Carolina

Geosyntec consultants

Santee Cooper

Figure 1

Project No. GSC5242

October 2018

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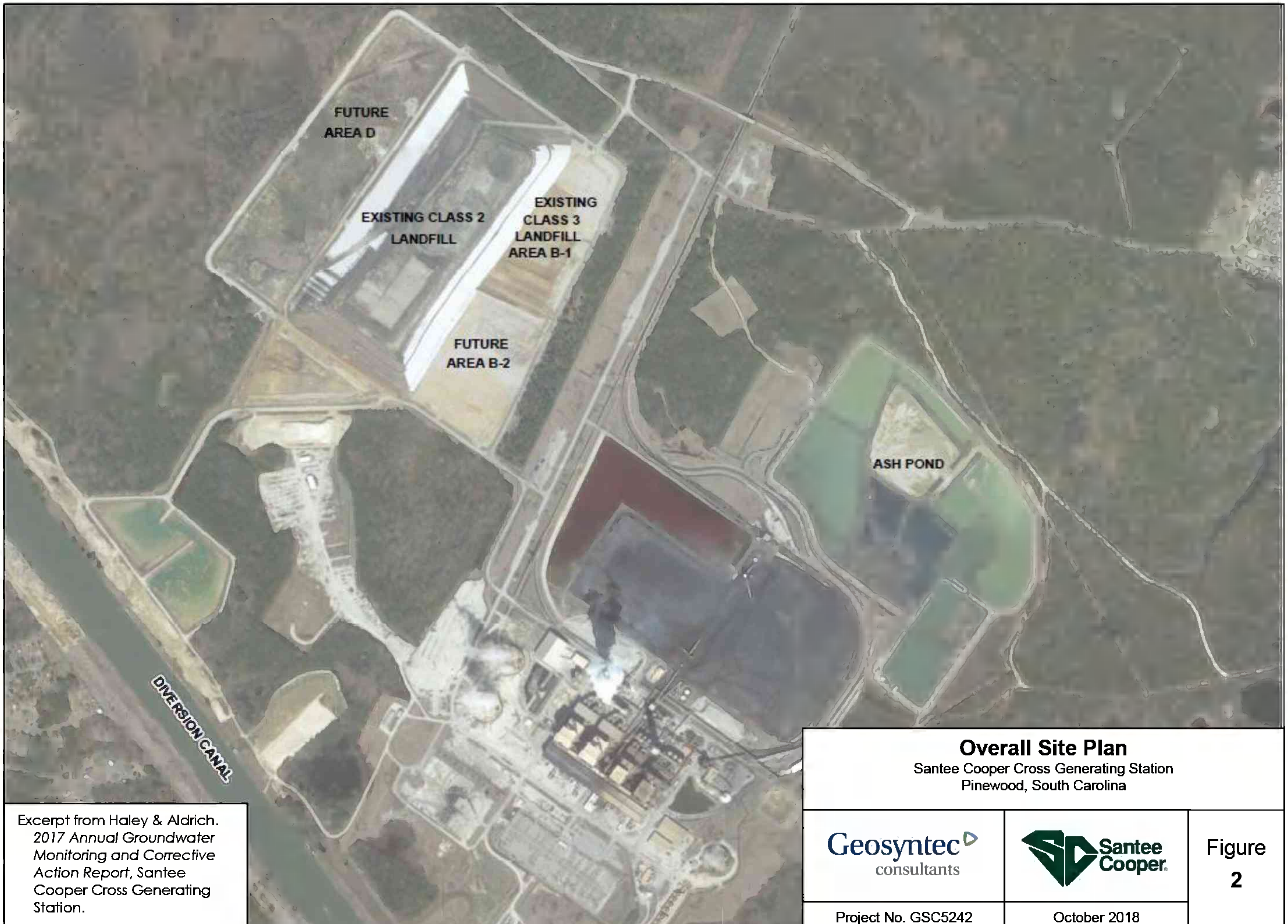

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Overall Site Plan

Santee Cooper Cross Generating Station
Pinewood, South Carolina

Geosyntec consultants	Santee Cooper	Figure 2
Project No. GSC5242	October 2018	

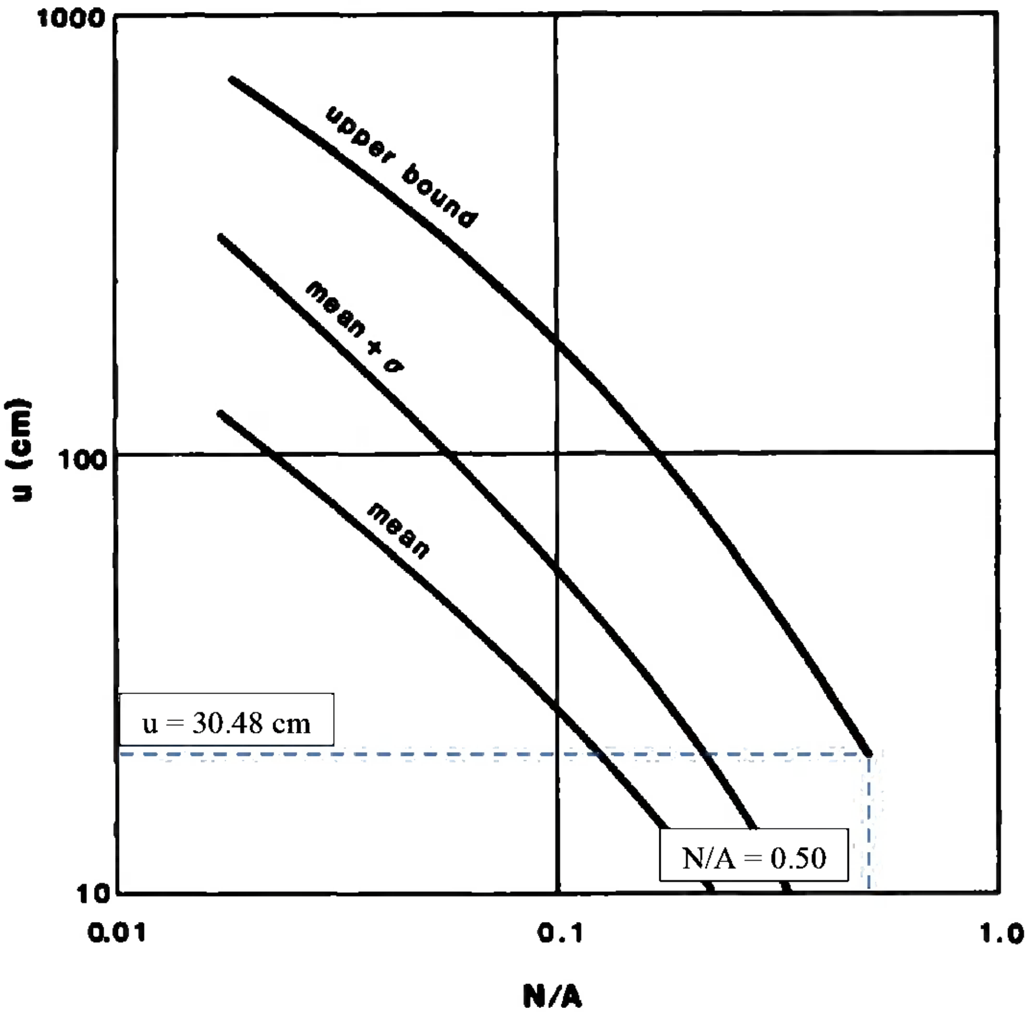

Figure 3. Allowable Displacement (u) for Pseudo-Static Slope Analysis (from Figure 7 of Hynes-Griffin and Franklin, 1984)

Notes:

1. An allowable deformation (u) of 12 inches (30.48 cm) and the "Upper Bound" curve were selected during these analyses.
2. A ratio of N/A of 0.50 was selected assuming 12 inches of displacement.

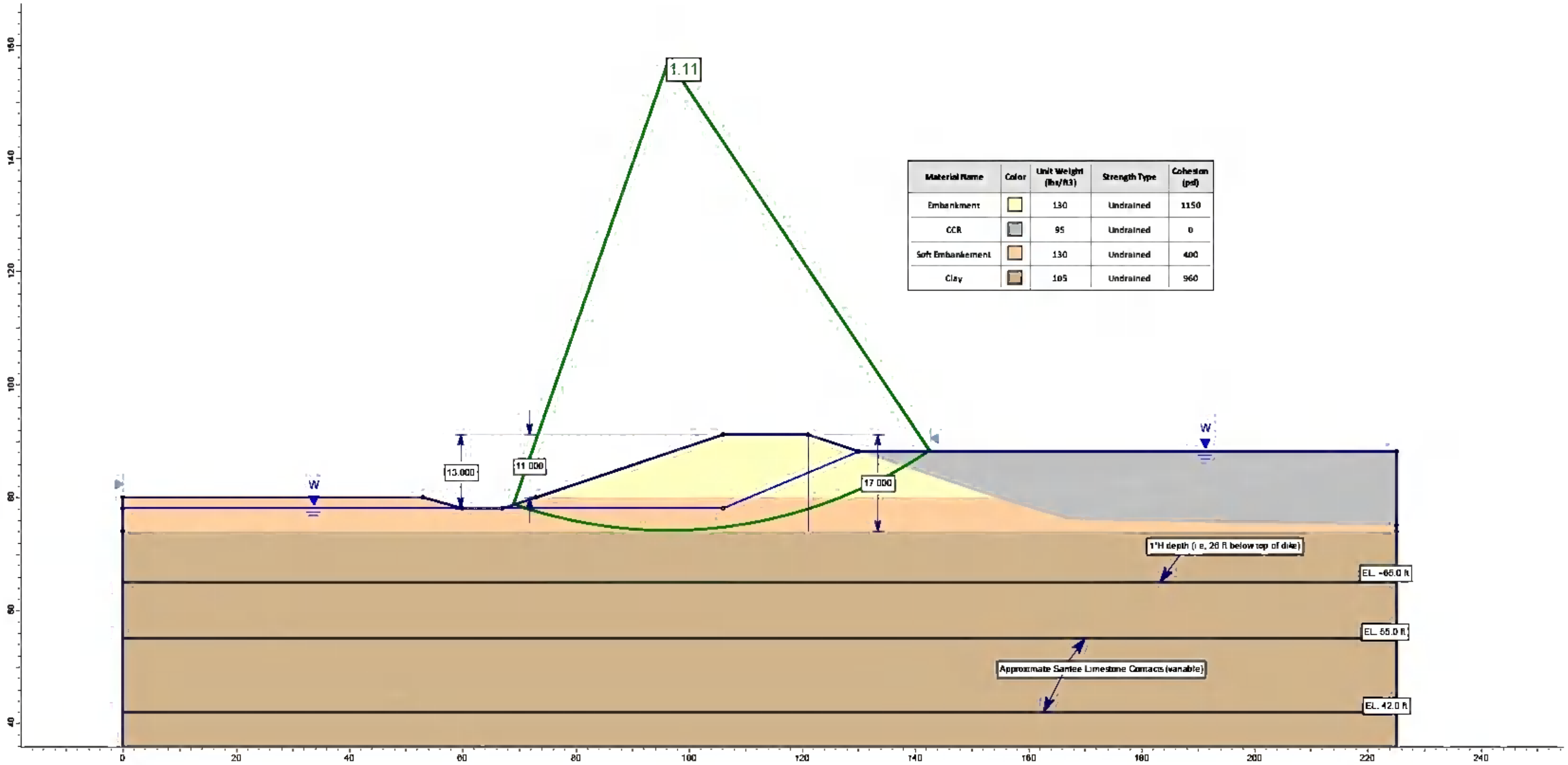

Figure 4. Pseudo-static Slope Stability Analysis for Bottom Ash Pond

Note:

Critical cross section selected and modeled from information presented within the *Bottom Ash Pond Initial Safety Factor Assessment* (WorleyParsons, 2016a).