Charleston Peninsula Coastal Storm Risk Reduction Feasibility Study (CPS)

Geology and Geotechnical Engineering

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1. INTRODUCTION

The Geology and Geotechnical report documents the subsurface conditions and the geotechnical engineering evaluation completed for the Charleston Peninsula Coastal Storm Risk Management (CHS CSRM) Feasibility Study.

1.1. Area Description

The study area is defined as the peninsula of Charleston. The structural alternative studied by the CHS CSRM was located along the edge of the peninsula, mainly in the tidal marsh areas.

1.2. Existing Data and Its Use

For the CHS CSRM Feasibility Study, no new geotechnical data were collected as part of this study due to funding and time constraints. Only existing and available geotechnical data were used. Various geotechnical reports were obtained from various engineering firms. This information was used in making design assumptions.

2. DATUMS

The horizontal and vertical datums used for the project are:

Horizontal Datum: South Carolina State Plane, North American Datum of 1983

Vertical Datum: North American Vertical Datum of 1988 (NAVD88)

3. REGIONAL GEOLOGY

A compilation of geotechnical data were sent to the study's geology and geotechnical team from various consulting agencies within the public and private sector. Over 200 Cone Penetration Tests (CPTs) and Standard Penetration Test (SPT) borings were obtained and plotted into ArcMap. Borings were analyzed for easting and northing coordinates, depth of boring, and top of Cooper Marl Formation. Data plotted in ArcMap used coordinates provided on the logs; however, if easting and northing coordinates were not present, the borings were plotted visually from the maps provided by the consulting agencies. Based on the boring data collected, the top of the Cooper Marl Formation is depicted similarly to Figure 1. The rest of the document depicts the geologic setting and stratigraphy beneath the Charleston Peninsula.

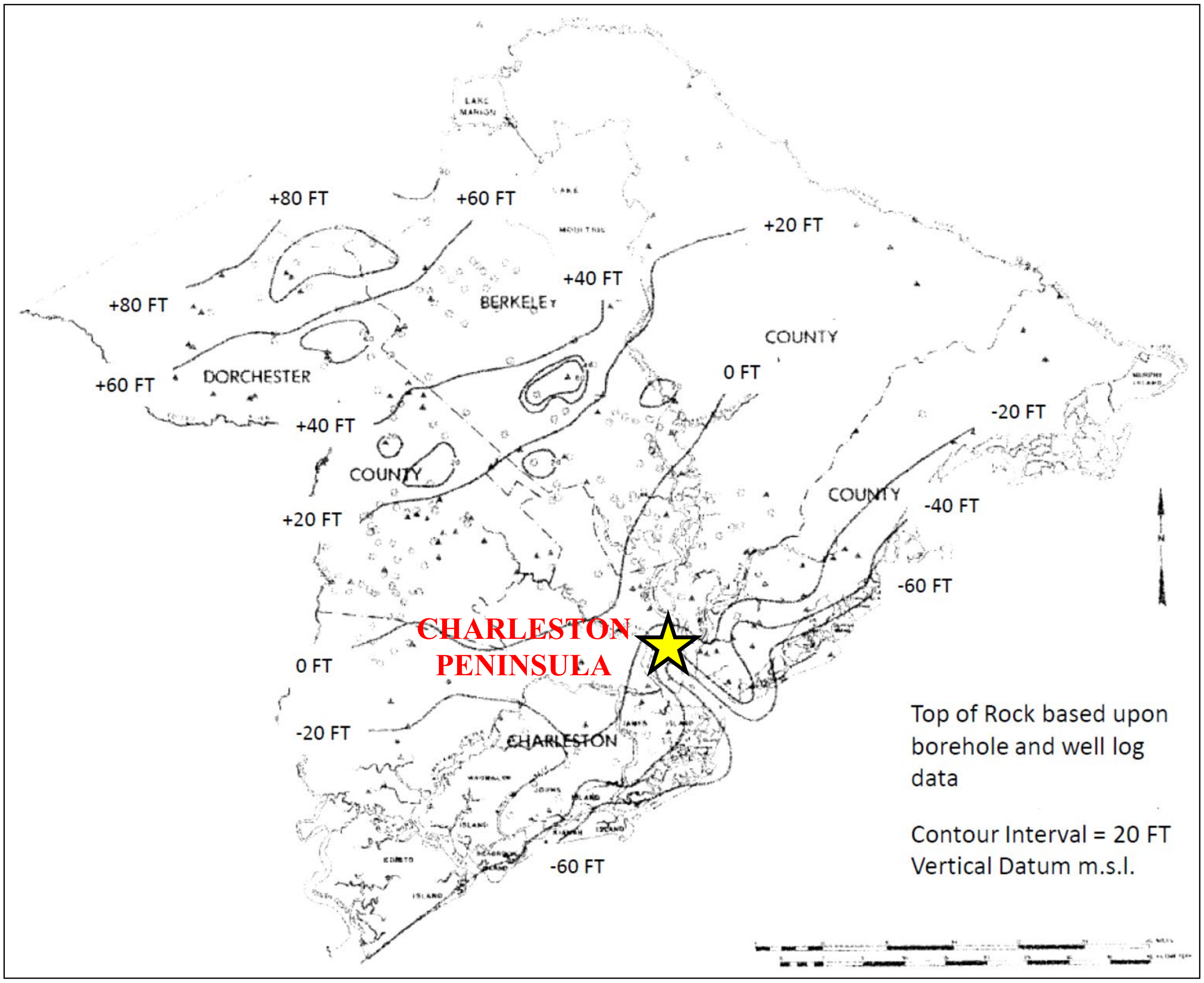


Figure 1: Structure contour map showing top of Cooper Formation, from Park (1985).

However, the term "Cooper Formation" (Toumey, 1848) is the most recognized name for the unit, and is hereby informally extended to encompass the Ashley and Chandler Bridge Formations described by Weems and Lemon (1993) and Weems and Lewis (2002). Therefore, for the purposes of this study, the term "Cooper Formation" will be used to describe the fine-grained, stiff to very stiff, low permeability strata that comprise much of the subsurface with the upper and lower harbor.

3.1. Geologic Setting

The Charleston Peninsula project site lies within the South Carolina Coastal Plain (Figure 2). Deep crustal faulting associated with Late Triassic rifting produced a subsiding depositional basin which contains Cretaceous and Tertiary sediments (Harris et al., 1979; Horton and Zullo, 1991; Harris et al., 2005). The stratigraphy of the South Carolina Coastal Plain consists of partially consolidated, unconformity bound, southeast dipping estuarine-marine shelf Tertiary deposits, which are overlain by unconsolidated Quaternary barrier and nearshore deposits. Superimposed upon this stratigraphy are escarpments and terraces that were carved into the strata as a result of interglacial sea-level fluctuation that began as early as 240,000 years ago (Weems and Lemon, 1993). The development of the modern shoreface with its barrier islands, inlets, and intertidal waters was strongly influenced by the geology and topography of resistant strata (Harris et al., 2005).

3.2. Stratigraphy

The stratigraphic units that are most significant to the project are Tertiary in age. Specifically, these units are the Black Mingo Group, Santee Limestone, Cooper Formation, Edisto Formation, and Marks Head Formation. These stratigraphic units are relevant because of their hydrogeologic properties or their occurrence within the project site (Figure 3). The units are lithologically distinct from each other and are disconformity bound. Pre-Cretaceous basement crystalline rocks and Cretaceous-age strata belonging to the Middendorf, Black Creek, and Pee Dee Formations lie at elevations of -3000 to -200 feet mean sea level (MSL), and are too deeply buried to be of engineering concern for this project. Quaternary units are generally found as surficial unconsolidated deposits along the shoreline and inland areas.

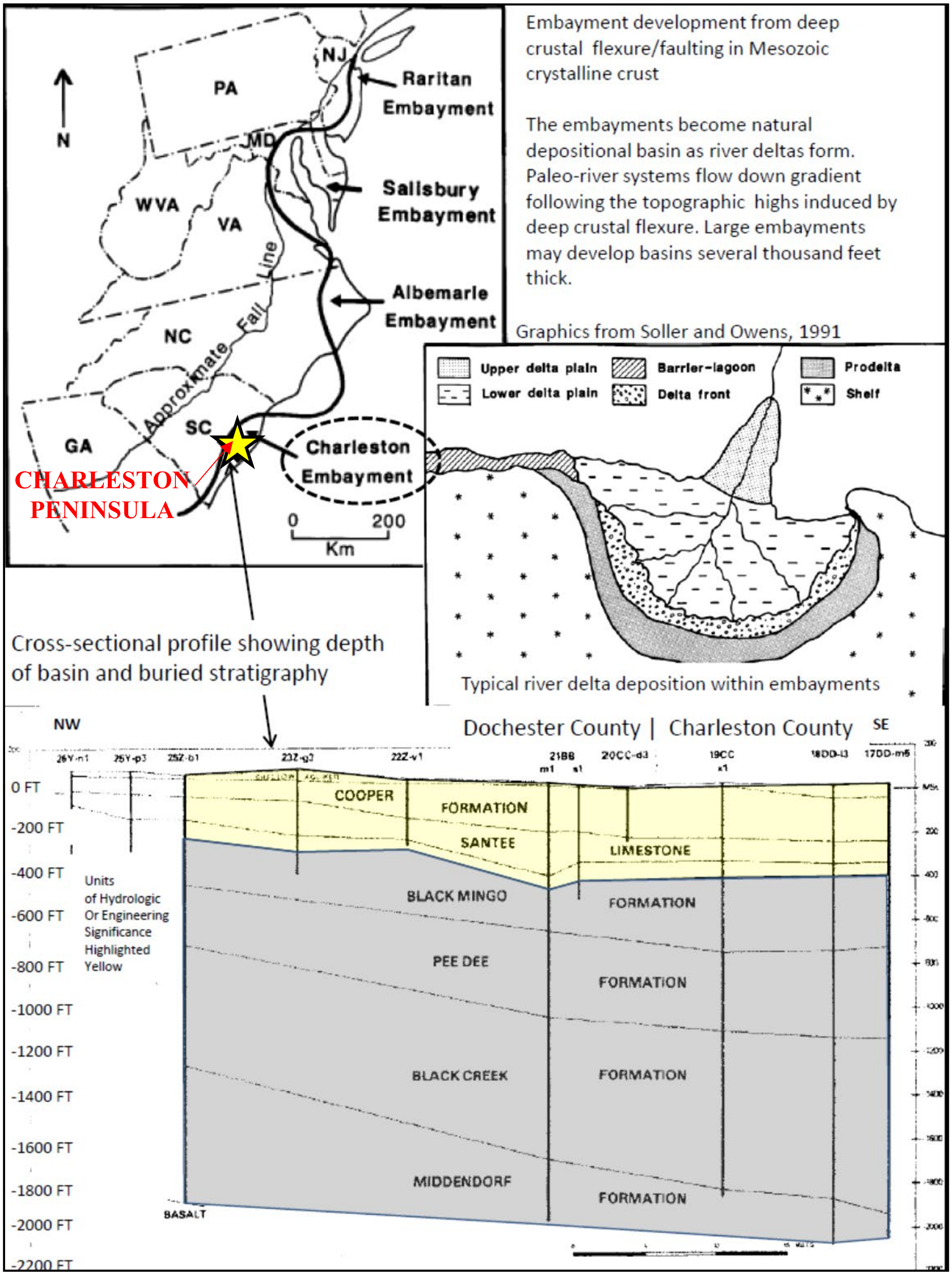


Figure 2: Regional geologic setting of the Charleston Embayment from Soller and Owens (1991) and Park (1985).

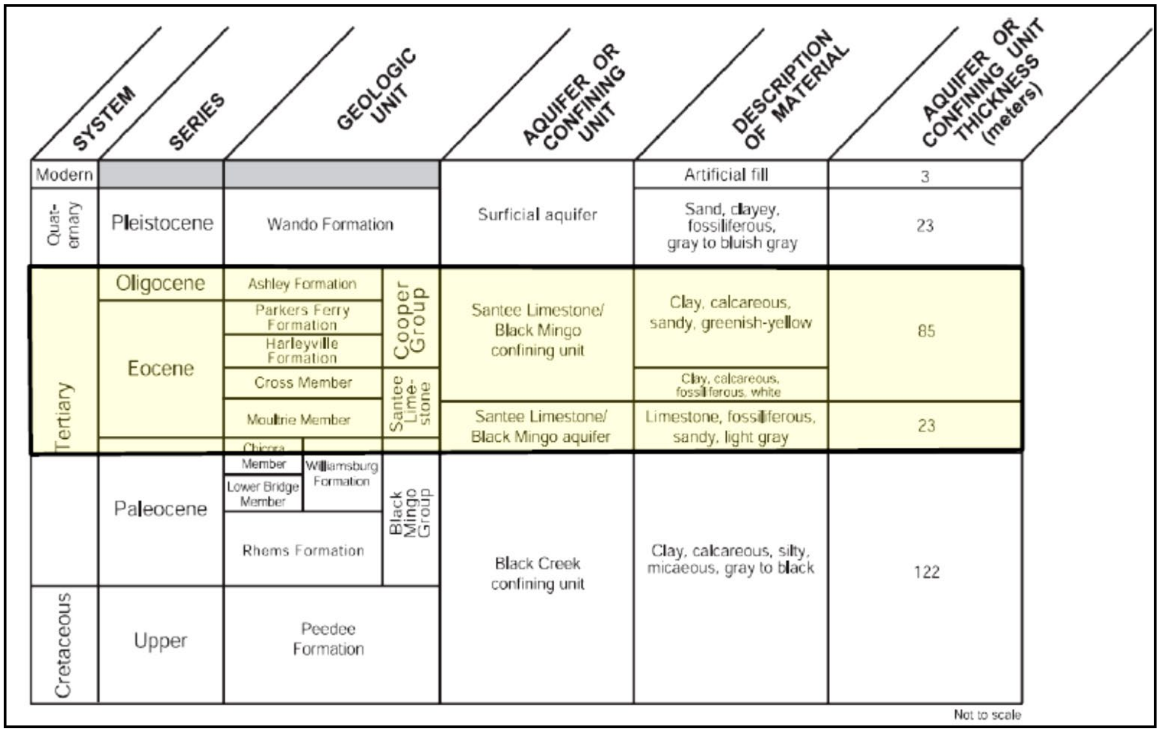


Figure 3: Project relevant stratigraphic & hydrogeologic units, from Petkewich et al. (2004).

3.2.1. Black Mingo Group

The Black Mingo Group was named for exposures of mudstone along the Black River and Black Mingo Creek by Sloan (1907). Other agency and private drill core data indicates that the unit is heterogeneous and comprised of interbedded sequences of laminated clay, mudstone, sand, and limestone. The base of the unit is predominantly composed of mudstone and silty-clay interbedded with calcareous sands with occasional limestone, whereas the top of the unit is predominantly fossiliferous limestone interbedded with quartz sand and occasional clay (Bybell et al., 1998; Edwards et al., 1999). The Black Mingo sediments are generally a mixture of clastic detrital material and volcanic ash that were deposited within inner shelf and marginal marine environments during the Late Paleocene to Early Eocene. Outcroppings of the formation occur in Monck's Corner and surrounding counties, and it dips south-southwest into the subsurface to a depth of -600 feet MSL below southern Charleston County (Park, 1985).

3.2.2. Santee Limestone Formation

The Santee Limestone is named for exposures that occur along the Santee River in South Carolina where it underlies the Cooper Group (Sloan, 1908). The Santee Limestone is creamy-white to gray, fossiliferous, glauconitic, and has sand to mud-supported matrix. The unit is middle to late Eocene in age and disconformity bound (Park, 1985). Two members are generally recognized within the Santee Limestone; the middle Eocene Moultrie Member and middle to late Eocene Cross Member (Figure 3). The Moultrie Member of the Santee Limestone is approximately 7-feet thick (from recovered drill cores) and the limestone matrix tends to be coarse-grained, bioturbated, moldic, and sandy. The Cross Member is significantly thicker (39-

feet thick from drill core) with a finer-grained, clayey matrix. Deposition of the Santee Limestone occurred 45-41 million years before present, when shallow open marine-shelf environments were drowned and transformed into deeper outer continental shelf environments (Petkewich et al., 2004). The Santee Limestone is exposed in surficial exposures located along a 5-mile wide belt that extends across northern Dorchester, Berkeley, and Charleston Counties, and it dips into the subsurface towards the south-southeast (Figure 4). The top of the formation lies at elevation -300 feet MSL beneath Charleston Harbor. The unit thickens southwestward from 20-feet thick near Lake Moultrie to over 260-feet thick beneath Edisto Island (Park, 1985).

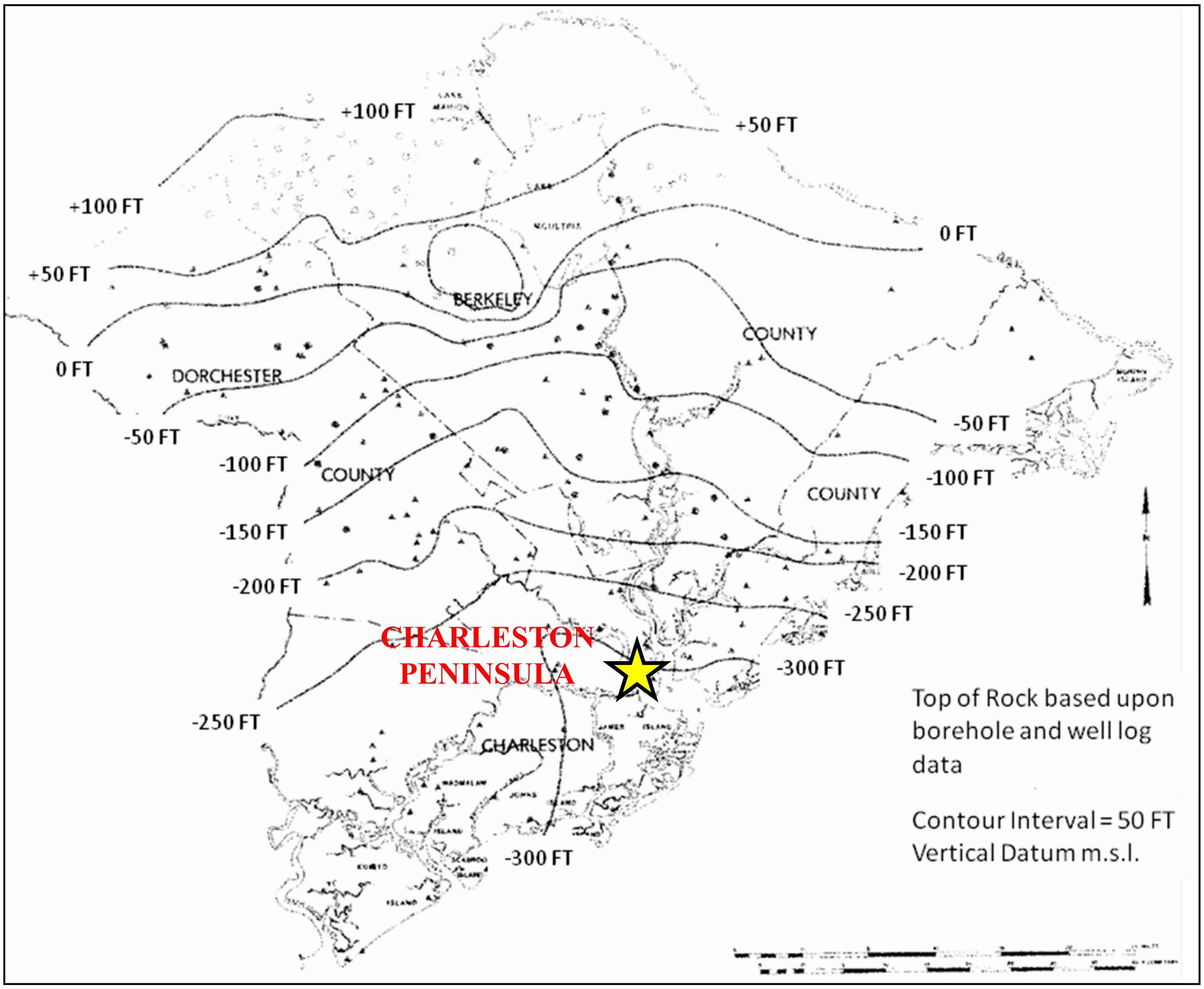


Figure 4: Structural contour map showing top of Santee Limestone, from Park (1985).

3.2.3. Cooper Formation

The Cooper Formation was originally termed "Cooper Marl" by Toumey (1848) for exposures of soft, very fine-grained, impure carbonate material found along the Cooper River and Ashley Rivers in South Carolina. This unit has been described by various workers in surficial exposures within the coastal plains of North Carolina, South Carolina, and Georgia (Toumey, 1848; Cooke, 1952; Malde, 1959; Weems and Lemmon, 1993; Weems and Lewis, 2002). Carbonate-rich sections of the unit were extensively studied and served as a source for agricultural lime production between 1867 and 1920. Upland exposures of the Cooper Formation are described as consisting of fine-grained calcareous foraminiferal shell material (Malde, 1959; Gohn et al., 1977; Park, 1985). In contrast, soil borings, grab samples, and surficial exposures of the Cooper Formation within Charleston Harbor resemble a consolidated and low permeability soil that ranges in composition from stiff clayey silt to dense silty sand. Weems and Lemon (1993) indicated that the Cooper Formation (Toumey, 1848) actually consists of a composite sequence of variably consolidated silt and clay, soft clayey and sandy limestones, and phosphatic deposits of Eocene-Oligocene age (Park, 1985; Weems and Lemon, 1993).

Structural contour maps indicate that the Cooper Formation dips into the subsurface toward the south-southeast at a gradient of 8 feet per mile (Figure 1). Beneath the city of Charleston, the top of the Cooper Group lies at an elevation of -20 feet MSL, but due to the dipping gradient and high subsurface relief, it plunges to a depth of -60 feet MSL near the mouth of the harbor. Parks (1985) determined that the stratum thickens to 280 feet beneath Charleston Harbor (Figure 5).

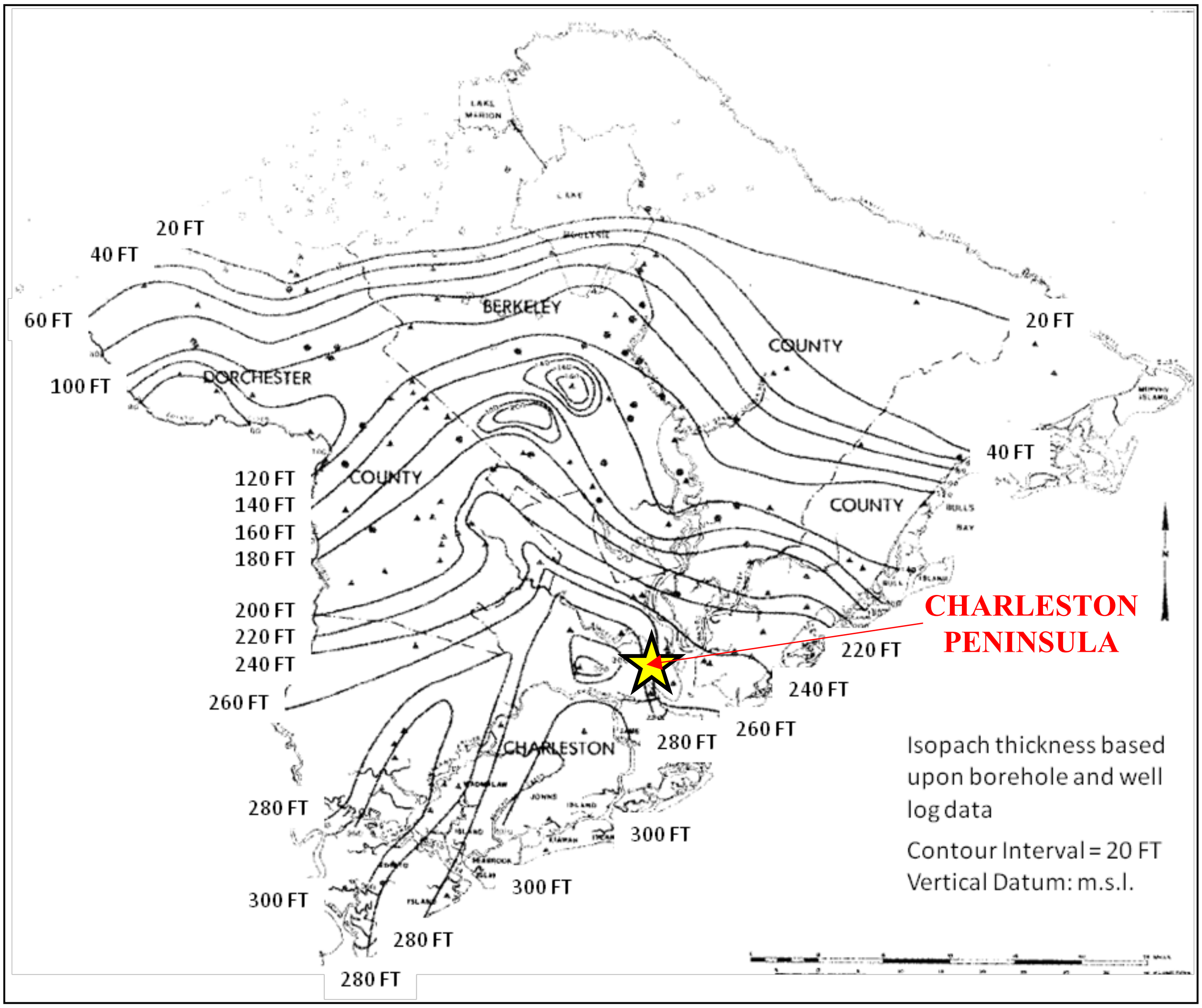


Figure 5: Isopach map showing thickness of the Cooper Formation, from Park (1985).

South Carolina Department of Natural Resources (SCDNR) describes the unit as a stiff, partially consolidated, calcareous silty-clay (SCDNR, Doars, personal communication, 2012). U.S. Army

Corps of Engineers (USACE) drilling logs that penetrate into the Cooper Group describe the soil as a stiff to very stiff or hard, brown to greenish-colored, clayey, inorganic silt to silty clay, which had been classified as (MH, CH, ML, MH-CH, and ML-CL) per ASTM D2487. This material appears to grade into and out of medium dense clayey sand and stiff to hard lean clay. Brainard et al. (2009) states that, historically, tunnel construction in Charleston area was conducted within the Cooper Formation (Cooper Marl) because of the unit's optimal engineering characteristics of low permeability, stiffness, and the relative ease by which it can be excavated. However, several water-bearing sand lenses 30-feet thick have been encountered during tunnel excavation (Brainard et al., 2009).

The Cooper Formation is comprised of at least four major subunits: the Eocene Harleyville and Parkers Ferry Formations, and the upper Oligocene Ashley and Chandler Bridge Formations. Collectively, these units were deposited in shallow to open marine environment 30 to 38 million years ago. The strata range in composition from phosphatic clay, to sandy limestone, to fine-grained, silty-clayey, phosphatic sand (Ward et al., 1979; Weems and Lemon, 1984; Weems and Lemon, 1993). Harris et al. (2005) verified the top of the Cooper Formation at elevation -60 feet MSL by seismic profile in the vicinity of Folly Island (Figure 6).

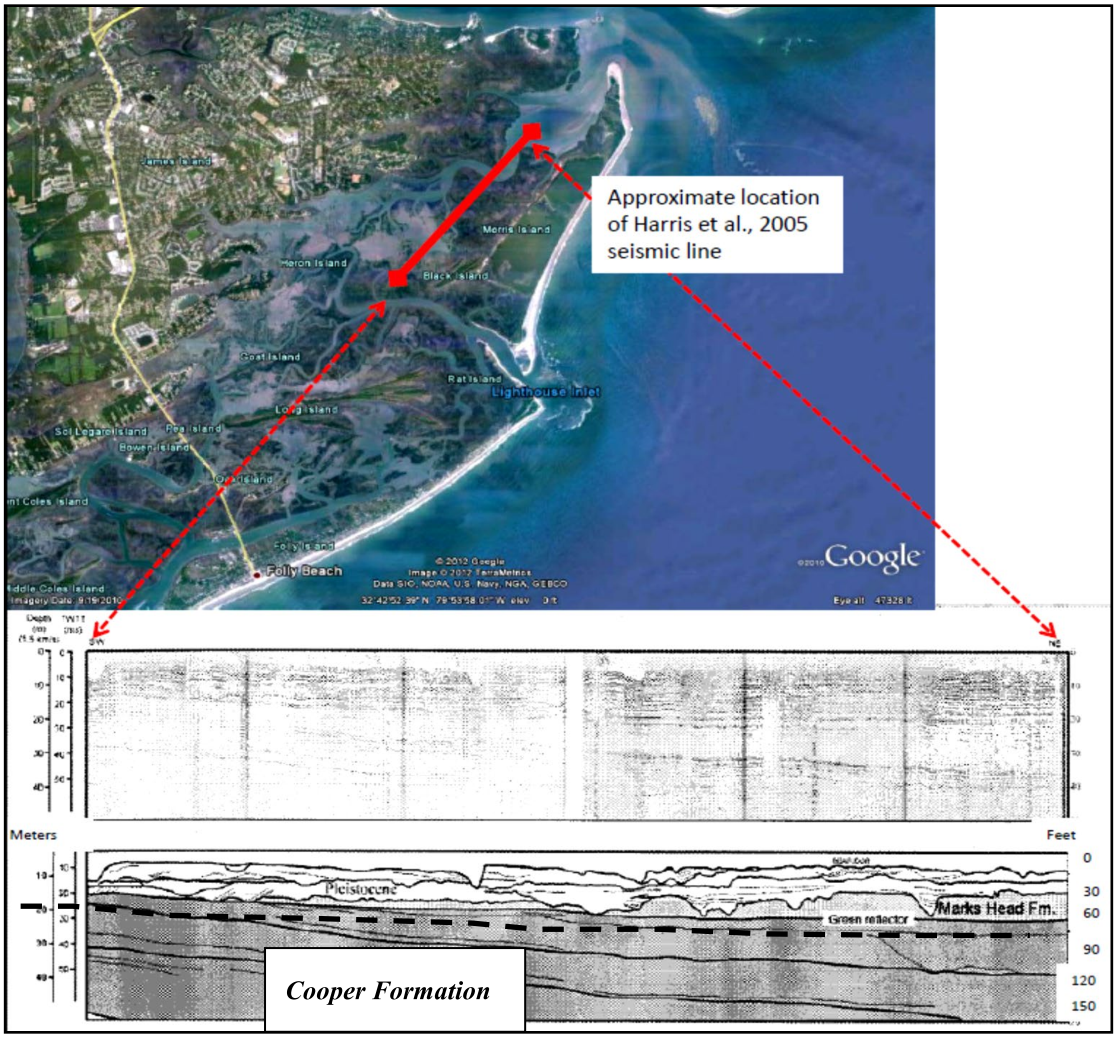


Figure 6: Seismic profile south of Charleston Harbor, from Harris et al. (2005).

3.2.4. Edisto Formation

Ward et al. (1979) applied the name "Edisto Formation" to sandy-shelly limestones of early Miocene age that unconformably overlie the Cooper Formation in southern South Carolina. Weems and Lemon (1993) describe the unit as consisting of light gray, fine-grained, calcareous sand to quartzose calcarenite[1] with locally abundant pelecypod shells. The Edisto Formation is generally composed of detrital, weakly-cemented sand, gravel, and shell hash. The unit was deposited in a shallow marine environment 24 million years ago during the Miocene-Oligocene. Weems and Lemon (1993) report the occurrence of phosphate nodules in land borings, but not in offshore borings. The Edisto Formation unconformably overlies the Cooper Formation within the

[1] Calcarenite is a type of limestone that is composed predominantly (> 50 percent) of detrital (transported) sand-size (0.0625 to 2 mm in diameter), carbonate grains. This material is derived from corals, shells, fragments of older limestones, and other carbonate clasts. Calcarenite is the carbonate equivalent of a sandstone. They can consist of grains of carbonate that have accumulated either as coastal sand dunes (eolianites), beaches, offshore bars and shoals, turbidites, or other depositional settings. Reference: http://en.wikipedia.org/wiki/Calcarenite

study area; however, the stratigraphic contact was not observed in drill core and the thickness of the unit is unknown.

3.2.5. Marks Head Formation

The Marks Head Formation is described as fine-grained, quartz-phosphate sand that is Miocene-aged. The unit is known to lie unconformably atop the Cooper Formation and was deposited in shallow-brackish water conditions. Weems and Lemon (1993) indicate that the unit is discontinuous and only occurs in the near subsurface northeast of Charleston, beneath Mount Pleasant and Sullivan Island. South of Charleston, the unit is present from -30 to -60 feet MSL and is no more than 30-feet thick (Harris et al., 2005). The Marks Head Formation dips into the subsurface south and east from surficial outcroppings north of Charleston (Weems and Lewis, 2002). The base of the unit is present at elevations -20 to -80 feet MSL near Charleston Harbor. The shallowest occurrence of this stratum is likely to occur within the Ashley River near Duck Island and north of the confluence of the Cooper and Wando Rivers.

3.2.6. Undifferentiated Quaternary Units

Nearly all of the surficial deposits in the Charleston area are Quaternary in age, and they unconformably overlie the Tertiary strata. These sediments were deposited during sea-level fluctuations caused by multiple interglacial cycles throughout the Pleistocene. Based upon the presence of Pleistocene-aged terrace deposits and erosional shoreline escarpments, at least five different sea-level stands are recognized near Charleston. These geomorphologic features lie as far as 45-miles inland and mimic the shape of the modern coastline (Weems and Lemon, 1993; Harris et al., 2005). The Quaternary age strata generally consist of interbedded sequences of clay, clayey to clean quartz sand, and fossiliferous sand that may be capped by peat, clean sand, or tidal marsh deposits (Weems and Lemon, 1993).

4. GROUNDWATER

Groundwater levels are relatively shallow within the Charleston Peninsula and will fluctuate with the tides, seasons, and precipitation. The CSRM features will be located along the exterior of the peninsula and the groundwater levels will be highly dependent on the tides. It should be anticipated that the groundwater table would be encountered at or near the elevation of the tide elevation. This relatively shallow groundwater table will likely require some dewatering during construction of the T-wall foundations. Steel and concrete elements will need to consider this in respect to corrosion.

5. SEISMICITY

The Charleston Peninsula is located in a "hot spot" of high seismic activity and is deem to be within a high seismic hazard zone as indicated in Figure 7. This area is known as the Charleston Seismic Zone. Additionally, Charleston, SC is also the site of the largest earthquake known to have occurred in the southeastern United States, which occurred on 1886.

A seismic evaluation was completed as part of the feasibility study and the details are presented in ATTACHMENT 1.

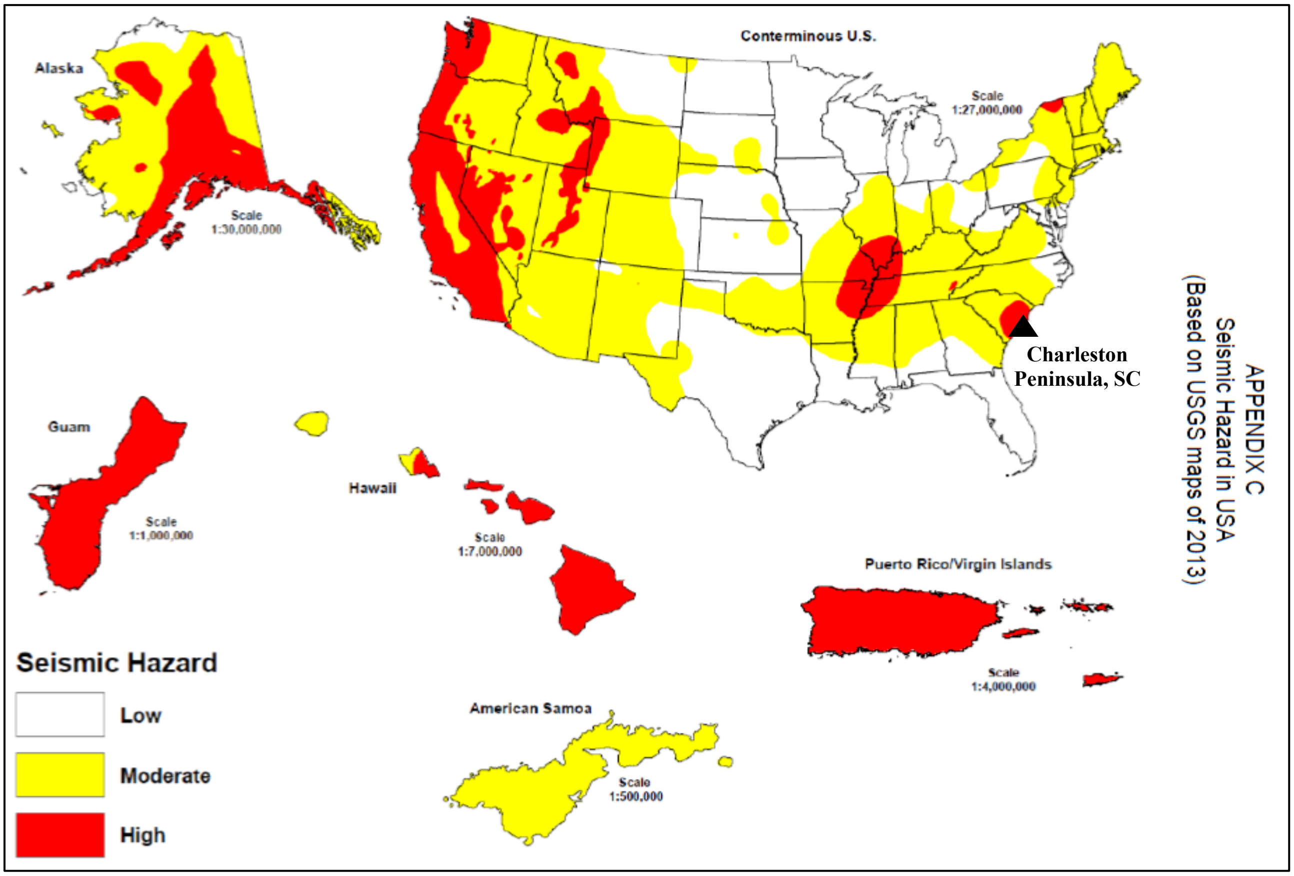


Figure 7: Project location shown on seismic hazard map of the USA, from ER-1110-2-1806.

5.1. Ground Motions

The seismic evaluate provided a range of ground motions for various events. A earthquake with a 2% probability of exceedance in 50 years could produce a PGA that ranges from 0.6 to 0.8g near the Charleston Peninsula [USGS 2014 seismic hazard map by Petersen et al. (2015)], shown in Figure 8. The site-predicted PGA for an earthquake having a return period of 2,475 years is approximately 0.973g, which is slightly higher than the USGS seismic hazard map shown in Figure 8. Spectral ground motion on the Charleston Peninsula was also predicted by the Uniform Hazard Response Spectrum (Figure 9). Based upon probabilistic hazard mapping, the PGA at the site is predicted to be 0.8561g, but the largest and most likely damaging ground motion is 1.3972g at a spectral period of 0.2 seconds (Figure 9).

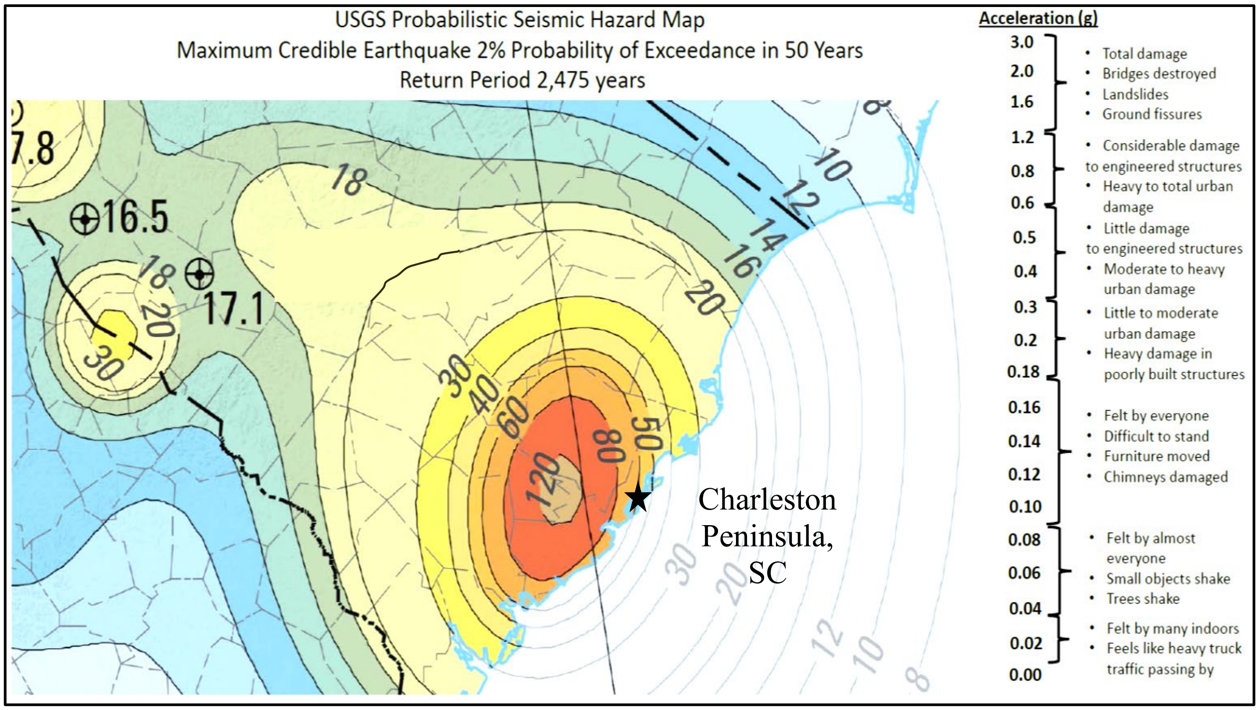


Figure 8: USGS Seismic Hazard Map, PGA, 2% Probability of Exceedance in 50 Years, from Peterson et al. (2015).

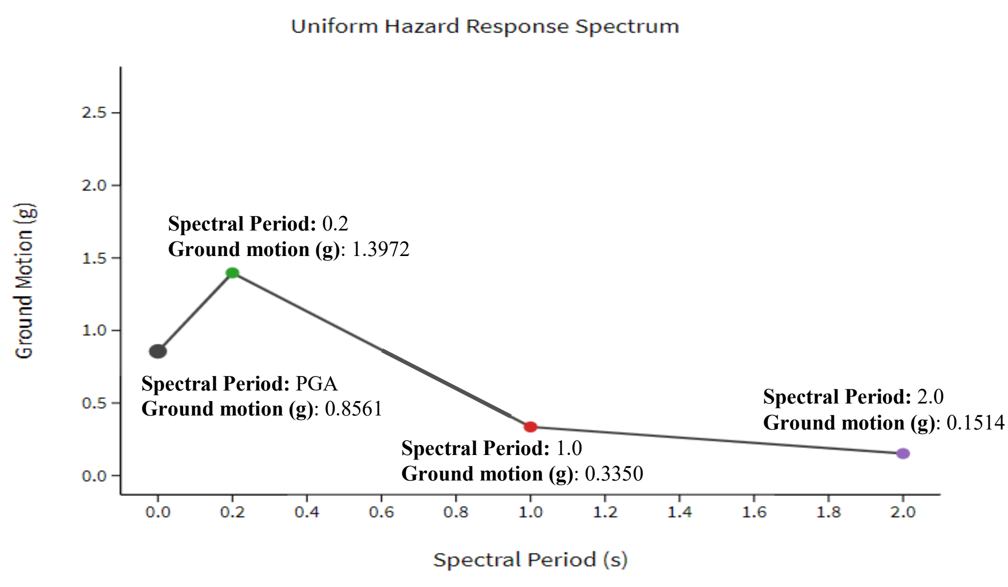


Figure 9: Uniform Hazard Response Spectrum predicted for the project site showing PGA with 2% in 50 years AEP (2,475 return period).

5.2. Maximum Credible Earthquake and an Operating Basis Earthquake

The Maximum Credible Earthquake (MCE) were deterministically derived. The MCE was determined to be an Mw = 7.3 and based upon the 1886 Charleston Earthquake event. The distance from the project site to the center of the MCE source zone is 10.00 km.

The Operating Basis Earthquake (OBE) was assessed using probabilistic methods that are informed by deterministic methods. An OBE PGA of 0.0548g and a SA of 0.09g (at 0.2 second period) is derived utilizing the USGS Unified Hazard Tool.

6. EXISTING FLOOD RISK MANAGEMENT STRUCTURES

The City of Charleston has two floodwalls, the Low Battery and High Battery Walls, which are located on the west and south sides of the peninsula, respectively. Both the Low and High Battery Walls have had their conditions assessed and recommendation for improvements developed. At the time of the feasibility study, the City of Charleston was pursuing a project to modify the Low Battery Wall and raise it to EL. 9 feet, which is the height of the High Battery Wall.

Due to the age and condition of the High Battery Wall, it was assumed that it would not meet USACE standards for design and performance and therefore a new floodwall would be required to be constructed as part of the CSRM project.

7. STUDY STRUCTURAL MEASURES

7.1. General

There were various structural measures considered during the CSRM study that fall into two main categories, levees and floodwalls.

7.2. Levees

Levees, including road raises and other earthen berms, were initially discussed as potential features but were not carried forward due to the larger footprint required by these features over that of floodwalls. Additionally, road raises create issues as ramps would be required to maintain access to connecting streets. The footprint of the access ramps would have an impact on adjacent properties.

7.3. Floodwalls

Multiple types were considered that included I-wall, double row sheetpile, combo wall, and concrete T-wall.

I-walls were initially considered in areas with exposed stem heights less than 4 to 6 feet. But due to the soft condition of the soils along with lack of any specific geotechnical data, I-walls were not carried forward in the study and replaced with a pile-founded concrete T-wall. A double row sheetpile wall that was tied together was also initially considered along reaches adjacent to and within the tidal marsh area. This concept included placing fill within the double row sheetpile wall to allow for a walking path on top of the structure. The double row sheetpile was not carried forward as placement of fill material would be problematic due to the unconsolidated nature of the tidal marsh material and the compressibility of the foundation material which could lead to excess settlement and drawdrag on the sheetpiles.

T-walls and combo types were selected and used in the feasibility project. The T-Wall concept used was based on a typical design that included piles (both vertical and battered) for structural support and a sheetpile cutoff as part of seepage mitigation. T-walls were placed in locations where the alignment was on ground.

The combo wall concept is a wall comprised of large circular piles (also known as king piles) and batter piles, the later to provide additional lateral resistance. Sheetpile placed in-between the circular piles to provide a continuous structural wall. These components are tied together with a concrete cap. The combo walls were placed in the tidal marsh areas.

8. GEOTECHNICAL ASPECTS OF FEASIBILITY STUDY MEASURES

The geotechnical aspects of the various feasibility study measures are discussed below. Due to the study area size, schedule and funding constraints, the geotechnical design is conceptual. It was developed based on assumptions made using information found within other CSRM project studies (Norfolk, Virginia and Galveston, Texas) and local geotechnical reports, along with engineering judgment. The geotechnical design is at a 10% conceptual level. Discussion are included on what future work is required during the Pre-construction Engineering and Design (PED) phase.

8.1. T-Wall

The T-wall will be pile founded using both vertical and battered piles. A steel sheetpile cutoff will was assumed to be installed to reduce underseepage and uplift on wall. It was assumed that the sheetpile would be 15 feet long for the EL. 7 and EL. 9 walls and 20 feet long for the EL. 12 wall.

8.2. Combo Wall

The king piles and battered piles for the Combo wall will be founded within the Cooper Marl formation. The steel sheetpile between the king piles will be installed to reduce underseepage. It was assumed that the sheetpile would be 35 feet long for the EL. 7 and EL. 9 walls and 40 feet long for the EL. 12 wall.

8.3. Piles

Many structures on the peninsula are founded on piles. Review of various engineering reports received, the typical type was either steel H-piles or square, pre-stressed concrete piles, either 12” or 14” in size. These piles are driven to bear within the Cooper Marl formation and it was assumed the embedment depth was 5 feet in the Copper Marl. The top of Cooper Marl beneath the proposed alignment is presented below in Figure 10. Additional maps can be found in Attachment 2.

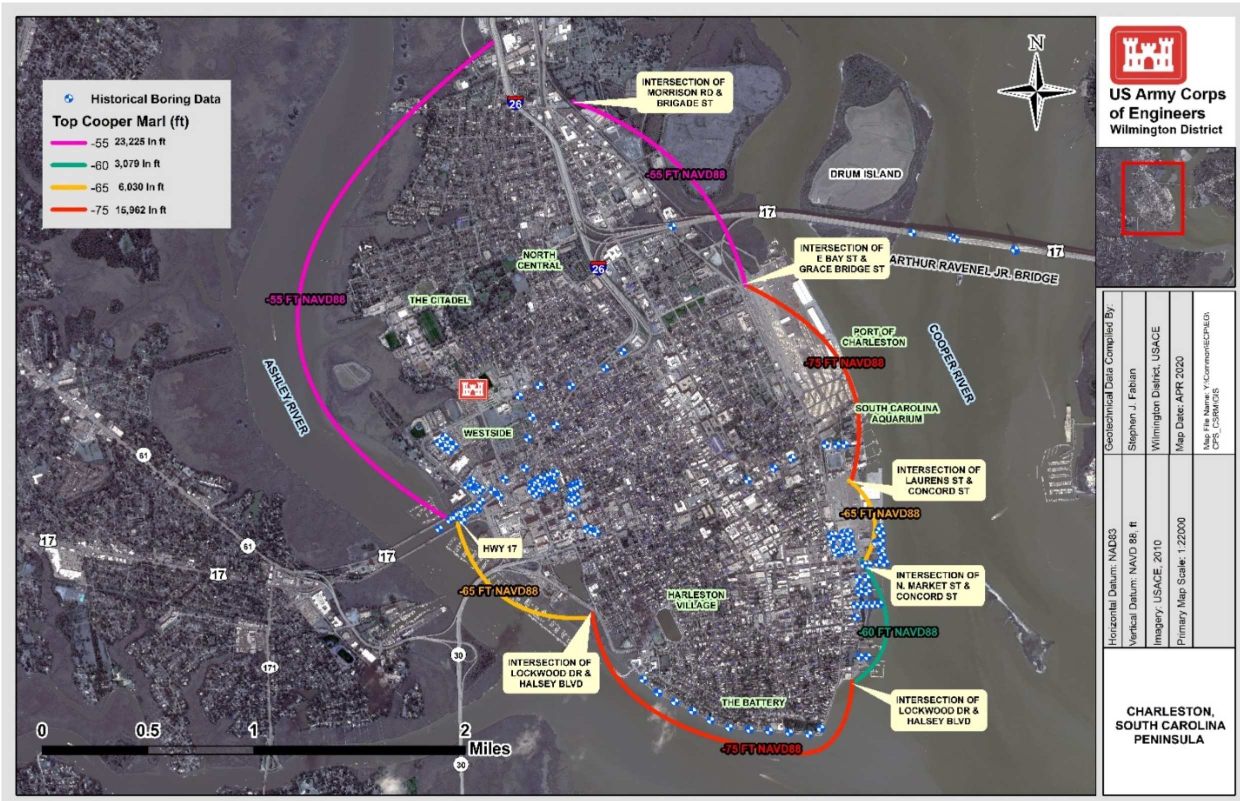


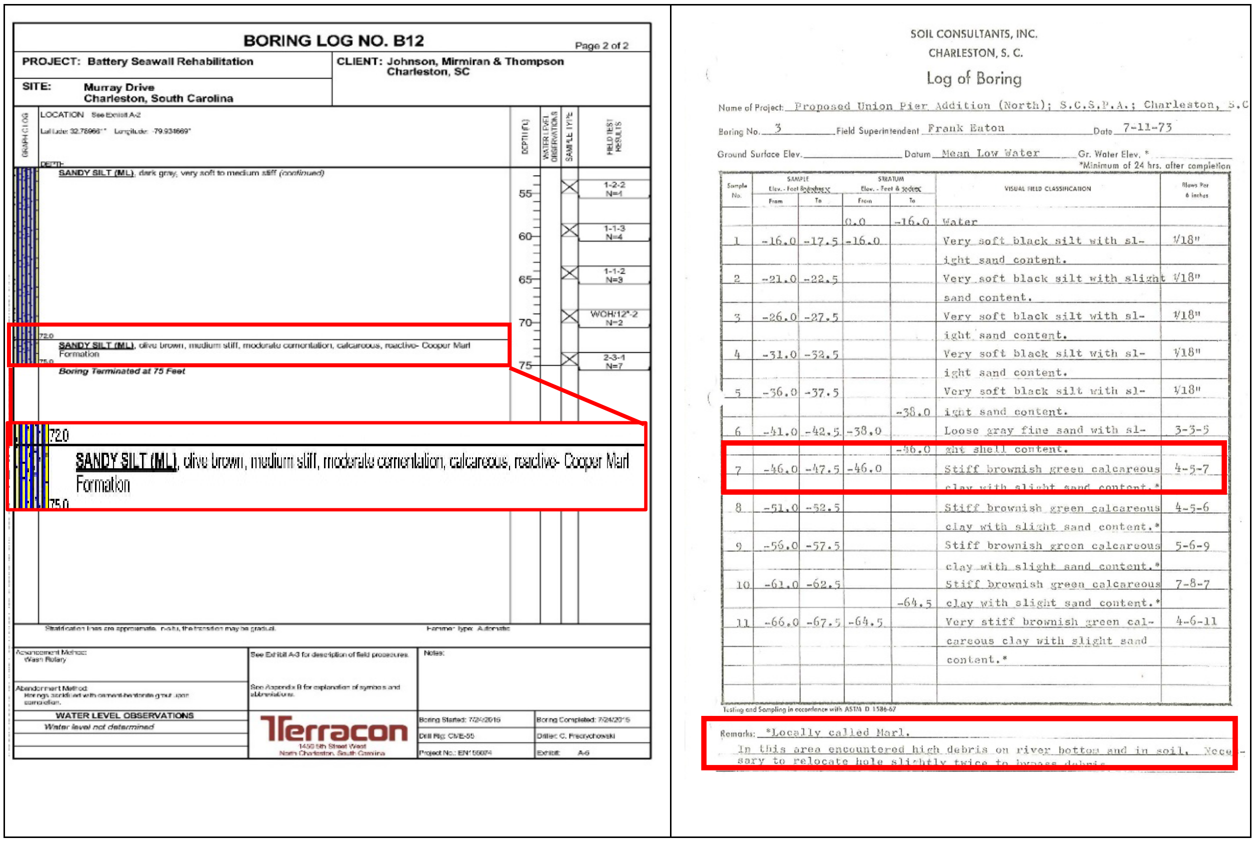
Figure 10: Top of Cooper Marl around the Charleston Peninsula

8.3.1. Determination of Top of Cooper Marl

The top of Cooper Marl was defined by taking a compilation of scientific literature and collection of over 200 SPTs and CPTs from 1977 to 2018. This geotechnical data was taken by several contractors and were not USACE affiliated projects. Prior to depicting the top of Cooper Marl, SPTs and CPTs were loaded into ArcGIS via "Go To XY" feature to locate each SPT and CPT. Following the input of each geotechnical point, a field was created in the attribute table called: "Top of Cooper Marl." The CPT data looked for differences in resistivity along stratigraphic boundaries indicating high plasticity silt and/or clay. Most of the CPTs taken were followed by a SPT in the same CPT location in order to ground truth the CPT collected. Majority of the SPTs drilled noted in the core descriptions specifically the top of the Cooper Marl (Figure 11). This was the primary way to delineate the top of Cooper Marl across the peninsula. Each elevation was inputted into the attribute table in ArcMap. Once this was done, the labels were turned "ON" and the elevations shown were used to mark the top of Cooper Marl in 5-foot intervals around the peninsula (Figure 10).

Moreover, in Figure 1, the Cooper Marl is shown dipping to the south with the top of the Copper Marl at EL. -60 feet underneath the Charleston Peninsula. The SPT data across the peninsula shows the top of Copper Marl ranging from EL. -55 to -75 feet with the top of the formation dipping from north to south. This coincides with Figure 1 showing this behavior. In addition, Figure 11, shows the southerly dipping trend in the top of the Copper Marl. The Port of Charleston which is located north of "The Battery" has the top of the Cooper Marl shallower than "The Battery." This is seen throughout the peninsula.

Combining the literature review and geotechnical data there is high confidence that the top of the Cooper Marl ranges from EL. -55 toEL. -75 feet across the peninsula. However, because of the data gaps along the outer edges of the peninsula the top of the Cooper Marl ranges drastically from one area to another. In order to achieve a better understanding additional exploratory SPTs and CPTs would need to be performed to delineate the top of Copper Marl more accurately.



BORING LOG NO. B12 — Page 2 of 2

PROJECT: Battery Seawall Rehabilitation — CLIENT: Johnson, Mirmiran & Thompson, Charleston, SC

SITE: Murray Drive, Charleston, South Carolina

LOCATION See Exhibit A-2 — Latitude: 32.76966° Longitude: -79.934669°

Depth (ft)	Description	Field Test Results
	SANDY SILT (ML), dark gray, very soft to medium stiff *(continued)*	
55		1-2-2 N=4
60		1-1-3 N=4
65		1-1-2 N=3
70		WOH/12"-2 N=2
72.0	SANDY SILT (ML), olive brown, medium stiff, moderate cementation, calcareous, reactive- Cooper Marl Formation	
75.0		2-3-4 N=7
	Boring Terminated at 75 Feet	

SOIL CONSULTANTS, INC.
CHARLESTON, S. C.
Log of Boring

Name of Project: Proposed Union Pier Addition (North); S.C.S.P.A.; Charleston, S.C

Boring No. 3 — Field Superintendent Frank Eaton — Date 7-11-73

Ground Surface Elev. — Datum Mean Low Water — Gr. Water Elev. * — *Minimum of 24 hrs. after completion

Sample No.	Sample Elev. - Feet From	Sample Elev. - Feet To	Stratum Elev. - Feet From	Stratum Elev. - Feet To	Visual Field Classification	Blows Per 6 inches
			0.0	-16.0	Water	
1	-16.0	-17.5	-16.0		Very soft black silt with slight sand content.	1/18"
2	-21.0	-22.5			Very soft black silt with slight sand content.	1/18"
3	-26.0	-27.5			Very soft black silt with slight sand content.	1/18"
4	-31.0	-32.5			Very soft black silt with slight sand content.	1/18"
5	-36.0	-37.5		-38.0	Very soft black silt with slight sand content.	1/18"
6	-41.0	-42.5	-38.0	-46.0	Loose gray fine sand with slight shell content.	3-3-5
7	-46.0	-47.5	-46.0		Stiff brownish green calcareous clay with slight sand content.*	4-5-7
8	-51.0	-52.5			Stiff brownish green calcareous clay with slight sand content.*	4-5-6
9	-56.0	-57.5			Stiff brownish green calcareous clay with slight sand content.*	5-6-9
10	-61.0	-62.5		-64.5	Stiff brownish green calcareous clay with slight sand content.*	7-8-7
11	-66.0	-67.5	-64.5		Very stiff brownish green calcareous clay with slight sand content.*	4-6-11

Testing and Sampling in accordance with ASTM D 1586-67

Remarks: *Locally called Marl.
In this area encountered high debris on river bottom and in soil. Necessary to relocate hole slightly twice to bypass debris.

Figure 11. Two SPT logs from the Charleston Peninsula.
The log on the left was taken from 2015 along "The Battery" while the log on the left was taken north of "The Battery" at the Port of Charleston. Refer to Figure 10 for locations.

8.3.2. Dense Sand / Gravel

It is reported that there can be a dense sand/gravel layer above the Cooper Marl that can make it difficult to drive concrete piles through it. Additional investigation will be required during PED to determine if/where there are dense sand/gravel layers along the alignment.

8.3.3. Vibrations During Pile Driving

Vibrations during pile driving is a concern as there will be many structures located adjacent to the CSRM project. Some of these structures have historical significance. There are methods to estimate distances but is dependent on soil stratigraphy, which detailed stratigraphy is unknown at this time. A general rule of thumb is that vibration damage is not likely to occur outside of 50 feet from the pile (either top or tip of pile, whichever is closer) for piles 50 feet or less in lengths or the length of the pile. With piles lengths approaching 90 feet and some piles being battered, preconstruction survey on properties within a 100-ft buffer from wall centerline was assumed.

Additionally, vibration monitoring will be required during construction as various locations throughout the area but not at each residential structure.

8.4. Structural Steel Elements

Structural steel elements that are exposed to air and salt water will require cathodic protection.

8.5. Future Work Required during Design Pre-construction Engineering and Design Phase

Due to the study area size, schedule and funding constraints, there is much geotechnical analysis and design required during the PED phases. Some of this work, such as subsurface exploration, will need to start immediately at the beginning of PED in order to obtain the necessary information to complete geotechnical and structural analyses. The work required during PED is discussed in detail below.

8.5.1. Subsurface Exploration

Subsurface information will need to be gathered along the alignment. Along with determining stratigraphy, it will be important to know if there is any man-made fill or construction debris that may affect construction and pile installation. When developing the soil exploration program, the PDT should determine areas where the presence of man-made fills are likely so additional exploration can be completed to define the type and extents of it. Soil exploration should be extended into the Cooper Marl, to a depth of at least 20 feet below the expected pile tip elevation (U.S. Department of Transportation Federal Highway Administration, Design and Construction of Driven Pile Foundations – Volume I, page 87). Soil exploration should consist of CPT soundings supplemented with SPT borings. The SPT borings will be used verify the soil behavior type determined during CPT data reduction. Additionally, undisturbed samples should be collected and tested. The testing should consist of both drained and undrained shear strength determination, consolidation, and soil classification tests (Atterberg limits and grain size distribution). The spacing between soil exploration will likely range from 250 to 1,000 feet.

If soil-structure interaction modeling will be required, in situ modulus values will need to be determined. Flat plate dilatometer or pressuremeter testing would be required. Additionally, the flat plate dilatometer could also be used to supplement the determination of shear strengths.

8.5.2. Seepage Analysis for T-wall and Combo-Wall Sections

Seepage analysis will need to be completed to determine the proper depth of seepage cutoff walls and the uplift pressures on the T-wall footing.

8.5.3. Pile Design

The design of the piles will be required. The design will include selection of pile type (steel H-pile, concrete piles, micro piles, etc.) considering costs, drivability, vibration generation, constructability, and longevity (related to corrosion). Determination of both axial and lateral load capacity with consideration of seismic loading will be required along with downdrag calculations, where applicable. Pile load tests (dynamic, static, and lateral) should be evaluated to determine the appropriateness of completing that at various stages of design and construction.

In addition to the typical pile design, pile driving generated vibrations will need to be evaluate. Both magnitude and distance travel will need to be determined. Maximum allowable vibration amplitudes along with construction monitoring requirements will be needed.

8.5.4. Lateral Earth Pressure

It is anticipated in some locations the wall will also act as a retaining wall. Appropriate lateral earth pressures will need to be determined to be used in the design of the retaining wall.

8.5.5. I-Wall Evaluation

There could be a cost savings potential if I-walls can replace T-walls and this should be evaluated along the project alignment where the exposed stem height is 4 feet or less. The PDT will need to realize that the design requirements for an I-wall are more intensive that T-walls and need to be considered this when developing the soil exploration program (smaller spacing) and design schedule.

8.5.6. Penetrations Through Barrier

Penetrations through the barrier will be necessary for utilities and stormwater drainage. These penetrations will need to be designed.

The PDT should consider determining utility corridors in which multiple utilities can penetrate the barrier in one designated segment. This would minimize the number of crosses.

8.5.7. Operation and Maintenance Manual

An Operation and Maintenance Manual (O&M Manual) will be required once the project is constructed and turned over to the Non-Federal Sponsor. Geotechnical input to the O&M Manual will be required during PED but mainly during and after construction.

9. CONSTRUCTABILITY

There are various constructability issues that could be encountered for the Charleston CSRM which are indicated and discussed below.

9.1. Pile Installation

Piles will be driven throughout the Peninsula Area, sometime very near to existing structures, with some having historical significance. Pile driving will cause vibrations and pre-construction surveys will need to be completed along with monitoring of vibrations.

If dense sand and gravel layers above the Cooper Marl are encountered during soil exploration, driving of displacement-type piles (i.e. square concrete piles or closed-end pipe piles) could be hampered and would require pre-augering. Additionally, driving displacement-type piles through very dense layers could increase the magnitude of vibrations and distance they travel.

9.2. Soft Soils

If soft soils are present, these could be problematic from the stand point that any additional load on the foundation will cause consolidation and downdrag on piles. Settlement and drawdrag will need to be considered during design.

Soft soils could also cause issues in which the soils cannot support construction equipment, or excess rutting occurs.

9.3. Loose Sands and Adjacent Shallow Foundations

If both loose sands and structures on shallow foundations are present along the alignment, pile driving and excess vibration may cause the loose sands to densify and lead to settlement of the shallow-founded structures. The density of foundation soils and type of structural foundation will need to be evaluated during design.

9.4. Man-Made Fills

Historically, the peninsula was expanded by placement of fill into the low areas around the perimeter. The man-made fills could make pile driving difficult and could require pre-augering.

10. DESIGN GUIDANCE

A list of anticipated design guidance documents that will facilitate design are as follows:

EC 1110-2-6066 Design of I-walls

EC1165-2-217 Review Policy of Civil Works

ECB 2018-15 Technical Lead for E&C Deliverables

ECB 2017-3 Design and Evaluation of I-Walls Including Sheet Pile Walls

EM 1110-1-1804 Geotechnical Investigations

EM 1110-1-1904 Settlement Analysis

EM 1110-1-1905 Bearing Capacity of Soils

EM 1110-2-1901 Seepage Analysis Control and for Dams

EM 1110-2-1902 Slope Stability

EM 1110-2-1906 Laboratory Soil Testing

EM 1110-2-1913 Design and Construction of Levees

EM 1102-2100 Stability Analysis of Concrete Structures

EM 1110-2-2502 Retaining and Flood Walls

EM 1110-2-2504 Design of Sheet Pile Walls

EM 1110-2-2902 Conduits, Culverts, and Pipes

EM 1110-2-2906 Pile Foundations

EM 1110-2-6050 Response Spectra and Seismic Analysis for Concrete Hydraulic Structures

EM 1110-2-6051 Time-History Dynamic Analysis of Concrete Hydraulic Structures

EM 1110-2-6053 Earthquake Design and Evaluation of Concrete Hydraulic Structures

EP 1110-2-18 Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures

ER 1110-1-12 Quality Management

ER 1110-1-261 Quality Assurance of Laboratory Testing Procedures

ER 1110-1-8100 Laboratory Investigation and Testing

ER 1110-2-401 Operation, Maintenance, Repair, Replacement, and Rehabilitation Manual for Projects and Separable Elements Managed by Project Sponsors

ER 1110-2-1802 Reporting Earthquake Effects

ER 110-2-1806 Earthquake Design and Evaluation for Civil Works Projects

ER 1110-2-8160 Planning and Design of Temporary Cofferdams and Braced Excavations

ETL 1110-2-39 Pile Foundations

ETL 1110-2-569 Design Guidance for Levee Underseepage

ETL 1110-2-575 Evaluation of I-walls

Naval Facilities Engineering Command Design Manual 7.02 Foundations and Earth Structures

Unified Facilities Criteria 3-220-01N Geotechnical Engineering Procedures for Foundation Design of Buildings and Structures

Federal Highway Administration. FHWA GEC 012 – Design and Construction of Drive Pile Foundations, Volume I. September 2016.

Federal Highway Administration. FHWA GEC 012 – Design and Construction of Drive Pile Foundations, Volume II. September 2016.

National Cooperative Highway Research Program. NCHRP 25-25/Task 74 Current Practices to Address Construction Vibration and Potential Effects to Historic Buildings Adjacent to Transportation Projects. September 2012.

11. REFERENCES

U.S. Army Corps of Engineers, Norfolk District. Integrated City of Norfolk Coastal Storm Risk Management Feasibility Study / Environmental Impact Statement. July 2018.

U.S. Army Corps of Engineers, Fort Worth District. Coastal Texas Protection and Restoration Feasibility Study, Draft Integrated Feasibility Report and Environmental Impact Statement. October 2018.

12. ATTACHMENTS

Additional details to various topics can be found in the following attachments:

Attachment 1: Seismic Evaluation
Attachment 2: Top of Cooper Marl and Existing Boring Locations

Charleston Peninsula Coastal Storm Risk Reduction Feasibility Study (CPS)
Geology and Geotechnical Engineering
Attachment 1: Seismic Evaluation

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Geology and Geotechnical Engineering
Attachment 1: Seismic Evaluation

1.1. General

This seismic evaluation was conducted in accordance with the guidance established in ER 1110-2-1806 “Earthquake Design and Evaluation for Civil Works Projects” dated 31 May 2016. Guidance outlined in ECB 1110-2-6000 (DRAFT) “Selection of Design Earthquakes and Associated Ground Motions” was also utilized in the evaluation. This evaluation was augmented by products from the United States Geological Survey (USGS) and USACE, Risk Management Center (RMC) at (https://radsii.usace.army.mil/RMCResources.aspx).

1.2. Project Hazard Potential

The Charleston Peninsula is located at coordinates latitude N 32.787° and longitude W 79.937°. The Charleston Peninsula is a very densely populated area with thousands of resident and non-resident buildings. Because of its proximity to the coast and low elevation, the Charleston Peninsula is frequently inundated from tropical systems and the occasional perigean spring tide. Additionally, the impacts of harbor traffic and wind driven waves have caused extensive erosion around the peninsula resulting in loss of shoreline and relocation of infrastructure. The city has already started mitigation efforts to protect a portion of the southern peninsula by constructing a retaining wall. However, the increase in the frequency and intensity of tropical systems coupled with sea-level rise and harbor traffic has put a heavily populated area at greater risk. Additional protective measures need to be considered in order to not only protect the southern tip of the peninsula, but the peninsula as a whole. According to Table 1, the Charleston Peninsula has a High Hazard Potential Project rating, due to the presence of a residential population at risk (PAR).

Table 1: Hazard potential classification for the Charleston Peninsula based off ER 1110-2-1806.

Table B-1
HAZARD POTENTIAL CLASSIFICATION
FOR CIVIL WORKS PROJECTS

	Category[1]			
Hazard Potential Classification	Direct Loss of Life[2]	Lifeline Losses[3]	Property Losses[4]	Environmental Losses[5]
Low	None Expected	No disruption of services – repairs are cosmetic or rapidly repairable damage	Private agricultural lands, equipment, and isolated buildings	Minimal incremental damage
Significant	None Expected	Disruption of essential facilities and access	Major or extensive public and private facilities	Major or extensive mitigation required or impossible to mitigate
High	Probable (one or more)	Disruption of critical facilities and access	Extensive public and private facilities	Extensive mitigation cost or impossible to mitigate

Guidance established in ER 1110-2-1806 and ECB 1110-2-6000 states that projects having a “High Hazard Potential Project” shall have a Maximum Design Earthquake (MDE) that equals the Maximum Credible Earthquake (MCE). The required seawall performance under the MDE is damage control performance and under the MCE is collapse prevention performance.

1.3. Previous Seismic Evaluations

There have been no seismic considerations in the original design and no subsequent seismic evaluations have been performed prior to this document.

1.4. Seismotectonic Setting

1.4.1. General

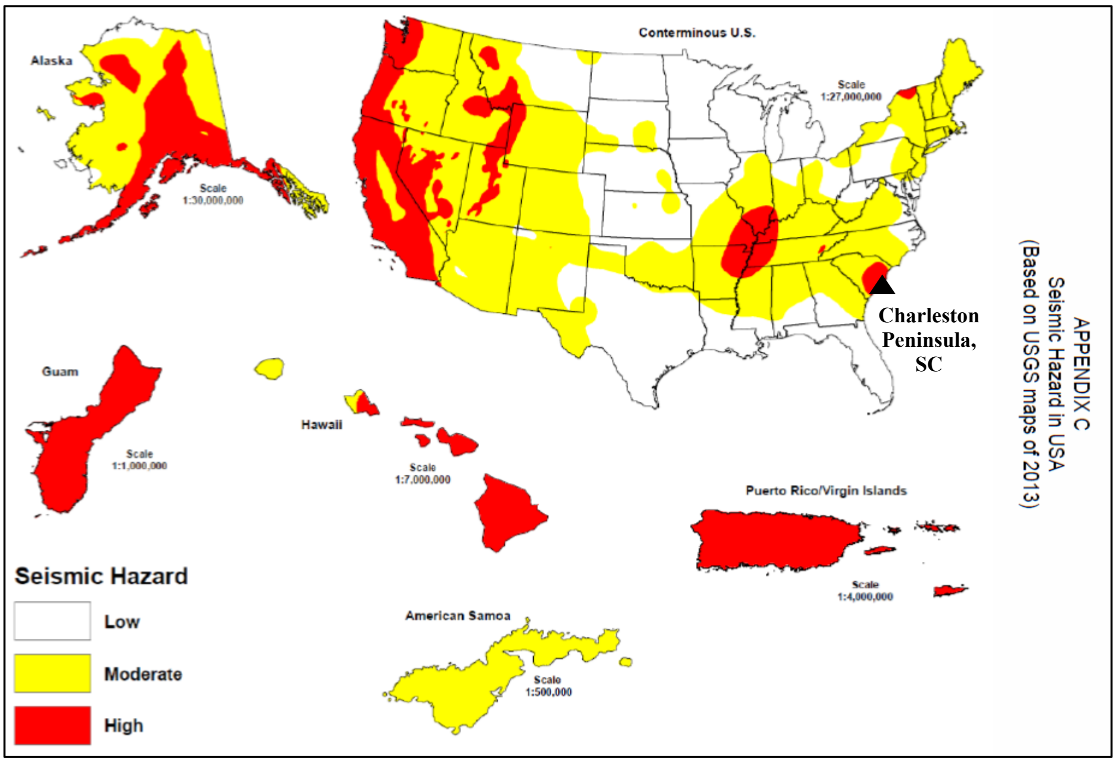

Figure 1: Project location shown on seismic hazard map of the USA, from ER-1110-2-1806.

ER 1110-2-1806 requires that the project site be located on the seismic hazard map (Figure 1). The Charleston Peninsula is within a high seismic hazard zone. While this map is generalized, it does indicate that ground motions will need to be considered as part of the construction design. Present state-of-the-art practice has moved toward methods that generate regional and site-specific (if needed) probabilistic and deterministic Peak Ground Acceleration (PGA), or response spectra analysis (Leyendecker et al., 2000; NEHRP, 2009; 2012; ASCE/SEI 7-10; International Building Code, 2012; USGS, 2014; 2016).

Seismic hazard maps presently available by the United States Geological Survey offer the best up-to-date seismic probability assessments. A more detailed USGS seismic hazard map, filtered and adjusted for seismicity in the southeastern U.S., is shown in Figure 2. This map shows the contoured peak ground acceleration (PGA) to be expected within southeastern U.S. from an earthquake having a return period of 2,475 years, or a 2% probability of exceedance in 50 years. This corresponds to the Maximum Credible Earthquake defined in ER 1110-2-1806, EM 1110-2-

6053, and ECB 1110-2-6000. Ground motions and spectra will be described in detail later in this chapter. The USGS 2014 seismic hazard map by Petersen et al. (2015), shown in Figure 2, suggests that an earthquake with a 2% probability of exceedance in 50 years could produce a PGA that ranges from 0.6 to 0.8g near the Charleston Peninsula. ER 1110-2-1806 and Krinitzsky (2003) discourage the use of probabilistic methods alone to estimate ground motion parameters because they may be much different from those using deterministic methods. This is due to inherent sampling bias and limited recorded history (<100 years) in probabilistic methods, which when extrapolated to longer time periods results in much larger ground motions than those using deterministic methods that utilize kinematic fault movement/history parameters. ER 1110-2-1806 states that the MCE should be developed by deterministic methods, but it also needs to be informed by probabilistic methods as well.

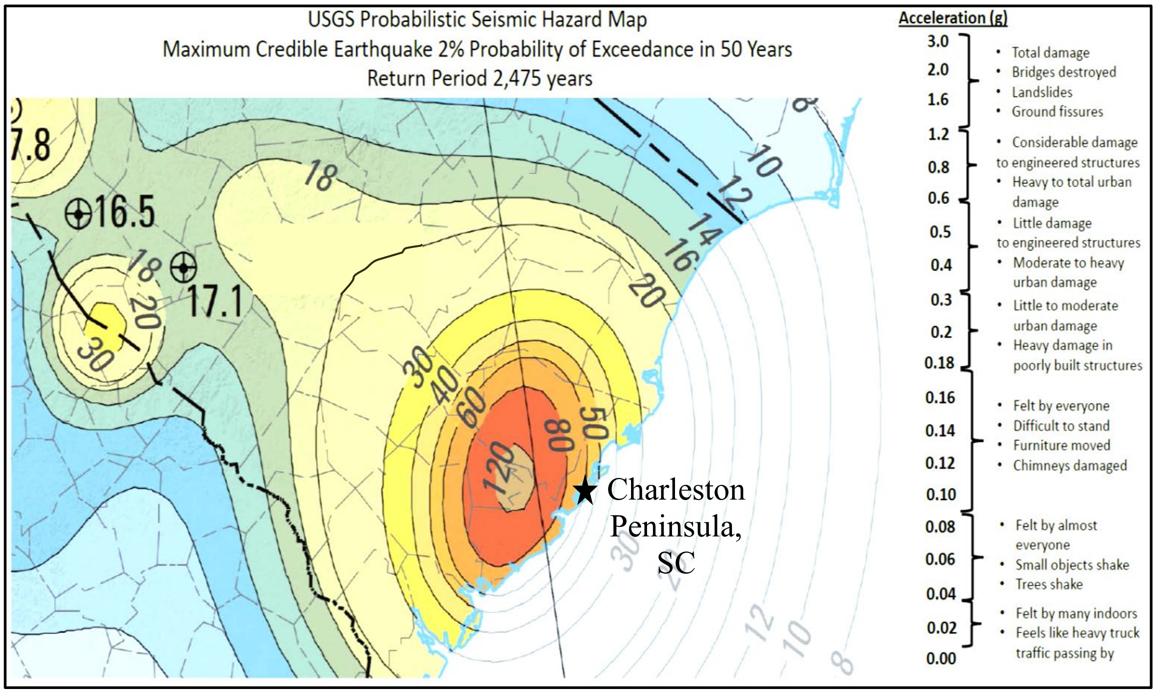

Figure 2: USGS Seismic Hazard Map, PGA, 2% Probability of Exceedance in 50 Years, from Peterson et al. (2015).

Note: Contours of peak acceleration expressed in percent of gravity (%g). Point values shown indicate local minimum %g.

Because the project lies within an area known to be highly influenced by a zone of high seismic activity, and that the project has been deemed to have a high hazard classification, the use of both deterministic and probabilistic methods is deemed to be appropriate in order to preserve engineering conservatism (ECB 1110-2-6000; ER 1110-2-1806; Krinitzsky, 2003).

1.4.2. Geology of the Central and Eastern U.S. Seismotectonic Zone

The project site lies within the Central and Eastern U.S. Seismotectonic Zone (CEUS), the seismotectonic zone is located hundreds of miles from active plate tectonic boundaries and is characterized by relatively low rates of seismicity. However, the Charleston Peninsula is a localized “hot spot” of high seismic activity. This area is known as the Charleston Seismic Zone. A generalized regional geologic map of the CEUS is shown in Figure 3. The CEUS is comprised of Pre-Cambrian stable interior cratonic crust, Paleozoic-aged imbricated, thrust sheet stacks of metamorphic, igneous, and metasedimentary sediments comprising the Appalachian chain, Mesozoic-aged rift basin sequences of intermediate and mafic intrusive igneous rocks, metavolcanic and sedimentary rocks, and younger Gulf Coast sedimentary rock. These areas have slightly different bulk rock seismic velocities, and slightly different rates of seismic occurrence, which may be due to effects of reactivation of pre-existing faults and planes of weakness in response to present-day remote tectonic forces.

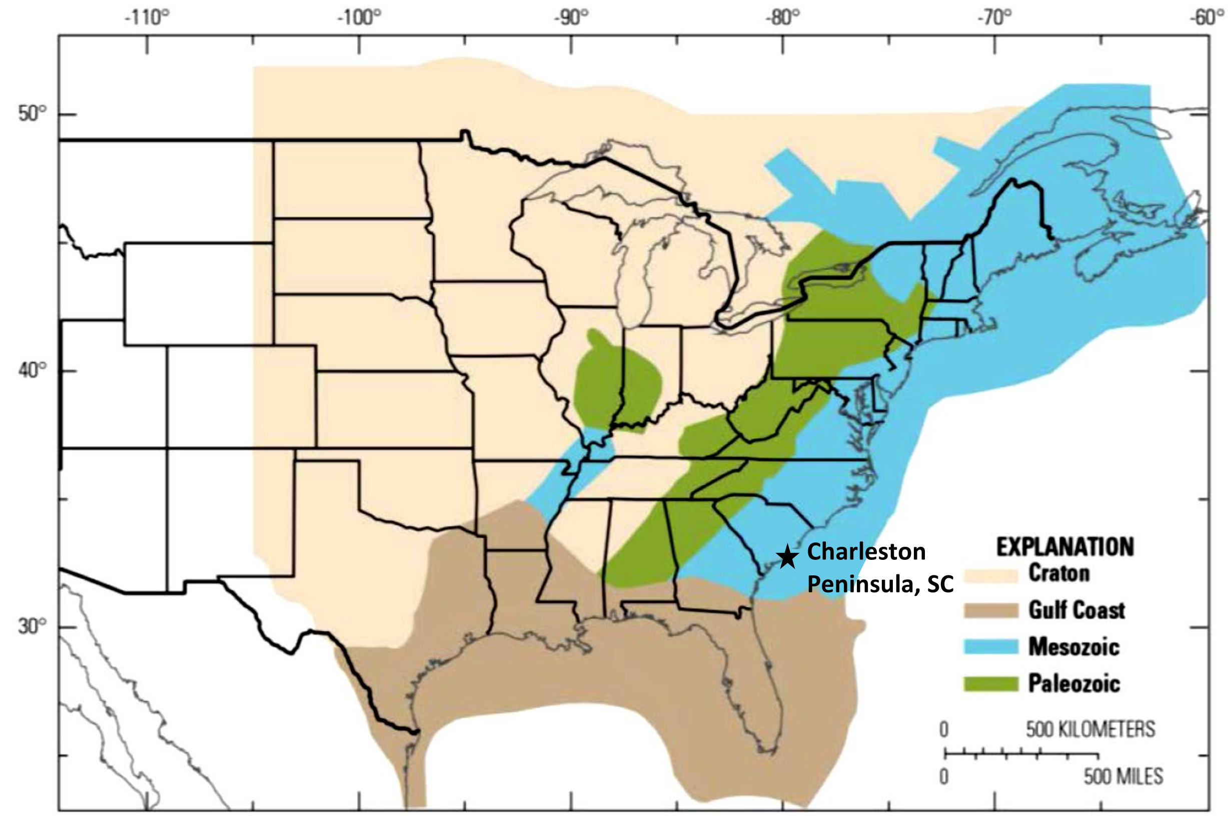

Figure 3: General geologic and seismic velocity structure of the CEUS seismotectonic zone.

There are no active surficially expressed, regional-scale transform faults such as the San Andreas Fault Zone, or active subduction zones. For the CEUS seismotectonic zone, most large scale fault movement had occurred during the Late Paleozoic and Mesozoic eras. The last major tectonic event was related to Mesozoic rifting and opening of the Atlantic Ocean. However, there is active low magnitude seismicity and large regional earthquakes have occurred in the CEUS in historic time. The fault source for many strong motion earthquakes in CEUS is generally not well defined because there is little to no surficial fault expression, as seen in the western U.S. While there is general agreement that large magnitude earthquakes in the CEUS are the result of shallow to deep basement crustal fault slippage, a clear association of even some of the largest historical earthquakes (e.g. the 1886 Charleston, S.C. earthquake) with a particular fault has been difficult to recognize. Therefore, in the CEUS, earthquake sources are generally defined as areas or volumetric source zones which is deemed acceptable in accordance with ER 1110-2-1806, EM 1110-2-6053, and ECB 1110-2-6000.

1.4.3. Earthquake Catalogue, 1964 - Present

An earthquake map and event catalogue was created for this evaluation using data from the Search Earthquake Catalog which is managed by the USGS. It can be accessed https://earthquake.usgs.gov/earthquakes/search/. Input parameters used to query the online application are tabulated in Table 2 below:

Table 2: Search parameters used to query the Search Earthquake Catalog for Charleston Peninsula, South Carolina.

ANSS PARAMETER DESCRIPTION	INPUT PARAMETERS
Start Date/ Time	January 01, 1800
End Date/ Time	December 18, 2019
Minimum Latitude	31.376
Minimum Longitude	-74.839
Maximum Latitude	38.465
Maximum Longitude	-84.990

Figure 4 shows the earthquake event map for all earthquakes recorded by the Search Earthquake Catalog using the input parameters denoted in Table 2. The query returned a total of 1,327 earthquakes that were measured from 1800 to present, and nearly all were less than moment magnitude (Mw) 5.0. It is assumed that the seismological record in the early 20th century is underrepresented due to the lack of seismological monitoring. Nonetheless, large magnitude earthquakes are documented to have occurred within the area, specifically, the Charleston, SC, 1886 earthquake (Mw = 7.3). Table 3 shows the number of earthquakes by strength, PGA, and relative effects from Figure 4. The majority of these earthquakes are very weak to weak and may not have been noticed by the public.

Table 3: Magnitude distribution of earthquakes from seismic catalogue query (1800 to present) within 300 km of the project site.

EQ Strength (Mw)	# EQ Measured ANSS	Est. PGA Range (Epicentral g)	Observed Effects
Magnitude 0 to 1	11- quakes	< 0.002g	Felt by very few people; barely noticeable.
Magnitude 1 to 2	80-quakes	0.002g – 0.008g	Felt by few people; mostly upper floors.
Magnitude 2 to 3	139-quakes	0.008g – 0.014g	Noticeable indoors, especially on upper floors, may not be recognized as an EQ.
Magnitude 3 to 4	31-quakes	0.014g – 0.039g	Felt by many indoors, few outdoors. Feels like a heavy truck passing.
Magnitude 4 to 5	12-quakes	0.039g – 0.18g	Felt by almost everyone, some people awakened. Small objects moved. Some plaster falls. Chimneys slightly damaged.
Magnitude 5 to 6 *1 quake >6.0 Mw	1-quakes	0.18g – 0.30g	Little to moderate urban damage. Heavy damage in poorly built structure.

The Charleston Seismic Zone, however, is characterized by a dense clustering of earthquakes (1.0 < Mw < 5.0) that indicate active seismicity. Northwest of the project site, there is a broad zone of weak to moderate seismicity (Eastern Tennessee Source Zone) that is associated with the Western Blue Ridge and Valley and Ridge geologic provinces (see Figure 4). Seismicity within this area is unique to its geology and does not relate to the project site because, at >300 km away, it is outside of the maximum source to site consideration for this project (ER 1110-2-6000).

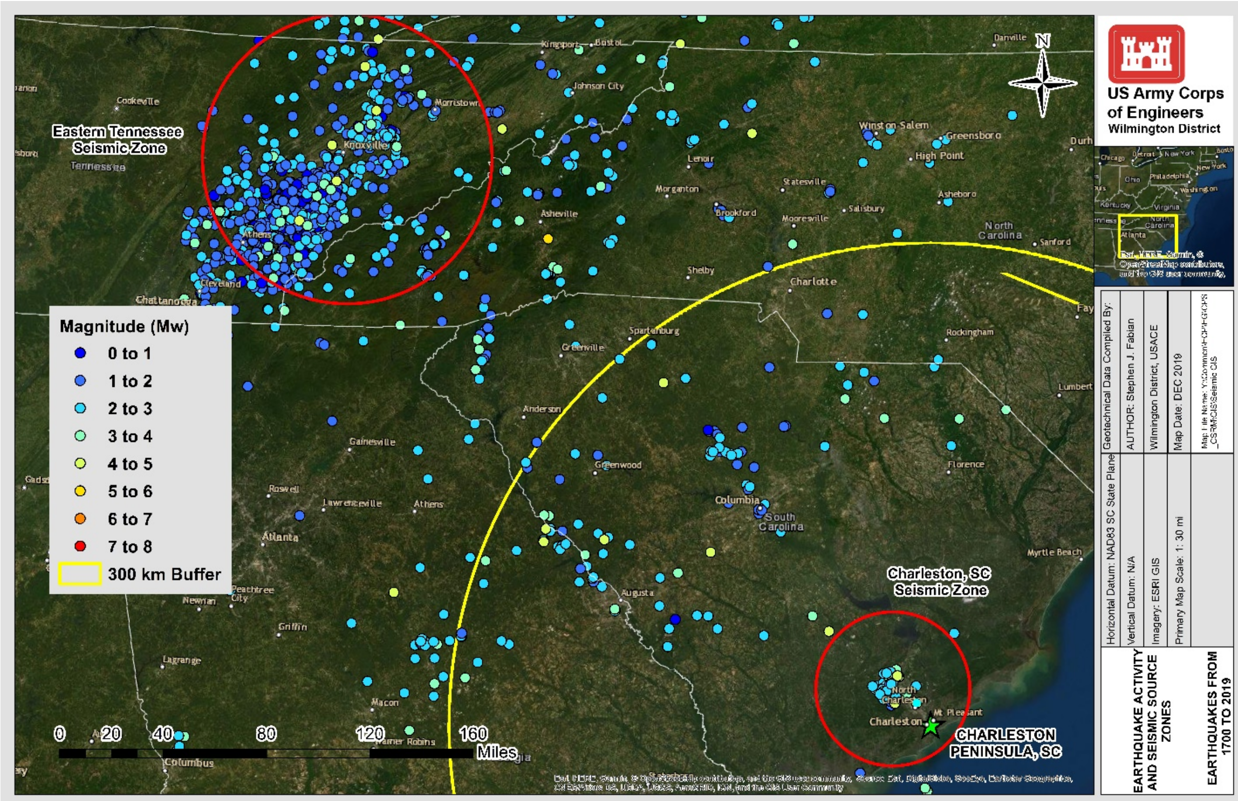

Figure 4: Map showing measured earthquake activity and seismic source zones within 300 km of the project site.

Note: Sourced data (1700-present) is from the seismic catalogue.

1.4.4. Regional Seismic Source Model Defined

The seismic source model used by the 2014 National Seismic Hazard Mapping Program for the CEUS (Peterson et al., 2014) considers both seismicity-based background sources and fault-based sources and utilizes data and models from the CEUS Seismic Source Characterization for Nuclear Facilities (CEUS–SSCN) project, which accounts for broader uncertainties and replaces older seismic source models. The Peterson et al. (2014) source model assumes that future large earthquakes are more likely to nucleate near previous earthquakes with M_w greater than or equal to 3.0 (see Figure 5). The model also distinguishes seismotectonic zones in the CEUS with distinct seismicity and maximum earthquake magnitudes in order to accommodate some possibility that the historical seismicity does not fully represent likely sources of background earthquakes. As shown in Figure 5, the project lies in an area characterized by high levels of weak to moderate seismicity (3 < Mw < 6) which is influenced by strong motion earthquakes originating from the Charleston Seismic Zone or by local diffuse background sources.

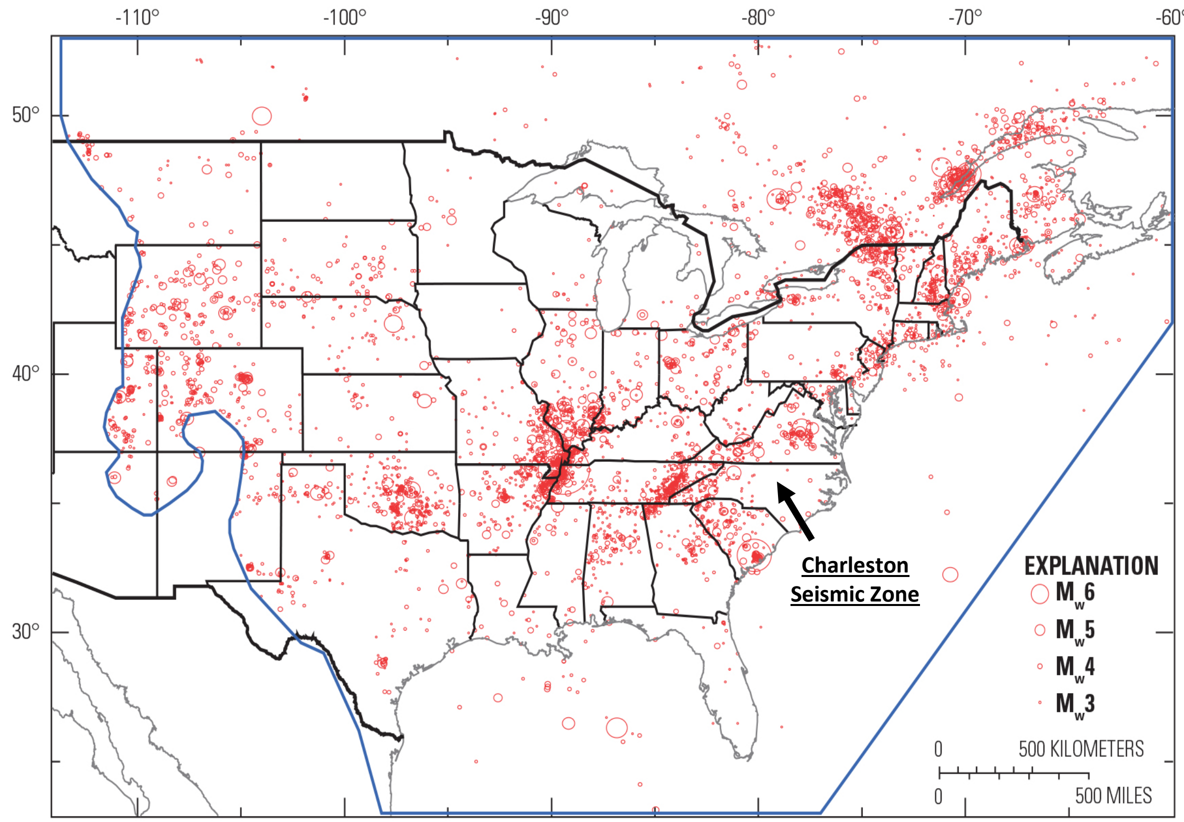

Figure 5: CEUS Earthquake data 1700 to 2012 used for USGS-sponsored seismic hazard mapping, from Peterson et al. (2014).

1.4.5. Review of USGS Quaternary Fault Database

Faults capable of producing a strong motion earthquakes (Mw > 5.0), that lie within a 50 km/31 mile radius of a project site, must be identified in accordance with ER 1110-2-1806 and ECB 1110-2-6000 (1898 Charleston Earthquake). The USGS Quaternary fault and fold database of the United States (http://earthquake.usgs.gov/hazards/qfaults/) was reviewed to locate any active Quaternary-aged (past 1,600,000 years) faults in close proximity to the Charleston Peninsula. No active Quaternary faults were found, but evidence of paleoliquefaction has been mapped along the coastal areas of North and South Carolina (see Figure 6)[1]. Guidance initially established by Krinitzsky (1995) and reinforced by ECB 1110-2-6000 states that if no active faults are found within 50km/31 miles of a project site, then far-field attenuation curves shall be used to evaluate

[1] There is evidence of large strong motion earthquakes that have occurred within the last 15,000 years during the latter part of the Holocene, which are related to the Charleston Seismic Zone. Liquefaction features such as sand boils and sand fissures, first recognized in the region following the 1886 earthquake, have been mapped and geochronologically dated throughout the coastal region of South Carolina. Though the liquefaction features demonstrate that strong prehistoric shaking occurred, they provide no information on specific source fault attributes such as azimuth, length, dip, sense of motion, or slip-rate (Wheeler, 1998).

MCE ground motions to a maximum distance of 300 km/186 miles. The Charleston Peninsula lies in an area where a highly active source is present and capable of generating large magnitude earthquakes.

Figure 6: Location of paleoliquefaction features, from USGS Quaternary Fault and Fold Database of the U.S.

1.4.6. Charleston Seismic Zone Defined

Charleston Seismic Zone is a region of high seismic hazard centered 30 kilometers northwest of Charleston, South Carolina, where a large earthquake (Mw = 7.3) caused widespread damage in 1886. The 1886 Charleston earthquake is the largest earthquake known to have occurred in the southeastern United States and was likely due to a reactivated deeply buried basement fault (Rankin, 1977). Observations of earthquake activity within the Charleston Seismic Zone suggest that it may be associated with a failed extensional rift basin[2] within the Mesozoic-aged extended crust (ECB 1110-2-6000). A detailed map showing the contoured seismicity, tectonic structure, and paleoliquefaction features within the epicentral region of the 1886 Charleston earthquake is shown in Figure 7. Previous workers have utilized mapping of sand boils (Amick et al., 1990), geologic well logs (Colquhoun et al., 1983; Weems and Lewis, 2002), seismic survey (Behrendt et al., 1983; Schilt et al., 1983; Marple and Miller, 2006), numerous kinematic and seismotectonic studies (chiefly, Dura-Gomez and Talwani, 2006), paleoseismic studies (Talwani and Schaffer, 2001) and even geomorphological mapping of the Ashley River (Marple and Talwani, 2000) to ascertain specific fault characteristics, but disagreements (e.g., Marple, 2011)

[2] Failed rift basins are deeply buried, sediment filled, faulted basins that are oriented at a high angle to adjacent oceanic plates or orogenic belts. They form by faulting from extensional tectonics and crustal thinning. These structures are thought to represent failed initiation points of ancient continental rifting and ocean basin formation.

among seismic workers forestall detailed fault modelling in this seismic evaluation. An MCE of 7.3 Mw + 1σ is selected for use in characterizing the Charleston Seismic Zone.

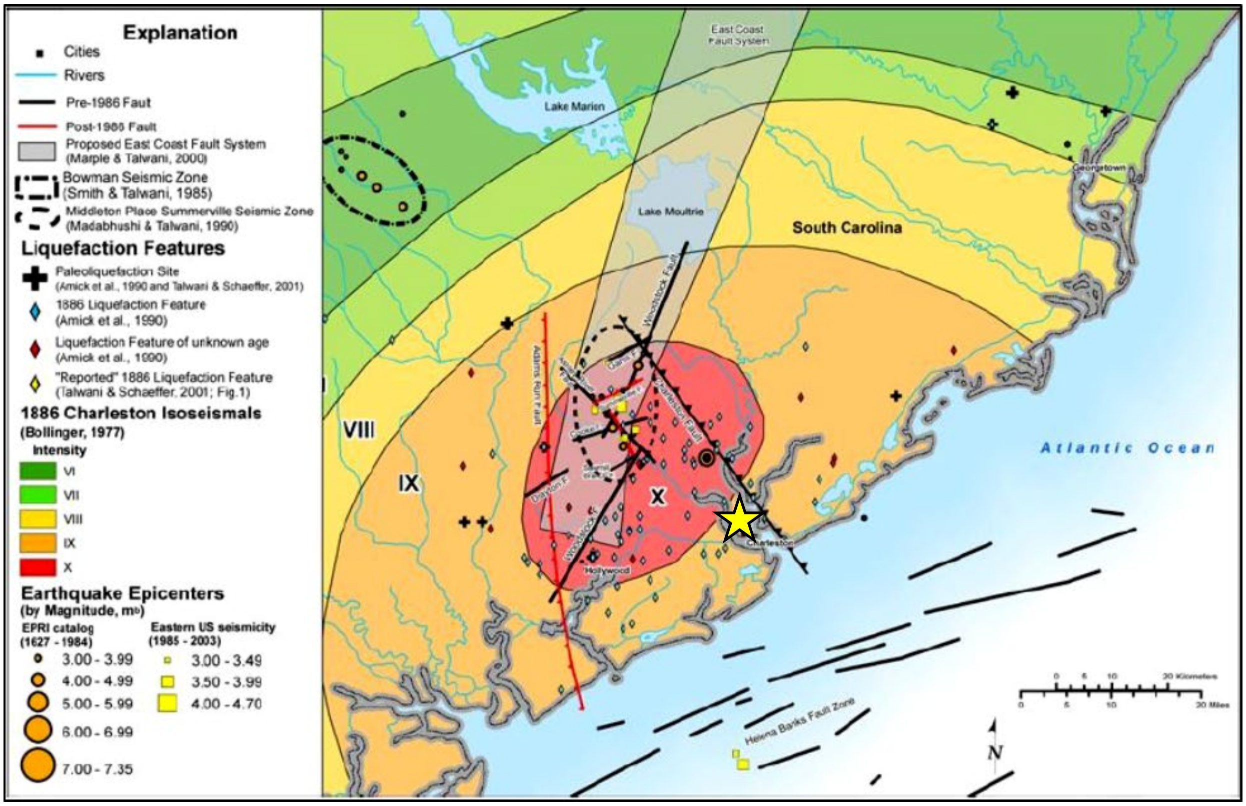

Figure 7: Map from ECB 1110-2-6000 showing seismicity, tectonic and paleoliquefaction, in the epicentral region of the 1886 Charleston earthquake, from Southern Nuclear Company (2007).

1.5. USGS Uniform Hazard Tool and Seismic Hazard Deaggregation for the Project Site

The USGS's Uniform Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/) was used to evaluate the seismic hazard to the site. Inputs to the tool include:

- USGS Probabilistic Seismic Hazard Map Edition: Dynamic conterminous U.S. 2014 (v4.1.1) was used because it was the only dataset capable of interacting with the deaggregation tool.
- Spectral Period: PGA, 0.2, 1.0, and 2.0 seconds evaluated.
- Latitude/Longitude Inputs: 32.787 Lat. / -79.937 Long.
- Time Horizon: Return period 2,475 year corresponding to a 2% in 50 years AEP selected.
- Site Class: Only one Vs30 site class was available in the application: B-C (760 m/s) designated "firm rock" and A (2000 m/s) which is designated "hard rock." Because the uppermost crustal strata in the region consists of loosely consolidated clayey sands underlain by dense silts and clays, a Site B-C boundary of 760 m/s was selected for use to initially evaluate the seismic hazard and corresponding deaggregation.

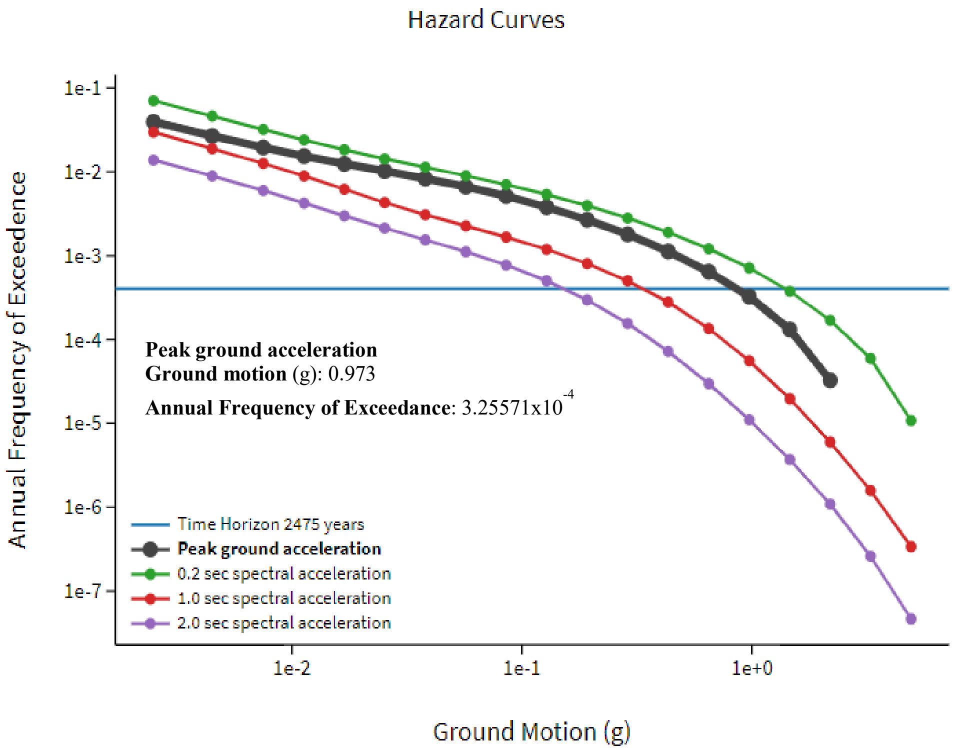

Figure 8: Probabilistic Seismic Hazard Curve showing PGA for 2% in 50 years AEP (2,475 year return period), using USGS 2014 Conterminous U.S. data.

Probabilistic seismic hazard curves showing the PGA 0.2, 1.0, and 2.0 second spectral acceleration predicted for the project site are shown in Figure 8. The site-predicted PGA for an earthquake having a return period of 2,475 years is approximately 0.973g, which is slightly higher than the USGS seismic hazard map shown in Figure 2 ranging from 0.6 to 0.8g. Spectral ground motion on the Charleston Peninsula was also predicted by the Uniform Hazard Response Spectrum (Figure 9). Based upon probabilistic hazard mapping, the PGA at the site is predicted to be 0.8561g, but the largest and most likely damaging ground motion is 1.3972g at a spectral period of 0.2 seconds (Figure 9).

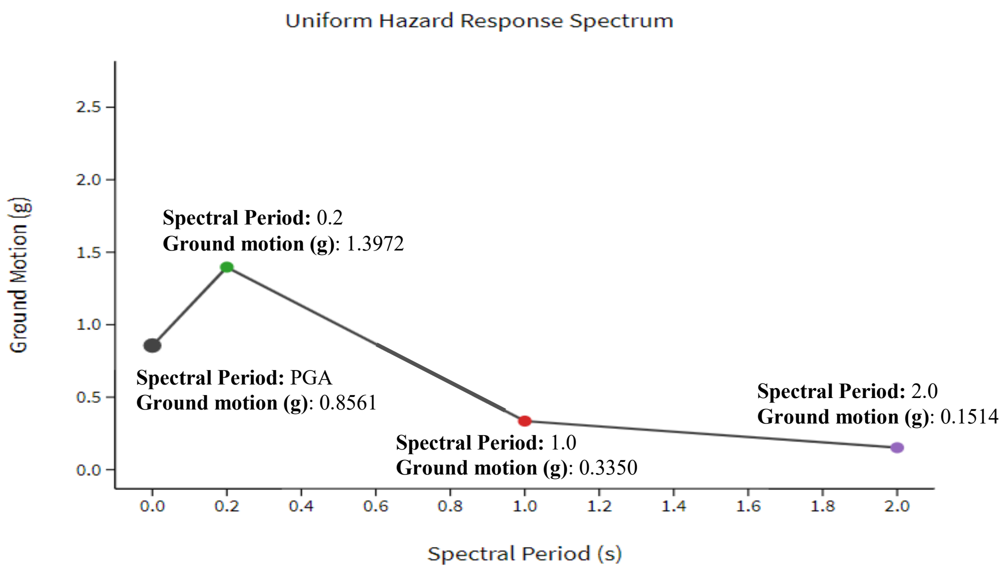

Figure 9. Uniform Hazard Response Spectrum predicted for the project site showing PGA with 2% in 50 years AEP (2,475 return period).

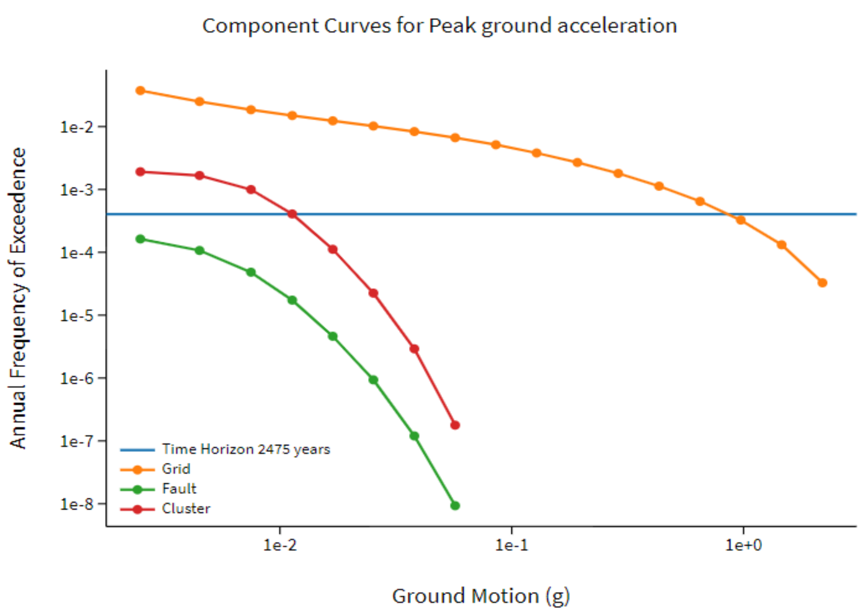

Figure 10: Component curves showing PGA differences based on different seismic sources as calculated in probabilistic hazard analysis.

Note, "gridded" seismic data produces the highest ground motions.

It should be noted that the probabilistic seismic hazard predicts the total seismic hazard by integrating all potential source magnitudes and distance, and applying a statistical prediction of the event return period. These return periods are generally greater than what is empirically supported by human observation. As a result, this may produce higher ground motions than what is geologically possible at a particular site, as discussed in Krinitzsky (2003). Figure 10 illustrates this by showing the PGA differences that arise during probabilistic seismic hazard analysis when considering point seismic sources, faults, or gridded seismic data. Probabilistic seismic hazard curves (see Figure 8 and Figure 9) display relatively high ground motions, but provide a good initial estimate of the total ground motion and associated seismic risk. Probabilistic analysis is used to inform site-specific deterministic seismic hazard analysis (ER 1110-2-1806; ECB 1110-2-6000), discussed later in this chapter.

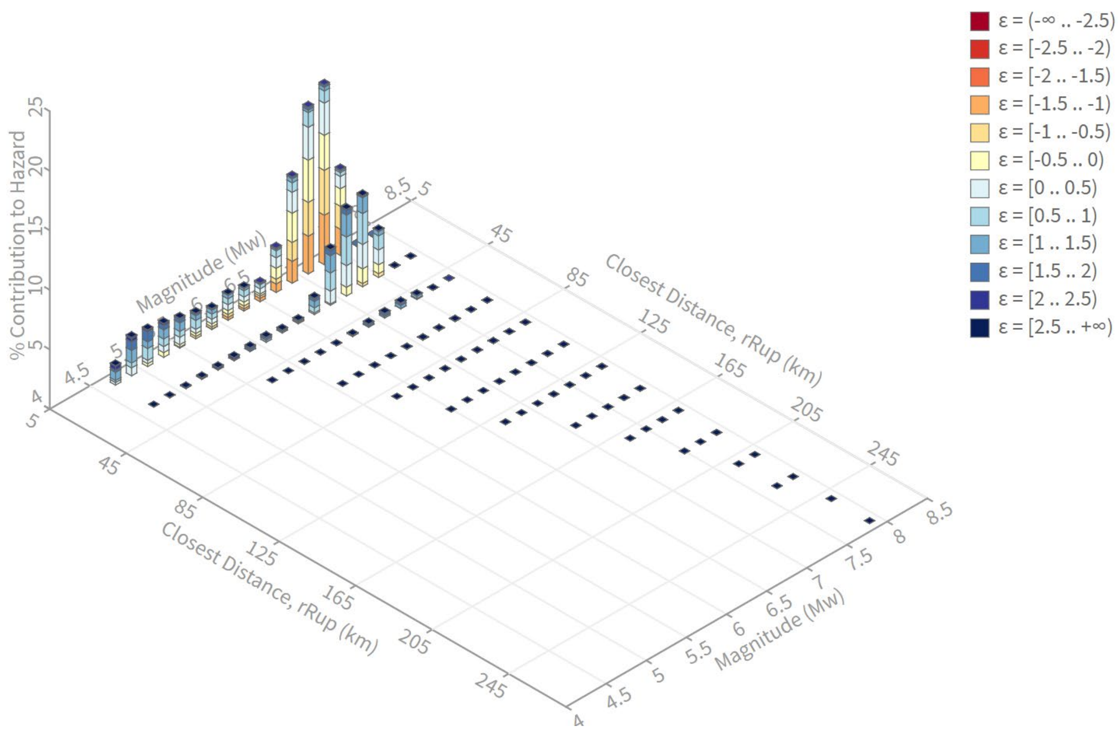

Figure 11: Deaggregation chart (USGS, 2014) showing itemized seismic hazard contribution to the Charleston Peninsula.

A seismic hazard deaggregation chart was constructed using USGS Uniform Hazard Tool. Deaggregation charts measure the hazard contribution from a number of seismic sources to the project site (see Figure 11). The deaggregation process takes integrated ground motion from all seismic events within the U.S., statistically itemizes it, and projects each seismic hazard based upon magnitude (Mw) and distance (rRup) to the project site. The percent contribution to the total seismic hazard is measured in terms of ε, which is the number of logarithmic standard deviations from which the itemized seismic hazard deviates from the total mean predicted ground motion.

Figure 11 shows a unimodal distribution in the total seismic hazard to the project site. The most prominent contribution to the seismic hazard is the Mw = 7.3 earthquake[3] ($\varepsilon_o = -1.33\sigma$), located at a distance of 10.72 km.

[3] This earthquake corresponds to the 1886 Charleston Earthquake, which had an estimated moment magnitude of 6.9-7.3. The earthquake caused 60 deaths and destroyed 2,000 buildings. Damage estimated to be between 5-6 million dollars.

1.6. Charleston Peninsula Vs30 Designation

The seismic velocity of the upper 30 meters of soil (Vs30) was initially estimated using the USGS Global slope-based Vs30 model (Wald and Allen, 2007; Allen and Wald, 2009), found at: https://earthquake.usgs.gov/data/vs30/. The values derived range between 180 to 330 m/s. After looking at the overall seismic velocities for the Charleston Peninsula, a velocity of 255 m/s was determined. A Vs30 of 255 m/s falls within the range of a seismic "Site Class D" classification.

1.7. MCE Deterministic Analysis

1.7.1. General

USACE design guidelines utilize a Maximum Credible Earthquake (MCE) and an Operating Basis Earthquake (OBE). The MCE is defined as the greatest earthquake magnitude that can reasonably be expected to be generated by a specific source based on seismological and geological evidence. The MCE has no defined return period. According to ER 1110-2-1806, an OBE is based on the event with a 50% probability of occurrence during the 100-year service life of the project. This translates to a 144-year return period. The MCE is determined by a deterministic seismic hazard analysis, while the OBE is determined by a probabilistic seismic hazard analysis (see Section 5.8).

1.7.2. Charleston MCE and Background Earthquake

Deterministically derived MCE were developed using the methods described by Krinitzsky (1995) and in ECB 1110-2-6000. An Mw = 7.3 MCE is established for the Charleston Seismic Zone based upon the 1886 Charleston Earthquake event. The distance from the project site to the center of the MCE source zone is 10.00 km.

1.7.3. Ground Motion Prediction Equations and Source Attenuation

Ground motion prediction equations (GMPE) take into account bulk crustal seismic velocities and other components to attenuate ground motions as they are propagated to the project site. GMPE and attenuation curves are used to estimate the median site PGA that is propagated from an MCE epicenter or another designated seismic source. The GMPE of Boore and Atkinson (2006) is used because it was specifically developed for use within the eastern U.S. Furthermore, this GMPE was selected because it is readily available for use in open source, web-based ground-motion calculators. A +1 σ (standard deviation) was applied to the median PGA curves in order to account for uncertainty in assessing the MCE, and achieve the 84th percentile ground motion projection. Boore and Atkinson (2006) recommend that median ground acceleration values be multiplied by 10^ (log10 (ground acceleration) +0.3) to account for this uncertainty. Resultant ground motions for engineering consideration reflect Mean PGA+ 1σ. Figure 12 shows how the median PGA+1 σ is attenuated to the site from the epicenter.

1.7.4. OPENSHA Ground Motion Modeling and Attenuation Relationship Plotter

Ground motions were generated using OPENSHA, which is an open-source, web-based modeling and plotting program developed by Field et al. (2003): http://www.opensha.org/apps.

This web application is freely available through the USGS website and it is relatively easy to use. For the web application, the following inputs were used:

- A site seismic velocity (Vs30) of 255 m/s (site class D) was designated. The fault type was designated as "unknown" due to it being a deep crustal level feature. "Unknown" fault type yields the highest PGA, which is considered appropriate for conservatism.
- Charleston MCE was established at Mw = 7.3.
- X-axes were set to measure the shortest surficial distance to the surface rupture. Distances were set to 10.00 km for the Charleston MCE.
- Y axes were set to Median PGA.

1.7.5. Seismic Attenuation Curves

The median PGA output from OPENSHA was then plotted in Excel and the Median PGA +1 σ was calculated to generate the curve representing the +1 standard deviation or 84th percentile. Median PGA and +1 σ curves for the Charleston MCE is shown in Figure 12. The isoseismal contours 0.5g and 0.8g from the USGS seismic hazard map (Figure 2) are also plotted against the attenuation curves to compare the predicted ground motions for the project site by deterministic and probabilistic methods. The probabilistic site-specified PGA from Figure 8 is also plotted for reference. The distance from each epicenter, relative magnitude, and predicted attenuated PGA at the project site are given in Table 4.

Table 4: Seismic attenuation curves indicating median PGA and median PGA +1 σ at project site.

Seismic Source	Distance to Project	Max Credible Earthquake (Mw)	Boore & Atkinson, 2006 Median PGA at Project	Boore and Atkinson, 2006 Median PGA +1 σ at Project
Charleston, SC	10 km	7.3	0.28g	0.56g

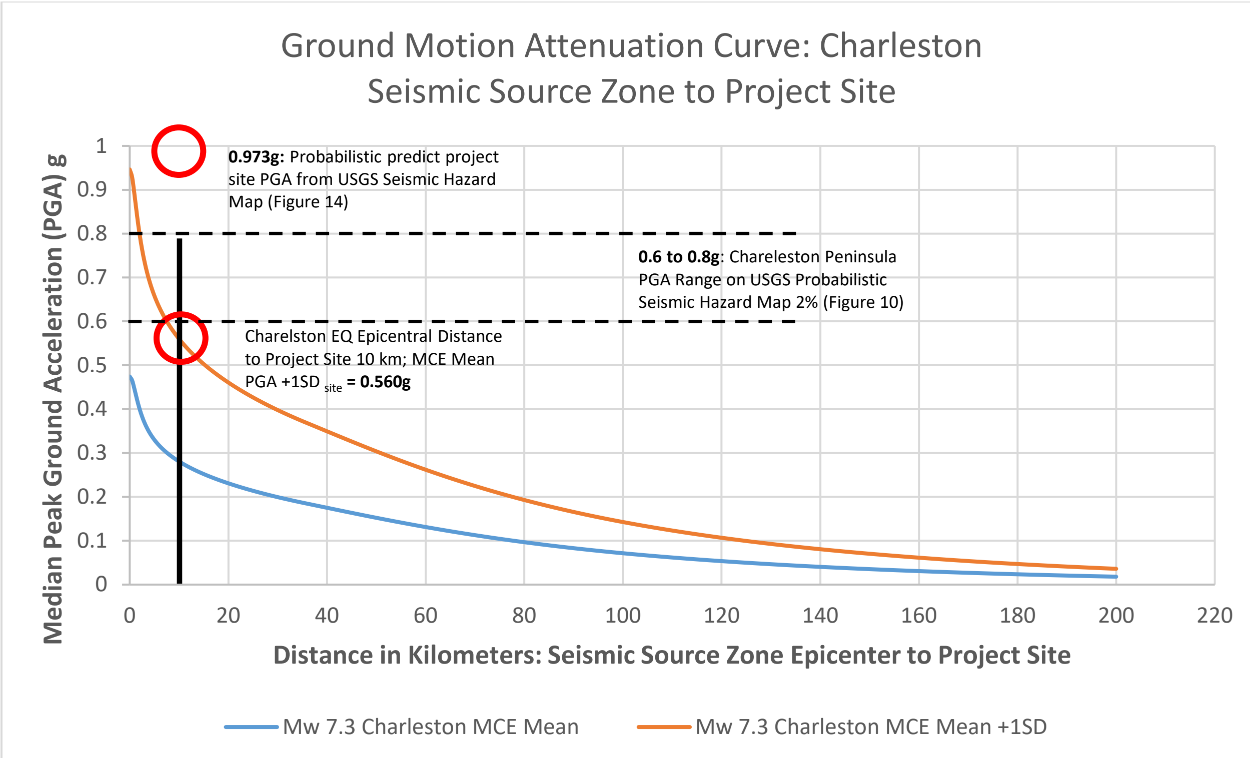

Figure 12: Attenuation curves for selected seismic source zones with respect to project site. Note: GMPE of Boore and Atkinson (2006) used to generate median PGA and median PGA +1σ. Curves generated using the attenuation relationship of Boore Atkinson (2006), with site Vs30 = 255 m/s. Mw Charleston MCE = 10 km distal source.

While attenuation curves are useful in understanding how ground motion is dampened with distance from the epicenter, site response is better understood by evaluating the spectral acceleration predicted for the site by the ground motion prediction equation. Furthermore, EM-1110-2-1806 mandates the evaluation of spectral periods between 0.2 and 5 seconds. The ground motion prediction equation of Boore and Atkinson (2006) was also used to predict the spectral wave for Charleston MCE events. The site response spectra was evaluated for an array of ground motions and is discussed in the following section.

1.7.6. Spectral Acceleration Ground Motion Response

OPENSHA was used to generate spectral acceleration data for the Charleston MCE, using the ground motion prediction equation of Boore and Atkinson (2006), for wave periods between 0 and 5 seconds. Input parameters for fault type, seismic velocity, event magnitude, and distance to epicenter are the same as discussed in Section 1.7.2. Spectral acceleration (SA) response curves for the Charleston MCE are shown in Figure 13. Table 5 shows the response spectra ordinates for the curves in Figure 13. A PGA of 1.261g at 0.30 sec period is selected as the design earthquake for follow-on liquefaction and stability analyses. The deterministically derived design PGA of 1.261g is considered to be conservative and agrees well with current USGS probabilistic seismic hazard data (see Figure 13) which yields a similar SA of 1.3972g at 0.2 seconds.

Table 5: Deterministic acceleration response spectra for median and median + 1σ ground motions generated from Charleston Seismic Zone and Background Earthquake.

Period (Seconds)	**Charleston Seismic Zone MCE Mw = 7.3**	**Charleston Seismic Zone MCE Mw = 7.3 Median PGA +1 σ**
	Acceleration g (m/s^2)	
0.05	0.387	0.772
0.10	0.466	0.930
0.20	0.616	1.230
0.30	0.632	1.261
0.50	0.576	1.151
1.00	0.355	0.709
2.00	0.183	0.366
3.00	0.120	0.240
4.00	0.087	0.174
5.00	0.070	0.141

Note: The spectral acceleration selected for design/liquefaction analysis is highlighted green.

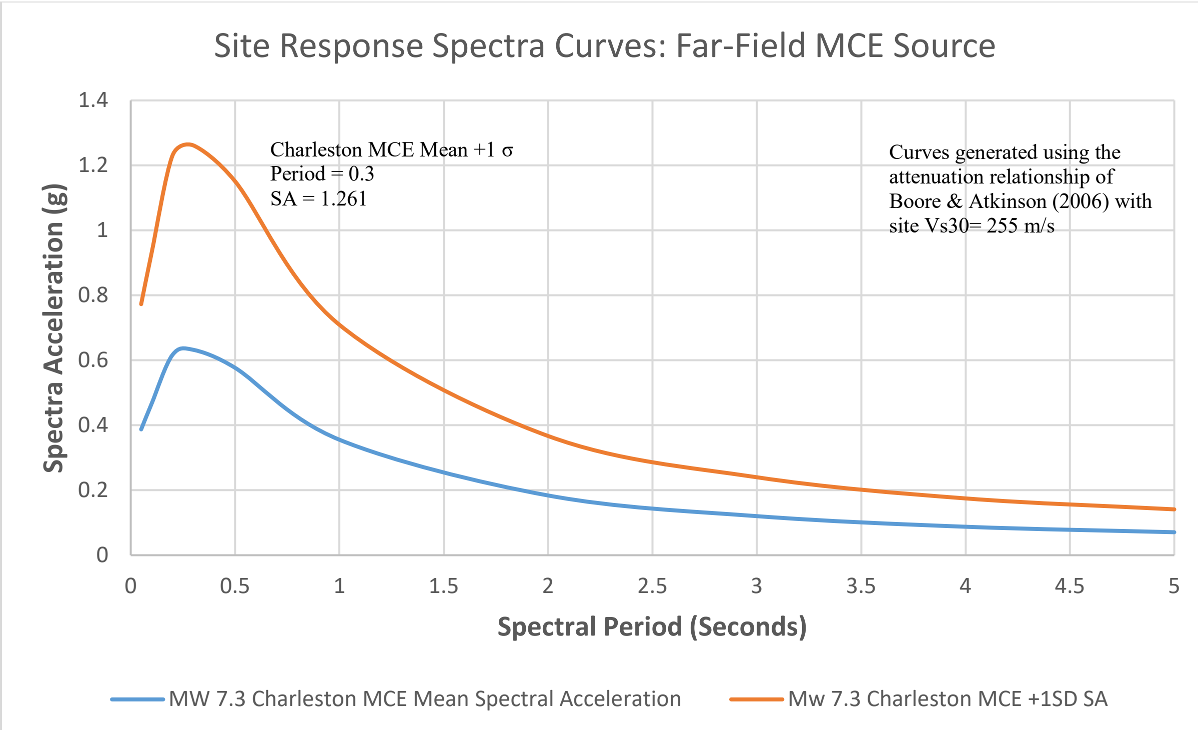

Figure 13: Deterministic acceleration response spectra curves for described ground motions.

1.7.7. Spectral Velocity Ground Motion Response

Spectral velocities from the Charleston MCE were interpolated using the following relationship: $V = V_0 + (a * t)$, where V = incremental ground velocity, V_0 = initial velocity, and t = period (sec). The relative velocities were calculated for the acceleration response spectra (see Table 5 and Figure 13) that were generated by the OPENSHA application using the ground motion prediction equation of Boore and Atkinson (2006). Table 6 contains the computed seismic velocities, the curves of which are plotted in Figure 14.

Table 6: Deterministic velocity response spectra for Charleston Seismic Zone (Toro et al., 1997; USGS, 2003; Boore and Atkinson, 2006).

Period (Seconds)	**Charleston Seismic Zone MCE Mw = 7.3**	**Charleston Seismic Zone MCE Mw = 7.3 Median PGA +1 σ**
	Velocity (cm/s)	
0.05	1.936	3.86
0.10	6.599	13.16
0.20	18.93	37.77
0.30	37.89	75.61
0.50	66.74	133.1
1.00	102.2	204.0
2.00	139.0	277.4
3	175.1	349.4
4	210.2	419.4
5	245.5	489.9

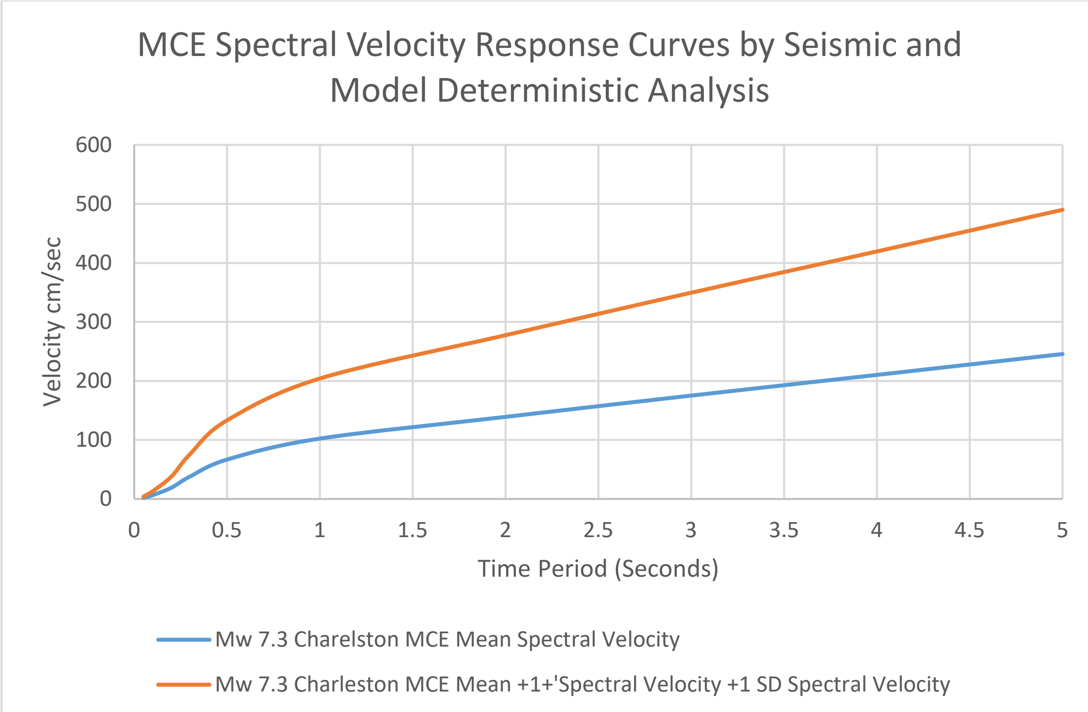

Figure 14: Deterministic velocity response spectra curves for described ground motions.

1.8. OBE Probabilistic Analysis

1.8.1. General and OBE Defined

The Operational Basis Earthquake (OBE) is defined in ER 1110-2-1806 as the earthquake that can reasonably be expected to occur within the service life of the project, typically a 50% probability of exceedance in 100 years (average return period of 144 years). The OBE is assessed using probabilistic methods that are informed by deterministic methods (see Section 5.7).

1.8.2. USGS Unified Hazard Tool Input Parameters

Probabilistic hazard characterization is based on existing USGS data by Frankel et al. (1996; 2002), and later revised by Petersen et al. (2015). Seismic hazard curves were generated using the USGS Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/). Input parameters utilized are nearly same as in Section 5.5, with exception to the return period as shown:

- USGS Probabilistic Seismic Hazard Map Edition: Dynamic conterminous U.S. 2014 (v4.1.1).
- Spectral Period: PGA, 0.2, 1.0, and 2.0 seconds evaluated.
- Latitude/Longitude Inputs: 32.787 Lat. / -79.937 Long.
- Time Horizon: Return period 144 years corresponding to a 50% in 100 years AEP.
- Site Class: Vs30 = 760 m/s (chosen for consistency with Section 1.5).

1.8.3. Hazard Response Spectrum Curves and OBE

Seismic hazard curves for the project site were generated for the PGA and spectral periods of 0.2, 1.0, and 2.0 seconds (Figure 15). The USGS Unified Hazard Tool utilizes seismic hazard curves to create the uniform hazard response spectrum (UHRS) curve shown in Figure 16. The UHRS curve is created (automatically by the tool) by selecting data points along each hazard curve corresponding to the 144-year return period. An OBE PGA of 0.0548g and an SA of 0.09g (at 0.2 second period) is derived utilizing the USGS Unified Hazard Tool.

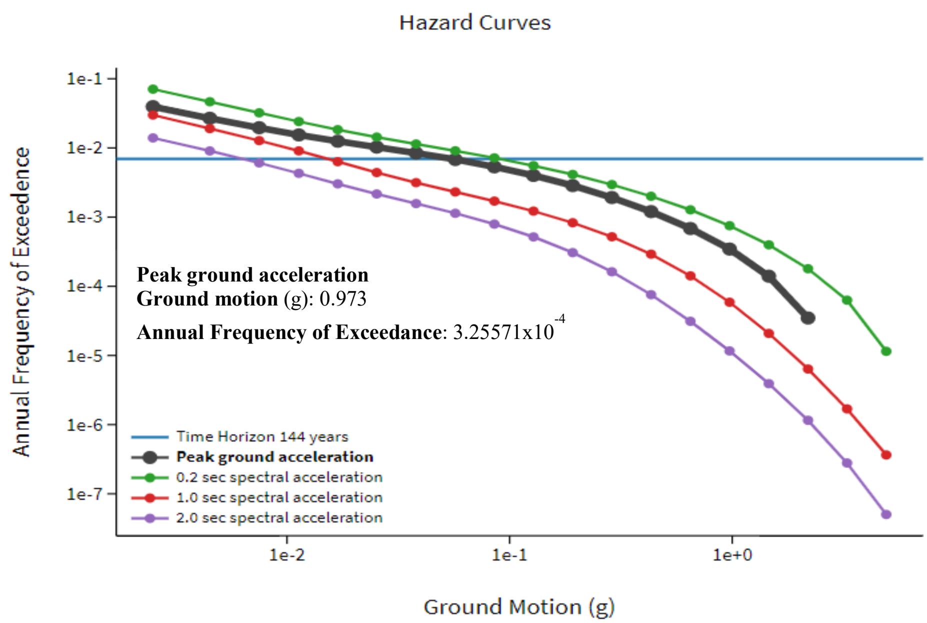

Figure 15: Site-specified seismic hazard curves showing ground motions for PGA and SA with 144-year return period.

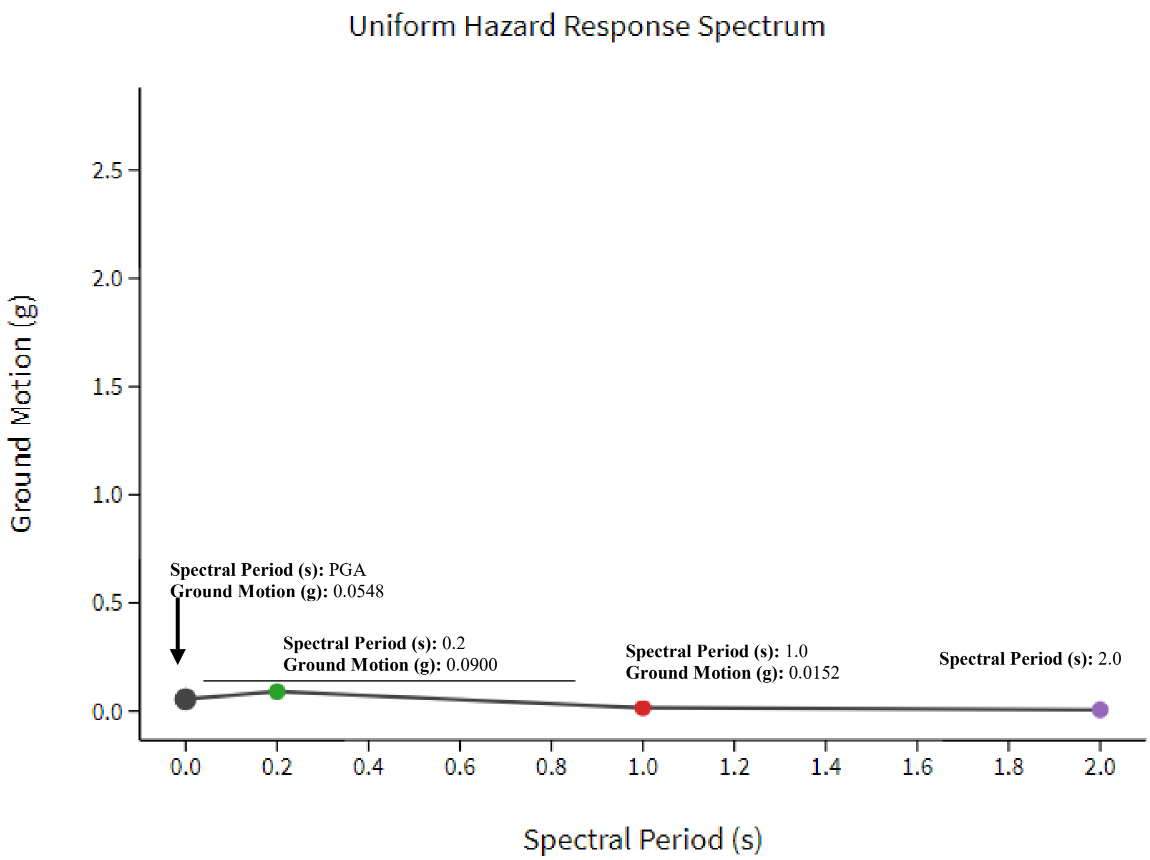

Figure 16: Site-specified uniform hazard response spectrum for the 144-year return period. PGA and SA Periods 0.2, 1.0, and 2.0 seconds shown.

1.9. Seismic Analysis Summary

1. The project site lies in an area that is subject to moderate to strong seismic activity. The largest earthquake recorded in the eastern U.S. occurred approximately 10 kilometers northwest of the project site. In accordance with ER 1110-2-1806, seismic ground motions must be accounted for in the seawall design. Deterministic methods, informed by probabilistic methods, were used to determine the design ground motion.

2. One ground motion was evaluated: Mw = 7.3 "MCE Charleston Earthquake". The ground motion prediction equation of Boore and Atkinson (2006) was used with OPENSHA software to evaluate the median and median +1 σ PGA and SA from this event. Comparison of attenuation, spectral acceleration, and spectral velocity curves reveal significant attenuation, spectral acceleration, and spectral velocity. The Charleston earthquake of 1886 should be utilized for Maximum Design Earthquake.

3. Figure 13 indicates the highest spectral acceleration being +1 σ spectral acceleration = 1.261g at a period of 0.3 seconds. This spectral acceleration corresponds to a 7.3 Mw

Charleston earthquake event, templated to occur within a radius of 25 kilometers from the site. An OBE PGA of 0.0548g and an SA of 0.09g at 0.2 second period is also designated for the project site for 144-year return period.

4. Figure 12 compares the peak ground acceleration (g) between the USGS seismic hazard map (Figure 2) and the probabilistic seismic hazard curve (Figure 8). Figure 2 indicates a range of 0.6 to 0.8g PGA while Figure 8 indicates a higher PGA of 0.973g with the greatest spectral period being 1.3972g at 0.2 spectral period (Figure 9). The higher ground motion of 1.3972g at 0.2 spectral period should be taken into account when designing.

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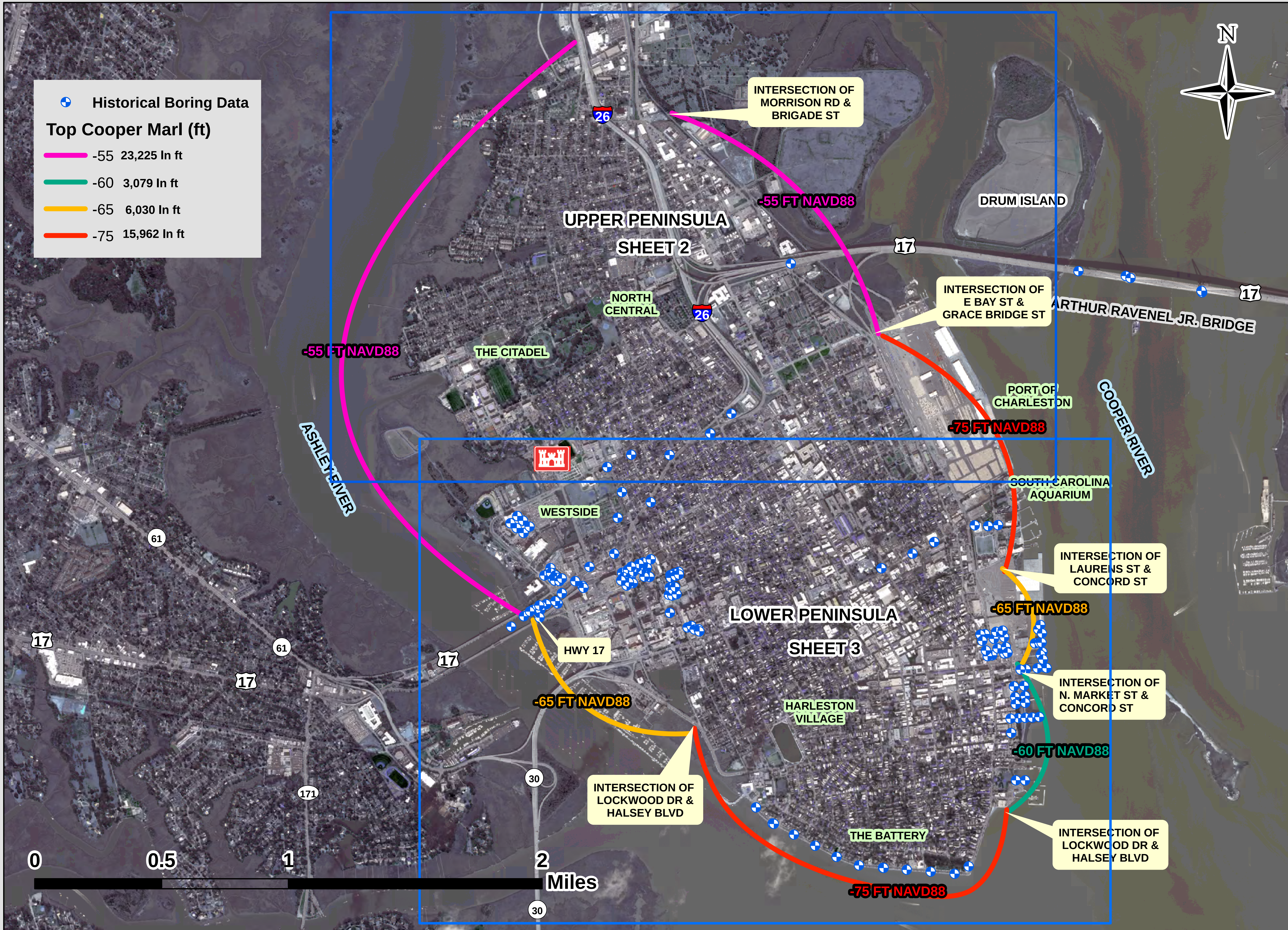

US Army Corps of Engineers
Wilmington District

Geotechnical Data Compiled By:
Stephen J. Fabian
Wilmington District, USACE
Map Date: APR 2020
Map File Name: Y:\Common\ECP\EG\CPS_CSRM\GIS

Horizontal Datum: NAD83
Vertical Datum: NAVD 88, ft
Imagery: USACE, 2010
Primary Map Scale: 1:22000

CHARLESTON, SOUTH CAROLINA PENINSULA SHEET 1 OF 3

US Army Corps of Engineers
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Vertical Datum: NAVD 88, ft
Imagery: USACE, 2010
Primary Map Scale: 1:12500

CHARLESTON, SOUTH CAROLINA UPPER PENINSULA

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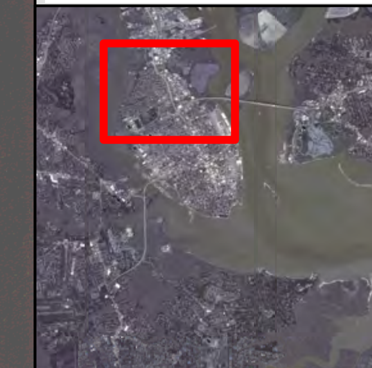
US Army Corps of Engineers
Wilmington District



Geotechnical Data Compiled By:
Stephen J. Fabian
Wilmington District, USACE
Map Date: APR 2020
Map File Name: Y:\Common\ECP\EG\CPS_CSRM\GIS

Horizontal Datum: NAD83
Vertical Datum: NAVD 88, ft
Imagery: USACE, 2010
Primary Map Scale: 1:12500

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