Waccamaw River,
Horry County, South Carolina
Flood Risk Management Study
Draft Integrated Feasibility Report and Environmental Assessment

Engineering Appendices A1-A5

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1.0 Introduction

This hydrology and hydraulics appendix serves as documentation of the engineering evaluation process for the U.S. Army Corps of Engineers (USACE) Waccamaw River, Horry County, SCFeasibility Study. This flood risk management study was authorized based on historical and potential future risks to life and property within the Waccamaw River watershed caused by the occurrence of flooding. There has been historical documentation of severe overland flooding along the Waccamaw River and its numerous tributaries. The purpose of the federal action is to improve life safety and reduce economic damages in the study area through development of assessed solutions that achieve federal interest. This appendix describes the development of existing conditions (EC) and future without project (FWOP) conditions in addition to the formulation, refinement, and design of structural study measures and alternative plans. Formulation of nonstructural measures is also included. This Engineering Appendix is in accordance with Engineering Regulation (ER) 1110-2-1150 (USACE, 1999), provides assumptions of underlying hydrology and hydraulic uncertainty in accordance with ER 1105-2-101 (USACE, 2019), and includes an assessment of climate change of the study area and potential effects of such change by Engineering and Construction Bulletin (ECB) 2018-14 Revision 1(rev. 2, 2022)(USACE, 2018, 2022).

1.1 Vertical Datum

All elevations in this repot are referenced to the North American Vertical Datum of 1988 (NAVD88) unless otherwise noted.

2.0 Basin Overview

2.1 Location

This area of interest covers the Waccamaw River and its tributaries from the South Carolina state line to its confluence with the Pee Dee River. Horry County (the non-federal sponsor) is situated within South Carolina's coastal plain and is bordered by North Carolina to the north and the Atlantic Ocean to the east. Water is a prominent natural feature throughout Horry County, encompassing 10 percent of the County's almost 1300 square miles.

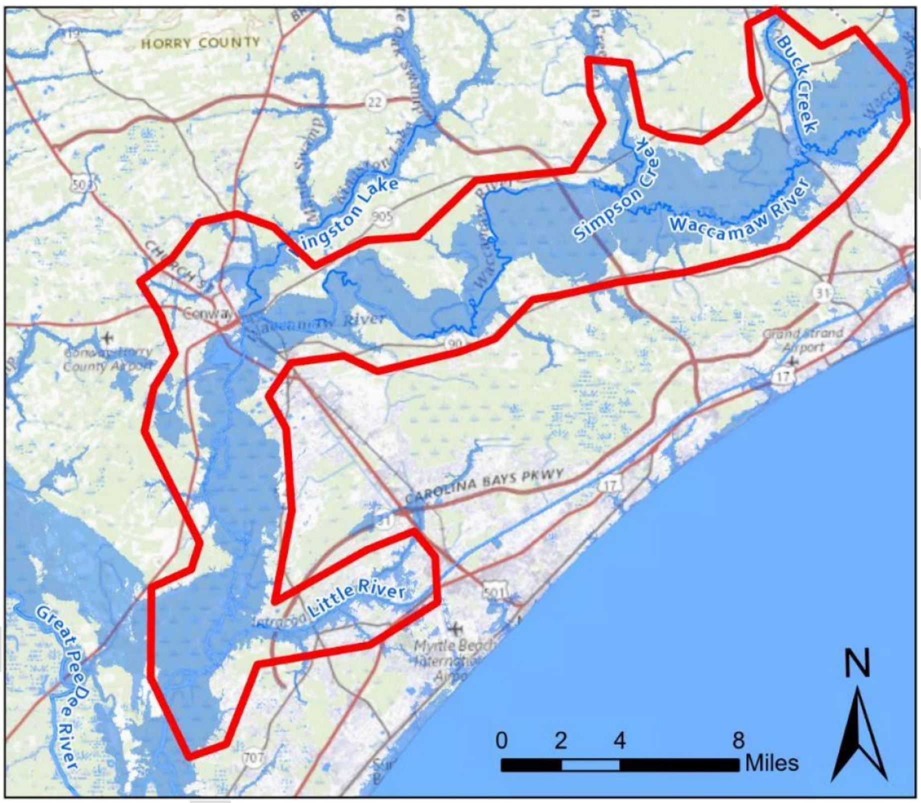


Figure 1: Waccamaw River FRM study area.

Two primary sources control flooding within the area of interest, the Waccamaw River and the Pee Dee River. Expanding the study area beyond the area of interest was necessary to establish the hydrologic input parameters for modeling. The Pee Dee River watershed covers parts of North Carolina, South Carolina, and Virginia and is approximately 12,000 square miles in size. The river is about 230 miles long and runs from North Carolina to the Atlantic Ocean in South Carolina. It is one of the largest river systems in South Carolina. The Waccamaw River watershed is in the southeastern end of the Pee Dee River watershed. It is within the Atlantic coastal plain and is approximately 1,100 square miles in size. The upper reaches are characterized by shallow and slow-moving wetland flow, while the lower reach includes a navigable section up to Conway, SC. Factors influencing its hydrologic response include soil saturation, Lake Waccamaw, agriculture, stream channelization, and urbanization. Figure 1 shows a portion of the Pee Dee River watershed boundary (upstream of the Black River confluence), the Waccamaw River watershed boundary, and the project area of interest.

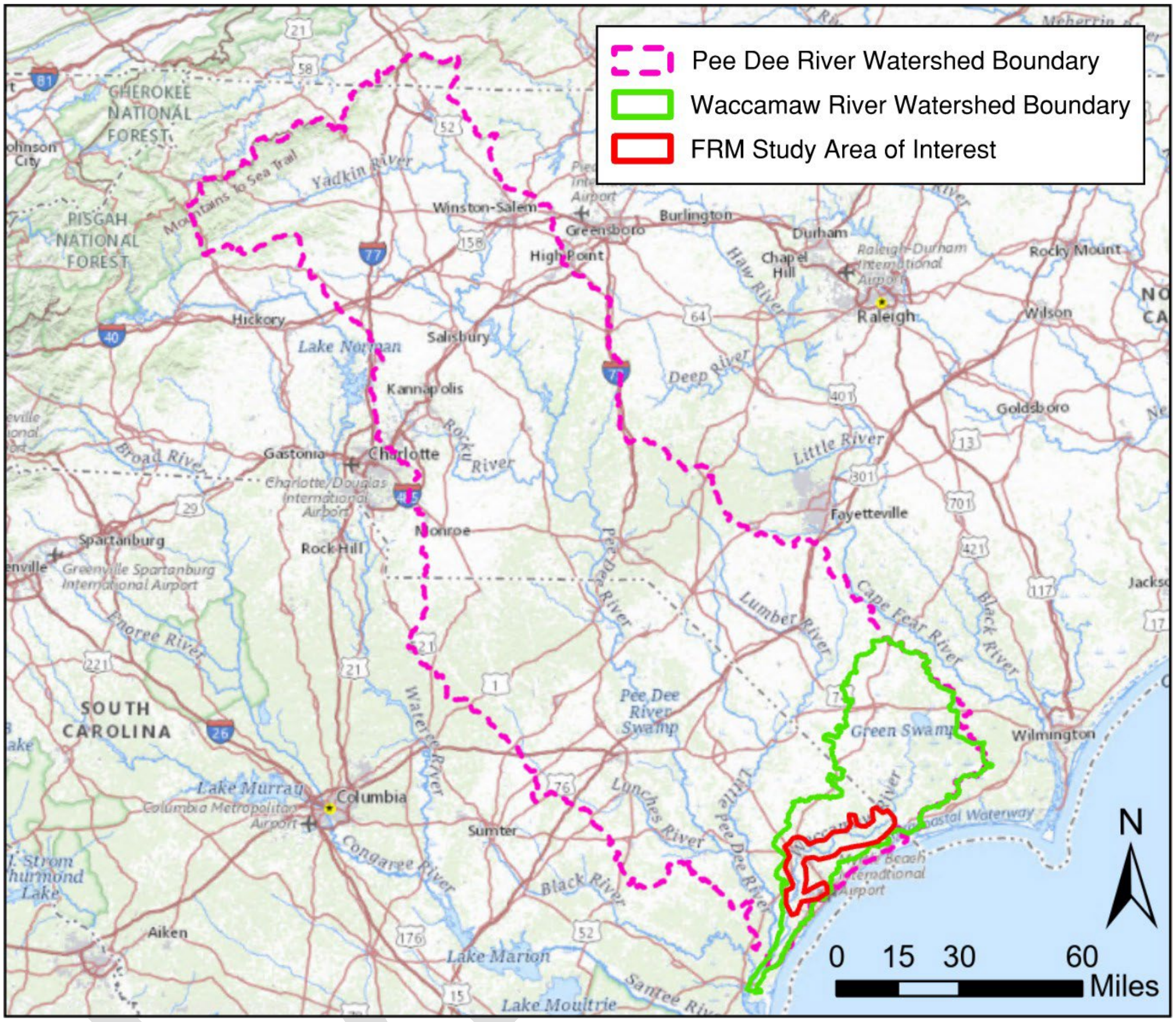


Figure 2. Waccamaw and Pee Dee River Watersheds and Area of Interest

Figure 23 shows the HUC 8 and 10 outlines for Waccamaw River Watershed. This image provides an overview of the watershed and sub watersheds that spans across North Carolina and South Carolina.

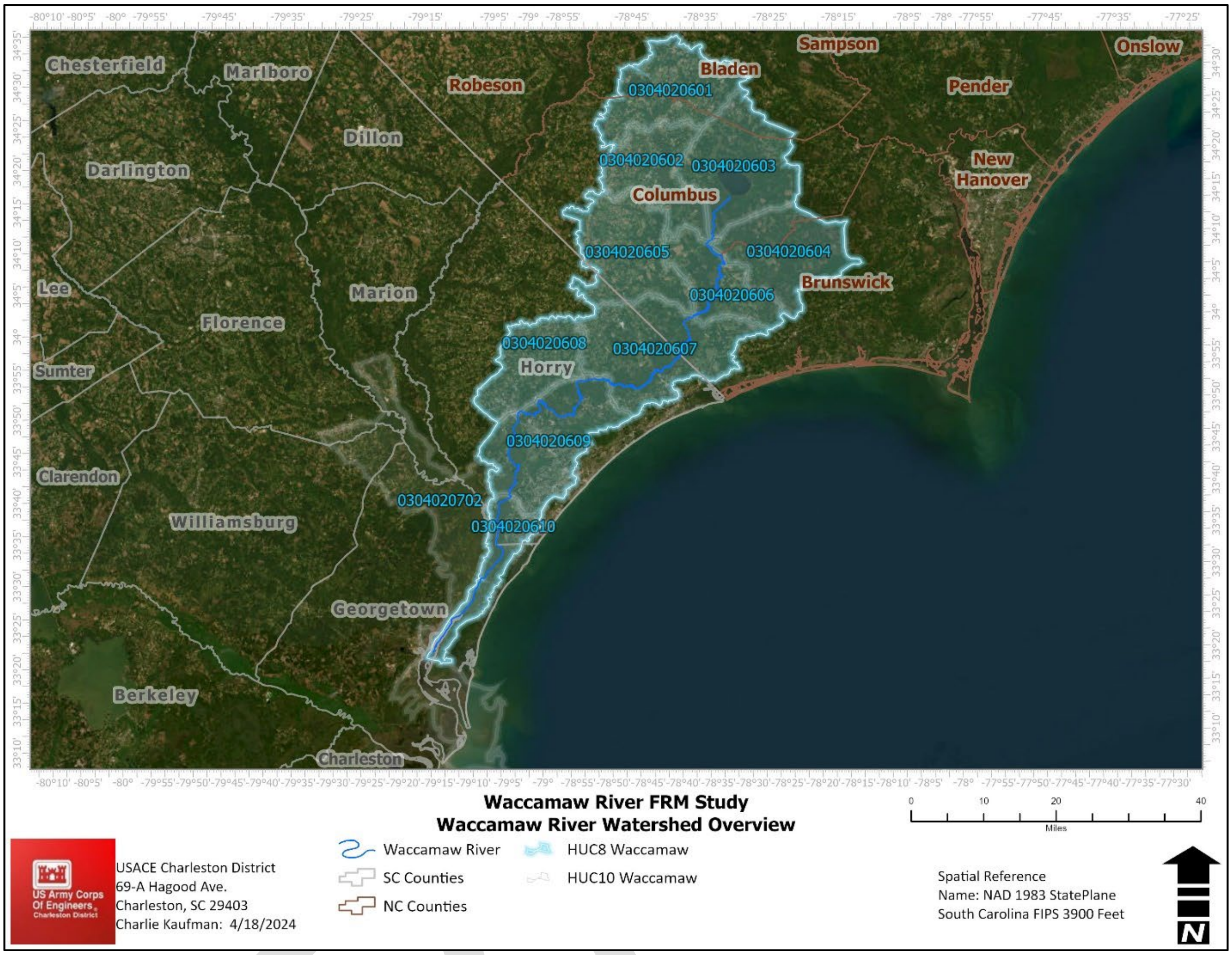


Figure 3. Waccamaw River watershed and sub watersheds

2.2 Flood Risk Management Infrastructure

There are no impoundments along the Waccamaw River, and one oxidation pond in Conway which is no longer in operation. Dams with an assigned Hazard Potential Classification (Low, Significant, or High) from the National Inventory of Dams (https://nid.sec.usace.army.mil/) are shown in Figure 2. There are no registered dams on the NID along the Waccamaw River. Lake Busbee in Conway is still indicated on the NID, but it is no longer impounding water and no longer registered to the NID.

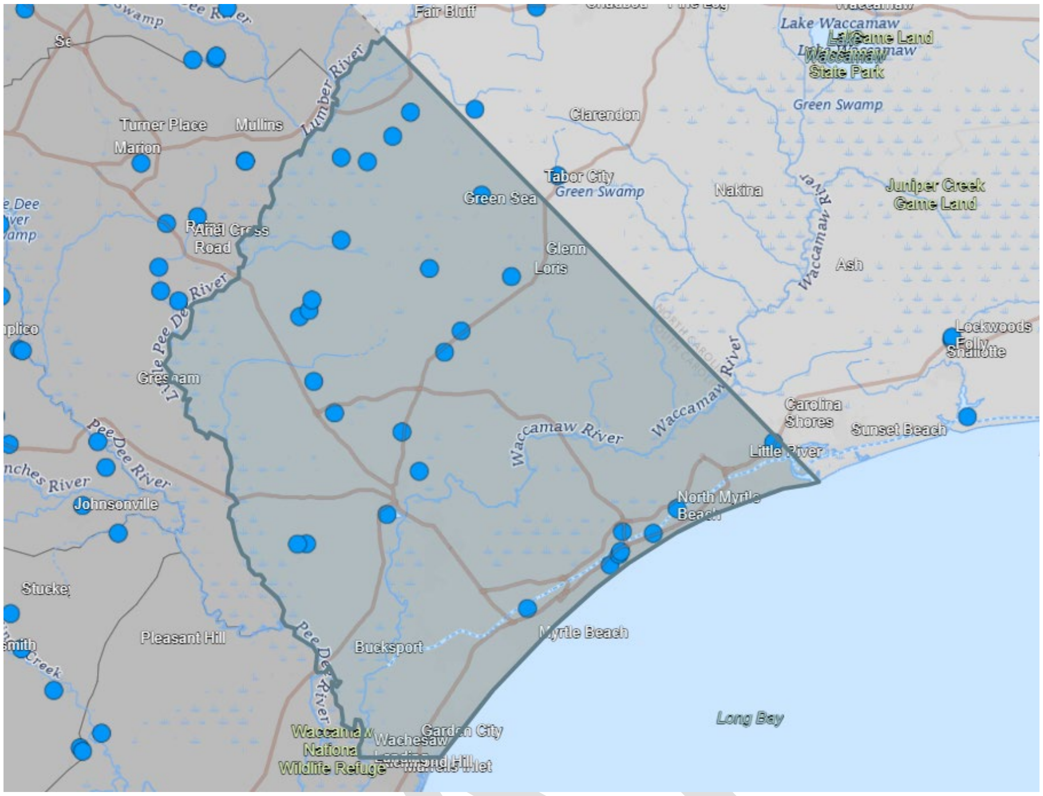


Figure 4. National Inventory of Dams locations within Horry County

2.3 Stream Characteristics

Horry County is the largest county by land area in South Carolina. It comprises 1,255 square miles of mostly flat topography, with elevations that range up to approximately 150 feet above sea level. Horry County is dominated by the Little Pee Dee and Waccamaw River watersheds. These watersheds, as well as many others in North and South Carolina, are part of the larger Yadkin/Pee Dee River Basin. Horry County is situated near the lowest point in this watershed before water exits the system through Winyah Bay. The headwaters of the Yadkin/Pee Dee River Basin begin in the Appalachian Mountains of North Carolina, hundreds of miles upstream of Horry County. The County rests in a large lowland basin that receives water from over 14,000 square miles of land and almost 6,000 miles of streams and rivers. The system flows through 21 counties and almost 100 municipalities, many of which are highly populated. As this larger region grows and attracts new residents, increased tree cutting and clearing and the loss of natural permeable surface to development increase the footprint of the floodplain and reduce the storage capacity throughout the system.

The rivers in Horry County flow southward on a primarily gradual slope through forested swamps and expansive floodplains. These rivers widen and merge with downstream rivers and have meandered over time to create the current coastal floodplain. Part of the floodplain is designated as the Waccamaw National Wildlife Refuge, but many homes and businesses also sit within this area. The flat topography and low elevation allow water to crest the banks during periods of high flow, filling up the adjoining creeks and tributaries which overflow into the larger floodplain.

The relatively flat conditions and the confluence of multiple waterways can cause floodwaters to “back-up” in times of high flow. Although the County’s stormwater ordinance requires reduced run-off rates from development, new development builds up and fills the land and creates additional impervious surfaces, increasing run-off and reducing the storage capacity of the floodplain and surrounding lands.

According to the South Carolina Department of Environmental Services’ (SCDES) general description of the Waccamaw River Watershed 03040206-09, the watershed consists primarily of the Waccamaw River and its tributaries from Simpson Creek to Socastee Creek (AIWW) and is primarily located within Horry County. The watershed occupies 136,304 acres of the Lower Coastal Plain and Coastal Zone regions of South Carolina. Land use/land cover in the watershed includes: 48.94% woody wetland, 16.9% developed land, 17.4% forested land, 5.21% agricultural land,1.41% Emergent Herbaceous wetland, 2.11% water, and 0.27% barren land. The mean base slope is 1.32%, Mean Basin Elevation is 52.8ft, Mean Annual Precipitation of 51.3 inches, an increase in impervious percentage by 1.4%, and 48.4% of the watershed is area of storage, with lakes, ponds, rivers and wetlands.

Horry County contains a portion of the Atlantic Intracoastal Waterway (AIWW), which was constructed by the United States Army Corps of Engineers (USACE) in the 1930s to provide a safe transportation route for commerce along the Eastern Seaboard. This tidally influenced waterway runs parallel to the Atlantic Ocean and is a significant recreational and commercial asset to the community. The AIWW connects to the Atlantic Ocean near the border with North Carolina through the Little River Inlet and continues south for over 70 miles before reconnecting with the ocean at Winyah Bay. This portion of the Waccamaw River accepts drainage from its upstream reaches along with Jones Big Swamp (Boggy Swamp, Horse Savannah, Watts Bay), Stanley Creek (Beaverdam Swamp, Big Swamp), Tilly Swamp (Bare Bone Bay, Cane Bay, Tiger Bay, Buck Bay, Long Branch), Round Swamp, and McCoy Bay. Dam Swamp enters the river next followed by Steritt Swamp (Skinners Swamp) East Prong, (South Prong). The river then flows past the City of Conway and accepts drainage from Bear Swamp (Butler Swamp, Willow Springs Branch, Busbee Lake), Pitch Lodge Lake, Cox Ferry Lake, and Thorofare Creek. Wadus Lake connects Busbee Lake to the river. Gravely Gully and Halfway Swamp (Big Branch) enter the river next, followed by Old Womans Lake, Big Buckskin Creek, and Peachtree Lake. Socastee Swamp and the AIWW (Folly Swamp) merge near the Town of Socastee to form Socastee Creek and flows into the Waccamaw River. Enterprise Creek connects the Waccamaw River and Socastee Creek just upstream of their confluence. There are a total of 226.2 stream miles and 477.1 acres of lake waters in this watershed.

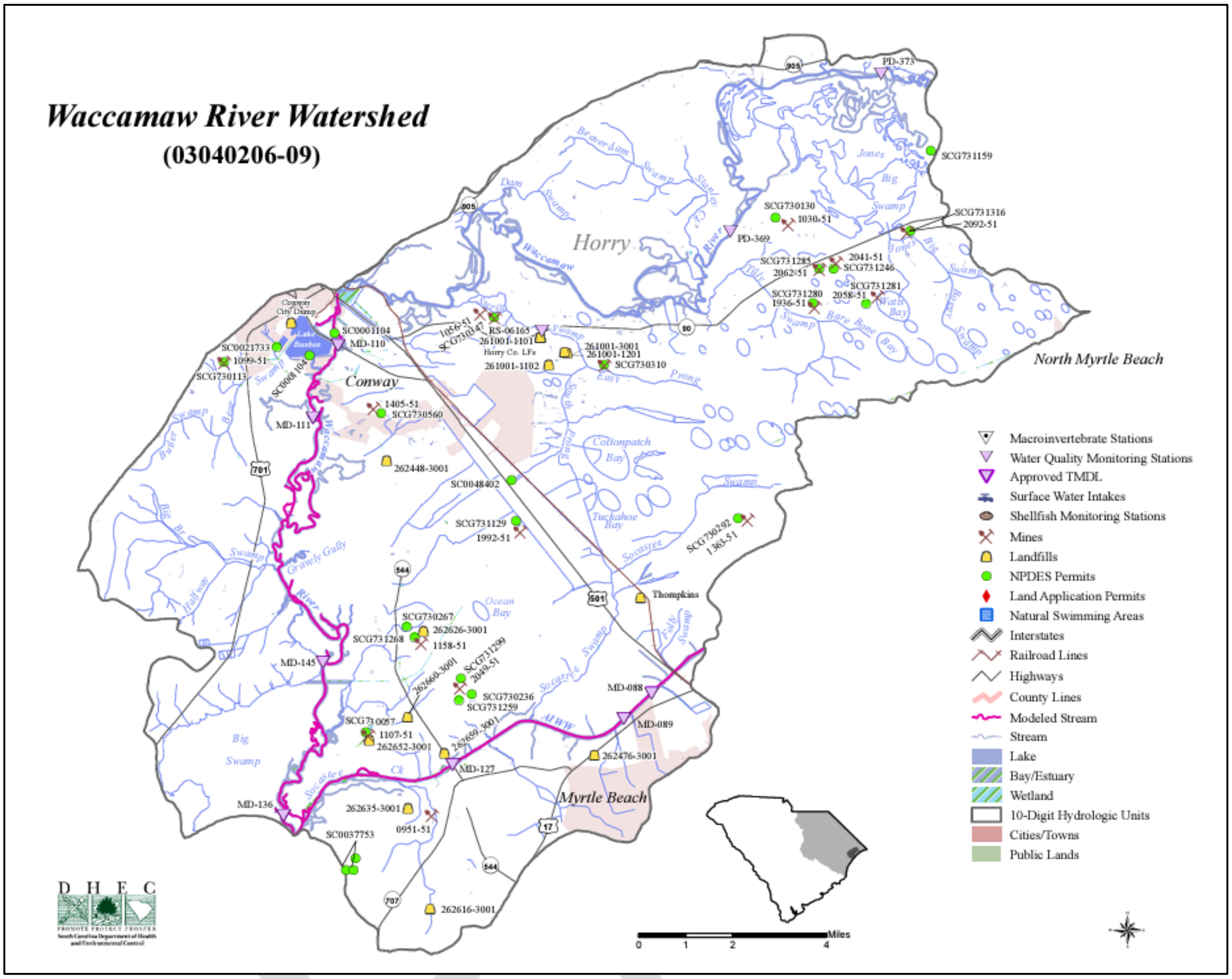


Figure 5. Waccamaw River Watershed location major highways, Lakes/bays and wetlands (SCDES 2023)

Table 1. List of Streams and corresponding drainage areas

Stream	Drainage Area (sq mi)
Waccamaw River at Freeland, NC	680
Buck Creek near Longs, SC	46.9
Waccamaw River Near Longs, SC	1100
Waccamaw River at SC-22 Below Longs, SC	1230
Waccamaw River Above Conway, SC	1250
AIW at Myrtlewood Golf Course at Myrtle Beach, SC	98.9
AIW At Highway 544 at Socastee, SC	771
Waccamaw River at Conway Marina	1440
Crabtree Swamp at Conway, SC	18.9
Waccamaw River at Bucksport, SC	1580
Waccamaw River Near Pawleys Island, SC	1620
Waccamaw River NR Hagley Land, NR Pawleys, SC	1640
PeeDee River at Highway 701 NR Bucksport, SC	14100

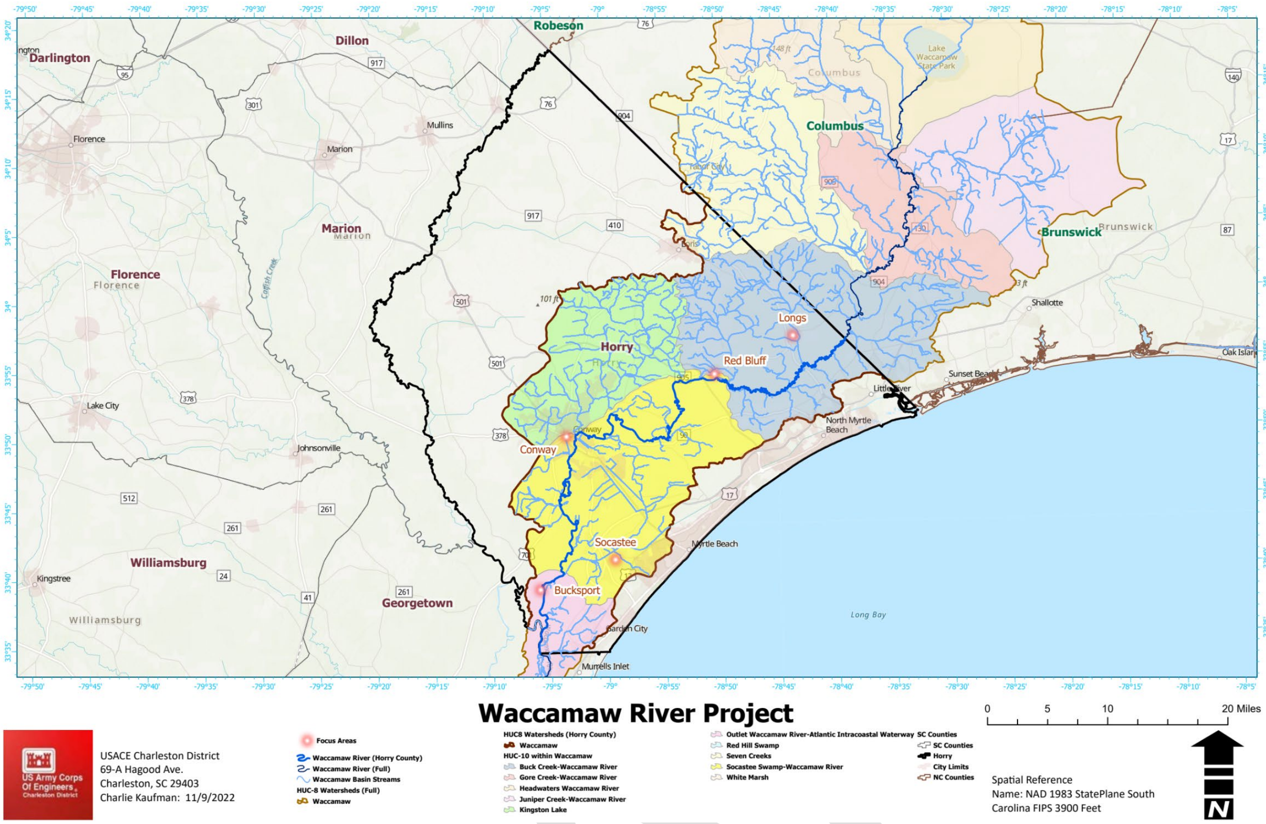


Figure 6: HUC 10 Watershed map for Waccamaw River with the study areas identified.

2.4 Land Cover

The most current (2019) National Land Cover Database (NLCD) for the Waccamaw River basin is shown in Figure 7. It provides a raster of descriptive land cover types at a 30-meter resolution and enables hydrologic characterization at a subbasin-level. Review of the dataset revealed physiographic trends distinct to the upper, middle, and lower portions of the basin.

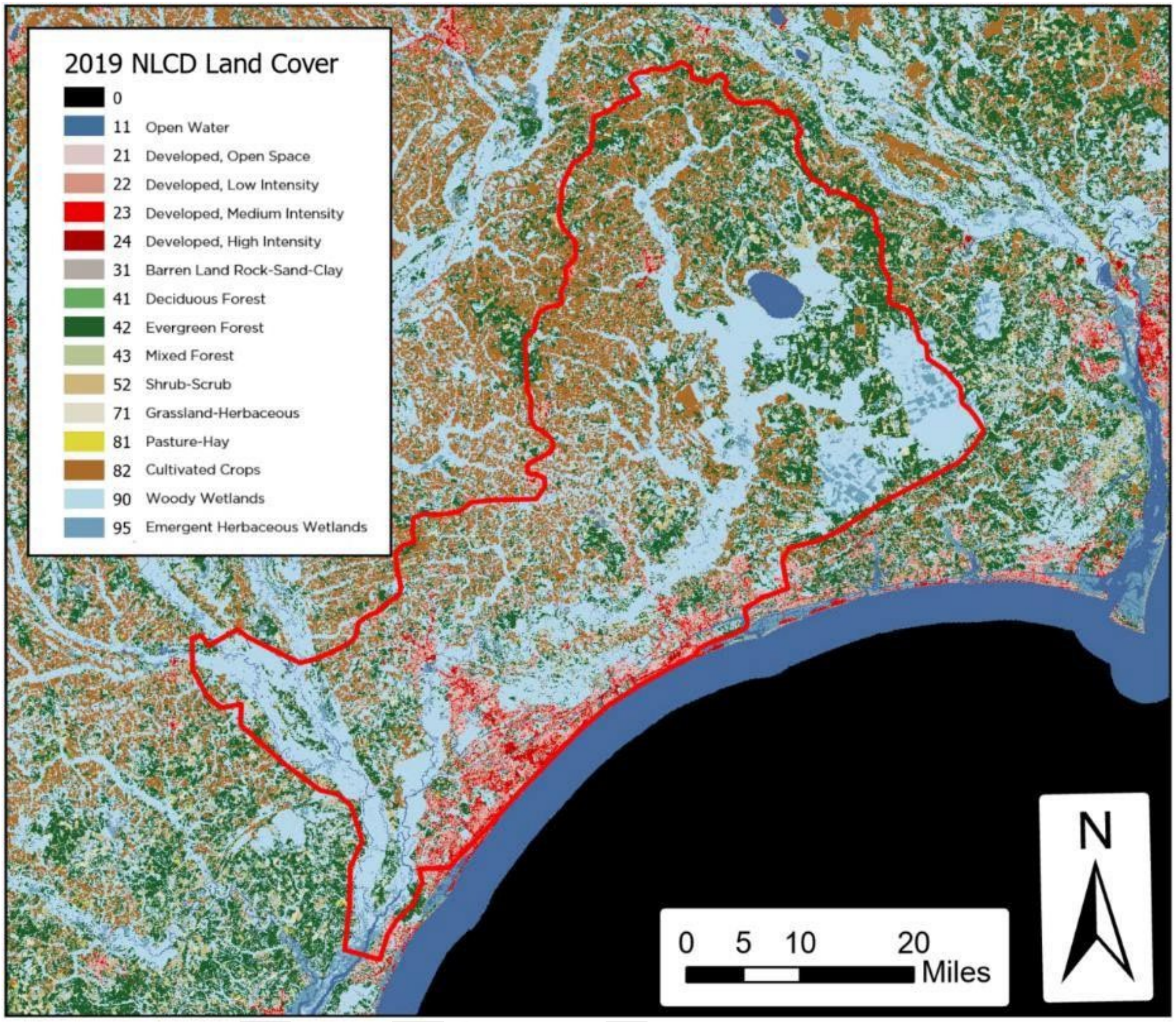


Figure 7. NLCD 2019 map of the Waccamaw River Watershed

In the Waccamaw River watershed over 50% of the land cover indicates some type of surface water storage such as lake, pond, river or wetland, as seen in Table 2.

Table 2. NLCD 2019 Land Cover Type Breakdown within the Waccamaw River Basin

Land Cover Type	Percentage of Total River Basin Area
Barren Land	0.27%
Cultivated Crops	5.21%
Deciduous Forest	0.18%
Developed High Intensity	0.55%
Developed Low Intensity	5.23%
Developed Medium Intensity	2.26%
Developed Open Space	8.86%
Emergent Herbaceous Wetlands	1.41%
Evergreen Forest	17.44%
Grassland/Herbaceous	3.00%
Mixed Forest	0.39%
Open Water	2.11%
Pasture/Hay	0.74%
Shrub/Scrub	3.41%
Woody Wetlands	48.94%

2.5 Climate

The Waccamaw River is a 140-mile-long river, located in southeastern North Carolina and eastern South Carolina in the flat Coastal Plain. It drains an area of approximately 1,110 square miles (2886 km²) in the coastal plain along the eastern border between the two states into the Atlantic Ocean. Along its upper course, it is a slow-moving, blackwater river surrounded by vast wetlands, passable only by shallow-draft watercraft such as canoe. Along its lower course, it is lined by sandy banks and old plantation houses, providing an important navigation channel with a unique geography, flowing roughly parallel to the coast.

The flow enters South Carolina and flows southwest across Horry County, past Conway. Near Burgess, it is joined from the northwest by the Great Pee Dee River, which rises in north central North Carolina. It continues southwest, separated from the ocean by only five miles (8 km) in a long tidal estuary. The long narrow point of land along the ocean formed by the lower river is called Waccamaw Neck. At Georgetown it receives the Black River (South Carolina) from the north, then turns sharply to the southeast and enters the ocean at Winyah Bay, approximately five miles (8 km) north along the coast from the mouth of the Santee River. Inland communities across the state are at risk from flooding due to extreme precipitation throughout the entire year. The Waccamaw River basin has a temperate climate with moderate winters and warm humid summers. Rainfall is well distributed throughout the year; however, rainfall is greatest near the coast, and decreases as the terrain transitions from Coastal Plain to Piedmont regions. The average annual precipitation over the Waccamaw River basin ranges from about 48 inches near Conway, SC up to 54 inches near Bucksport, SC. Rainfall is generally well distributed throughout the year, though it is greatest during the late spring to early fall when heavy localized rainfall and hurricanes are the most prevalent. The maximum monthly rainfall averages about 7 inches and occurs during July, whereas, the driest month is November with an average rainfall of 3.1 inches (NACSE, 2021).

Storm occurrences in the Waccamaw River basin are typically in the form of thunderstorms, northeasters, and hurricanes. The most severe floods of record over the basin have been associated with hurricanes. South Carolina lies in the path of tropical hurricanes as they move northerly from their origin north of the Equator in the Atlantic Ocean. These hurricanes usually occur in the late summer and autumn and have caused the heaviest rainfall and largest floods through the basin. These extreme hurricane events are characterized by heavy and prolonged precipitation. Flooding in the project area primarily results from; extensive rainfall throughout the year; multi-day rainstorms leading to saturated soils; warm Atlantic Ocean which is getting warmer contributing to the increased rainfall; and increase in intensity and frequency of Hurricanes. These climate factors are the primary cause of floods that damage infrastructure in the project area.

2.6 Topography

The Waccamaw River Basin lies entirely plain of North Carolina and South Carolina. It is approximately 161 miles long and 35 miles wide at its widest point. The total drainage area is 1,520 square miles, of which 483 are in South Carolina and 1,037 are in North Carolina. The Coastal Plain Unit is a compilation of wedge-shaped formations that begin at the “Fall Line” and dip towards the Atlantic Ocean with ground surface elevations typically less than 300 feet. The land to the southeast of the “Fall Line” is characterized by a gently downward sloping elevation (2 to 3 feet per mile) as it approaches the Atlantic coastline. The Coastal Plain Unit is divided into three subunits; the project area is contained in the Lower Coastal Plain. The Surry Scarp (-SS-)

separates the Lower Coastal Plain from the Middle Coastal Plain. The Surry Scarp is a seaward facing scarp with a toe elevation of 90 to 100 feet.

The Waccamaw, and many other streams that flow parallel to the coast, were probably determined by the position of lagoons, bays, or sounds that lay back of sand spits or barrier islands and that were drained by the lowering of sea level. Elevations in the basin range from 120 feet above mean sea level (msl) in the upper reaches of the basin to 50 feet msl in the vicinity of the North Carolina-South Carolina state line, and five feet msl near the mouth of the Waccamaw. Topography of the watershed varies from nearly level to gently sloping, with the sloping areas being, for the most part, adjacent to the river flood plain and along the tributaries. The flood plains of the river and many tributaries are broad and flat and subject to frequent and prolonged overflow (SCDOT Design Manual 2019).

2.7 Geology

In South Carolina the Piedmont Unit is separated from the Coastal Plain Unit by a “Fall Line” that begins near the Edgefield-Aiken County line and traverses to the northeast through Lancaster County. The Fall Line is an unconformity that marks the boundary between an upland region (bed rock) and a coastal plain region (sediment). The Waccamaw River Basin lies entirely in the lower coastal plain. It extends across five geological terrace formations which are of marine origin, having been formed by the advancement and recession of the ocean waters at different periods. These terraces are the youngest geological formations in the two states and are separated largely according to elevation along with the material and structural development of the soil.

The Coastal Plain is underlain by Mesozoic/Paleozoic basement rock. This wedge of sediment is comprised of numerous geologic formations that range in age from the late Cretaceous Period to Recent. The sedimentary soils of these formations consist of unconsolidated sand, clay, gravel, marl, cemented sands, and limestone that were deposited over the basement rock. The basement rock consists of granite, schist, and gneiss similar to the rocks of the Piedmont Unit. The thickness of the Coastal Plain sediments varies from zero at the “Fall Line” to more than 4,000 feet at the southern tip of South Carolina near Hilton Head Island. The thickness of the Coastal Plain sediments along the Atlantic coast varies from ~1,300 feet at Myrtle Beach to ~4,000 feet at Hilton Head Island. The sediment thicknesses in the project area range from ~900-1,700 ft. Predominantly, sediments lie in nearly horizontal layers; however, erosional episodes occurring between depositions of successive layers are often expressed by undulations in the contacts between the formations.

The vertical stratigraphic sequence overlying the basement rock consists of unconsolidated Cretaceous, Paleogene, Neogene, and Quaternary sedimentary deposits. The surface deposits of the Lower Coastal Plain were formed during the Quaternary Period that began approximately 1.6 MYA and extends to present day. The Quaternary Period can be further subdivided into the Pleistocene Epoch (1.6 MYA to 10 thousand years ago) and the Holocene Epoch (10 thousand years ago to present day). The Pleistocene Epoch is marked by the deposition of the surficial soils, the formation of the Carolina Bays and the scarps found throughout the East Coast due to sea level rise and fall. Barrier islands and flood plains along the major rivers were formed during the Holocene Epoch (SCDOT Design Manual 2019).

The 2019 National Land Cover Database (NLCD) raster (Figure 4) and the SSURGO soil data (Figure 5) were utilized to develop the HEC-RAS infiltration and land cover layers. The HEC-RAS infiltration layer was used to calculate the rainfall losses and rainfall excess at every mesh cell during each timestep when rainfall was occurring. The infiltration layer uses a combination of land cover type and hydrologic soil group to determine the SCS curve number values. The SCS curve number values were assigned to the infiltration layer based on the values listed in this report's Approach and Methodology section. The HEC-RAS land cover layer was used to determine a roughness value to each mesh cell face for the hydraulic computations. Manning's roughness values were assigned to each land cover type as listed in this report's Approach and Methodology section.

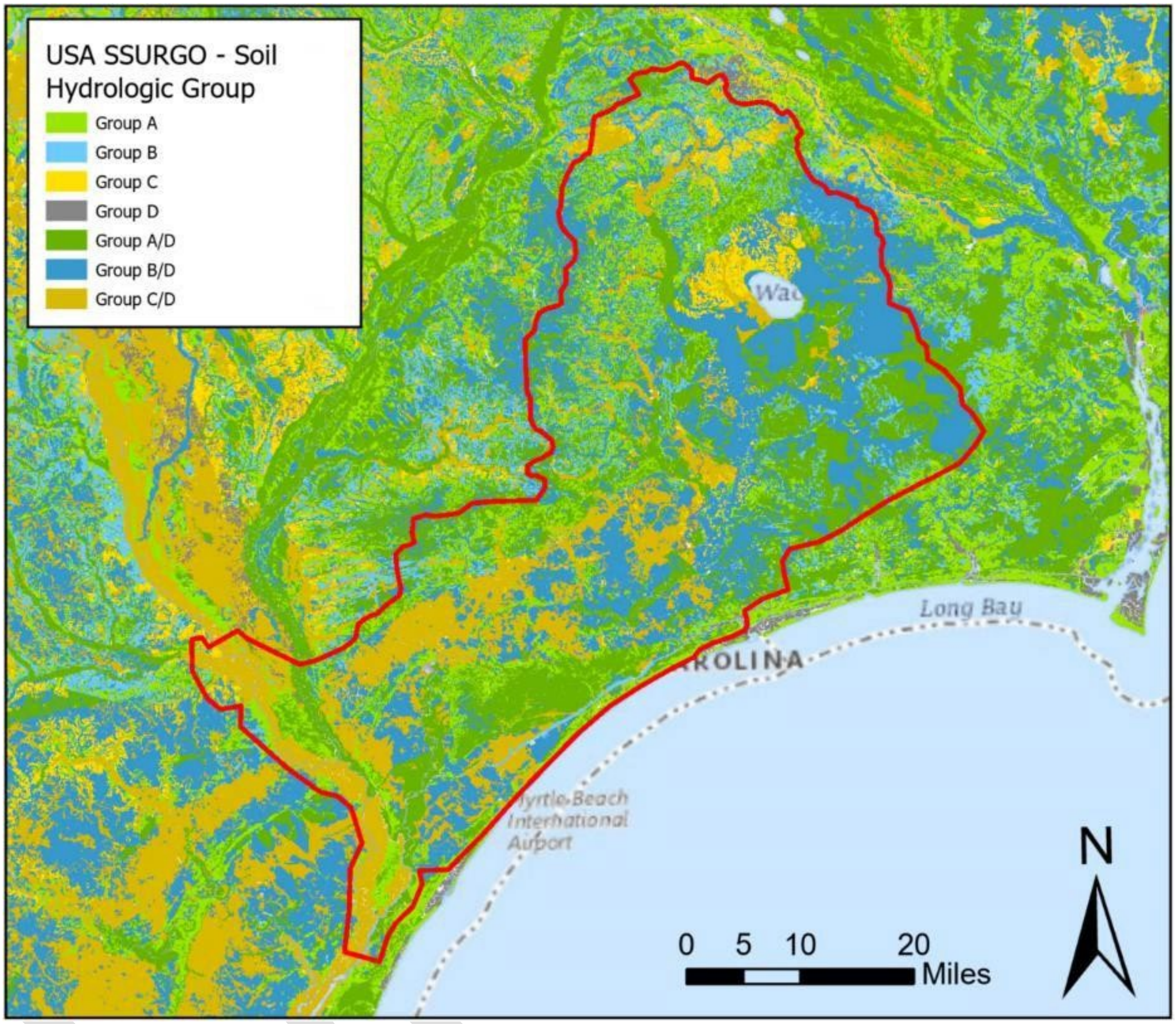


Figure 8. USA SSURGO data of the soil types in the Waccamaw River Watershed

2.8 Previous Studies

2.8.1 FEMA Flood Insurance Studies

Original Federal Emergency Management Agency (FEMA) Flood Insurance Studies (FIS) for counties within the Waccamaw River basin study area date back to the early 1990s. Including dates, original in 1988, with revisions 1991,1994, 1999, 2003 and most recently revised in 2021.These studies included hydrologic and hydraulic analyses for the majority of watercourses in the basin. Many of the initial FIS for these counties were prepared by USACE for FEMA under an inter-agency agreement. Streams were studied in varying degrees of detail due to the study's mixed rural and urban footprint and availability of engineering data.

2.8.2 USACE Studies

Studies listed below were the products of watershed-scale efforts directed towards identifying flood risk management improvements within the Waccamaw River basin. There were numerous technical reports for smaller, specific areas throughout the basin but were generally limited in scope.

2.8.2.1 Waccamaw River, North and South Carolina 1938.

This report investigated the need for flood protection (flood risk management), water supply, water-quality control, and reaction in the Waccamaw River basin. Local interest requested that the river channel be cleared of sunken logs and debris to accelerate runoff and that a diversion channel be provided near the North Carolina- South Carolina state line to tidewater in Little River Inlet, SC. The Chief of Engineers concluded that the anticipated benefits would be insufficient to justify the expenditure for the improvement.

2.8.2.2 Waccamaw River, North and South Carolina 1966.

This report was to review the water resource needs of the basin to present a general plan of development for water resources of the Waccamaw River Basin based on present and future needs. This report covers the needs for flood protection, navigation, water supply, pollution control, irrigation, and hydroelectric power. The improvements were primarily flood control improvements on the main stem of the Waccamaw River. The Chief of Engineers recommended that no improvements for flood control on the main stem of the Waccamaw River be undertaken by the Federal Government at this time.

2.8.2.3 Waccamaw River Basin Flood Damage Reduction Study Section 905(b) Analysis, 1981.

The purpose of the reconnaissance study was to evaluate the Federal interest in implementing solutions to flooding and other related water resource problems and needs along the Waccamaw River. Consideration of the following measures were assessed in this study; Channel Modification, Retention/Detention/Diversion. The study resulted in a recommendation for more development of the feasibility of these measures in order to understand the benefit and cost benefit ratio of the study.

2.8.2.4 Crabtree Swamp Aquatic Ecosystem Restoration Project, 2020.

This study of the feasibility of aquatic ecosystem restoration of Crabtree Swamp using Section 206 of the Continuing Authorities Program (CAP) was initiated in August 2015. The purpose of this study was to determine the feasibility of naturalizing the aquatic ecosystem processes in Crabtree Swamp and to improve survivability of resources of regional significance that have been identified. Documented manipulation of Crabtree Swamp goes back as far as the 1960s with a USACE project authorized under Section 208 of the Flood Control Act of 1954 (Table 1). The CAP Section 208 project allowed for snagging and clearing in a reach of Crabtree Swamp downstream of the current project footprint. Though CAP Section 208 projects are described as snagging and clearing of debris in a waterway, dredging was allowed in 7 miles of Crabtree Swamp upstream of Long Avenue. The dredging was performed in the entirety of the footprint of the current CAP 206 project. The purpose of the dredging was for flood control and drainage to minimize agricultural damages caused by a 3-year flood frequency. There was an anticipated 20-year project life after its completion in Fiscal Year 1966. Officially, the project was never de-authorized (USACE, 1982).

2.8.3 State Studies

The state studies listed below were selected based on their broad scope within the basin and is not presented as an exhaustive list. Throughout the course of this USACE feasibility study, both state and academia efforts have continued to investigate, evaluate, and improve flood risk within the Waccamaw River basin.

2.8.3.1 Horry County Multijurisdictional All-Hazards Mitigation Plan, October 2020.

This report was conducted by South Carolina Emergency Management and following the Hurricane Florence event in 2018. The report investigated primary sources of flooding within the Waccamaw River basin and identified and assessed possible mitigation strategies to prevent future flood damage. A quantitative hydrologic engineering model of the Waccamaw River basin was created for this effort by contractors of the State of South Carolina DNR and FEMA for portions of the Pee Dee River and Waccamaw River. Outcomes of this report were assessments of flooding sources, structural flood impact, and planning-level mitigation strategies for the Waccamaw River basin.

2.8.3.2 Horry County Resilience Plan 2022.

This report was conducted by Horry County. Horry County recognizes the need to understand the impacts of flooding and to put measures in place that can increase resilience to future flood events. The Horry County Flood Resilience Plan is a component of the County's Hazard Mitigation Plan and focuses on the development of flood mitigation strategies for the unincorporated areas of Horry County.

2.8.3.3 Conway Resiliency Effort

The City developed a Resiliency effort which included preservation and restoration of the community's essential basic structures and functions. The purpose of this resiliency document was to build on the resilience inventory, this element also included recommendations for future policies and projects to increase Conway's state of resilience. Flooding events in recent years, combined with the tremendous growth of the city, have put a strain on the City's essential services, infrastructure, and development. This document addressed the need for the City to identify challenges that occur as natural and man-made conditions change.

2.9 Existing Flood Risk

Horry County is situated in the northeastern corner of South Carolina, bordered by North Carolina to the north, the Atlantic Ocean to the east, and the Lumber and Little Pee Dee Rivers to the west. Horry County's extensive network of rivers, streams, and wetlands have been essential to residents for generations and sustained the rice, turpentine, and logging industries during the 18th and 19th centuries. Today, Horry County's access to the Atlantic Ocean, the Atlantic Intracoastal Waterway (AIWW) and other bodies of water in the region make it both a local and national tourist destination.

Horry County is the largest county by land area in South Carolina. It comprises 1,255 square miles of mostly flat topography, with elevations that range up to approximately 150 feet above sea level. Horry County is dominated by the Little Pee Dee and Waccamaw River watersheds. These watersheds, as well as many others in North and South Carolina, are part of the larger Yadkin/Pee Dee River Basin. Horry County is situated near the lowest point in this watershed before water exits the system through Winyah Bay. As this larger region grows and attracts new residents, increased tree cutting and clearing and the loss of natural permeable surface to development increase the footprint of the floodplain and reduce the storage capacity throughout the system.

Four target communities that were the focus of this study all have a significant number of buildings, transportation, and infrastructure assets that are highly vulnerable to flooding. Infrastructure vulnerability is often described as a combination of exposure and sensitivity. Assets in these communities are not only highly exposed to flooding, which means they are within a hazardous location (i.e., in a FEMA Special Flood Hazard Area or flooded in a past storm), but many are highly sensitive as well. Sensitivity is related to how an asset would fare if flooded and is a factor of the physical characteristics of the asset, such as elevation above the ground, age, construction, and condition. The following sections describe each of the target communities, including general characteristics, past storm impacts, and major infrastructure vulnerabilities.

The major water bodies that are in or run through Conway are the Waccamaw River, Crabtree Canal, Crabtree Swamp, Grier Swamp, Bear Swamp, Oakey Swamp, Altman Branch, and Kingston Lake. The Waccamaw River begins in NC at Lake Waccamaw, a freshwater lake within Carolina Bay. From this lake, the Waccamaw River winds 140 miles through Horry and Georgetown Counties, ending at the Winyah Bay estuary on the Atlantic coast. Kingston Lake and Crabtree Swamp are classified as streams in Horry County. Kingston Lake accepts drainage from many other bodies of water, including Crabtree Swamp. Crabtree Swamp was originally a low gradient coastal plain tributary to the Waccamaw River; the stream system was significantly modified by channelization projects in the 1960s and the 1980s.

The Horry County communities of Conway, Bucksport, Longs, Red Bluff, and Socastee were designated as the areas of focus for this study. Each of these communities has been continually impacted by riverine flooding and is representative of other areas in the County that also experience flooding from multiple waterways. Moreover, flooding in each community is uniquely impacted by the relationship of drainage basins, stream confluences, and topography in the area.

2.9.1 Conway, SC

The focus area of Conway, SC is in the middle of the watershed and is the most urbanized location. One of the oldest cities in South Carolina, Conway is a racially diverse coastal plain town just inland from the ocean. Part of the Myrtle Beach metropolitan area, Conway is prone to floods due to increasingly intense storms and hurricanes. The City of Conway is located within Horry County, a coastal plain county of almost 1300 square miles in the northeastern-most corner of the state of South Carolina. Water is a prominent natural feature throughout Horry County, and Conway is no exception. The community was founded on the banks of the Waccamaw River in 1732, and the 140-mile-long water body has been a powerful force in the life of Conway. The Waccamaw River is a blackwater sub-basin of the Pee Dee River, and the river's watershed provides drainage from communities in southeastern North Carolina through northeast South Carolina, ending at the Atlantic Ocean at Winyah Bay. In the past 10 years alone, river flooding due to hurricane or rainfall events (five of them major events) has resulted in millions of dollars of damages, FEMA buyouts, and a sense of urgency in the community to reduce further damages from future floods.

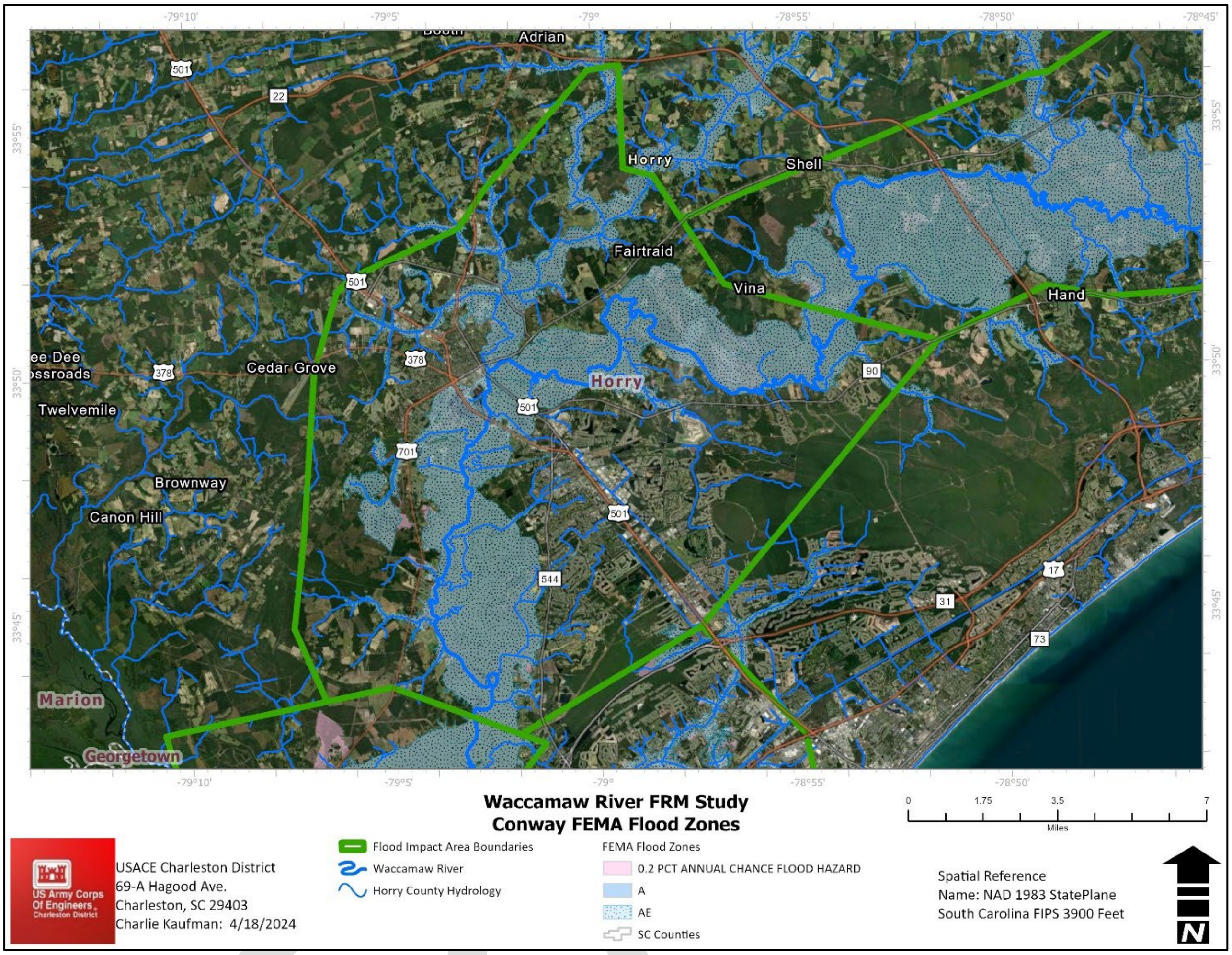


Figure 9. Conway, SC Focus Area within the Waccamaw River Watershed

For Conway – and nationally – the term floodplain has come to mean the land area that will be inundated by the overflow of water resulting from a 100-year flood – a flood which has a 1% chance of occurring any given year (SCDNR). Conway has non-tidal floodplains, or areas consisting of floodway and the floodway fringe along rivers and streams. Floodways carry the high velocity water, while the floodway fringe is subject to shallow flooding from the low velocity water. These areas are designated as AE or A1-30 zones on the Flood Insurance Rate Map (FIRM).

The City of Conway is in the Winyah Bay watershed, and more specifically, in the Waccamaw River Sub-basin. The Winyah Bay watershed covers most of northeastern South Carolina and extends into North Carolina. What one does in one area can affect people throughout the whole watershed.

Figure 10. Flooding from Hurricane Florence, 2018 in Conway, SC (Horry County, 2021)

Besides an increase in flood events, the city and county have both experienced overwhelming growth over the last two decades. According to the Horry County Flood Resilience Master Plan (2021), the county population has swelled by almost 25% since 2010, to 351,029 residents; Conway has doubled its population since 2000. With a temperate climate, a relatively inexpensive cost of living, and Myrtle Beach as a regional destination, Horry County is projected to double in size by 2040.

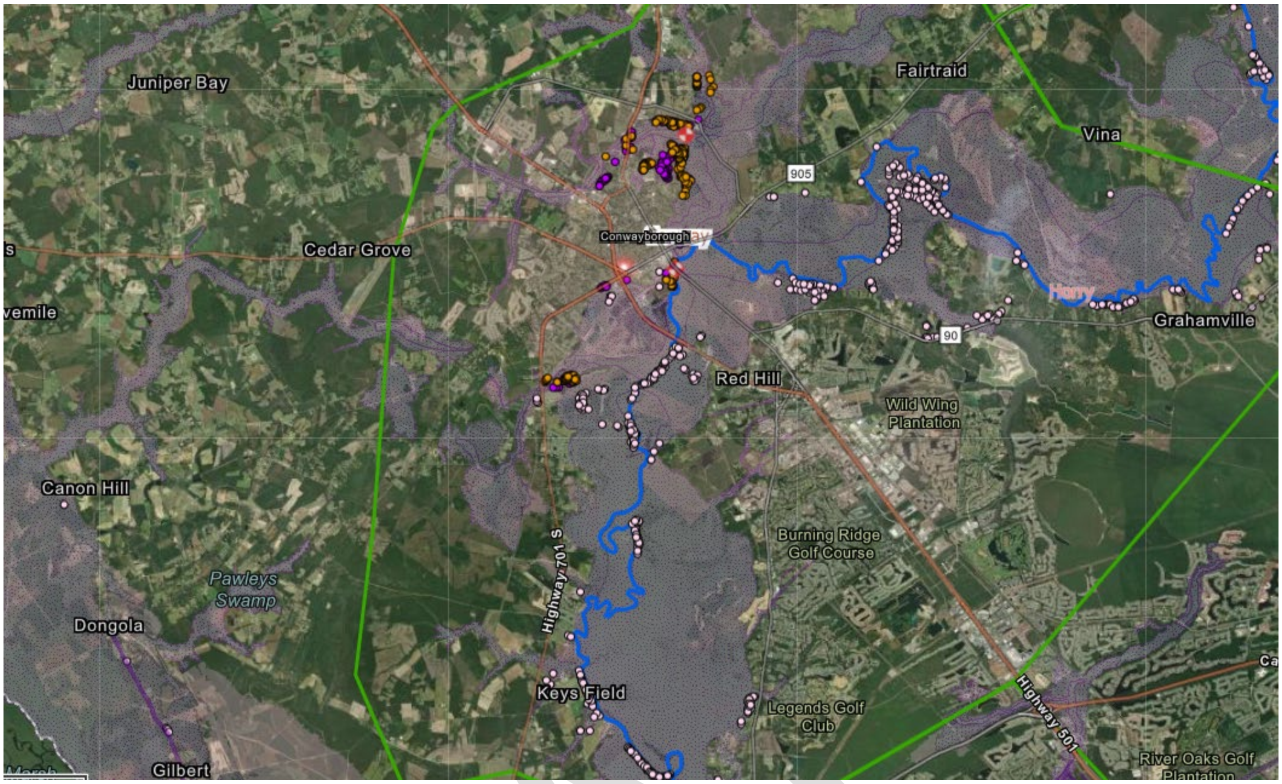


Figure 11. Locations of repeated flooded properties from Hurricanes and major flooding events.

NOAA's National Weather Service defines flooding as an overflowing of water onto land that is normally dry. In the City of Conway, flooding occurs most often during and after rainstorms and hurricanes. Factors contributing to nuisance flooding include the city's location in the South Carolina Coastal Plain, 14 miles west of the Atlantic Ocean; being developed on the western banks of the Waccamaw River; and with relatively low elevations in relation to sea level. Flooding is the most frequent and costly natural hazard in the United States (EPA).

The types of flooding that Conway generally experiences because of named storms or rain events are riverine and flash flooding. Riverine flooding is characterized by widespread rainfall across a river basin resulting in stormwater that accumulates in volume as it moves downstream (Horry County Flood Resiliency Plan). Flash flooding occurs when rainfall amounts exceed what can be absorbed or retained onsite, causing runoff that affects adjoining properties and streets. Flash flooding is felt immediately during and after a storm; however, it is seldom a multi-day event. In addition to riverine and flash flooding, compound flooding – when combined with riverine and flash flooding, increases the water table and the extent of flooding beyond what is expected from a single type of flooding. Conway is also considered to be within the coastal zone, and riverine flooding is exacerbated by tidal backwater flooding (South Atlantic Coastal Study (SACS)). Conway GIS estimates that a total of 5,460 acres (divided by 16,437 acres – total acreage of city), or 33.21% of all properties in the city limits, are within a flood zone, per the 2019 Revised Flood Maps.

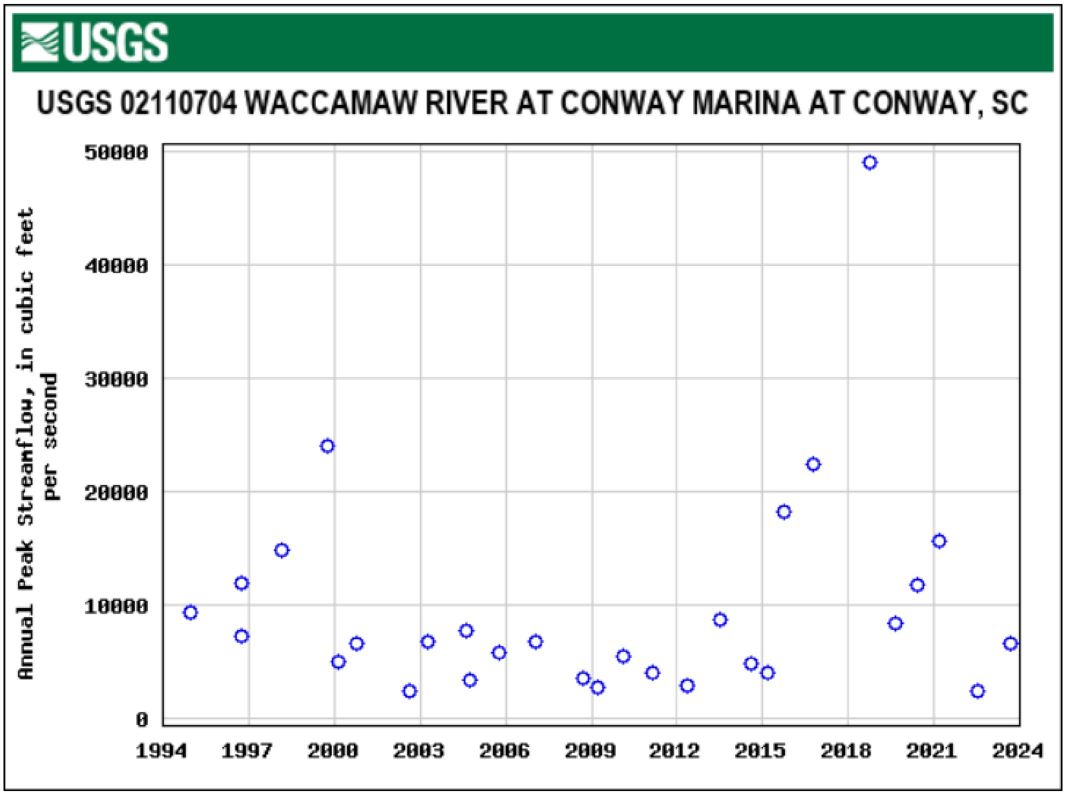


Figure 12. USGS Gage 02110704 Waccamaw River at Conway Marina Annual Peak Streamflow since 1994

A historic rain event can be described as a severe rain occurrence, whether associated with tropical storms, hurricanes or not, that results in major flooding in areas that may not have had flooding in prior years. Rain events, in combination with other factors, result in widespread flooding, drainage issues, and storm surges. The City of Conway frequently experiences flooding from rainstorms not associated with tropical storms or hurricanes. These storms occasionally result in structural damage; more often, road and park closures.

A list of historic crests for USGS gage 02110802 Waccamaw River near Bucksport can be viewed in Table 3. As seen in the table, six of the top ten highest peaks occurred within the past ten years, indicated by an asterisk (*). On February 27, 2021, the Waccamaw River peaked at 23.13' – the highest crest ever occurring in a non-hurricane rain event. Due to the City's resiliency efforts, minimal damage was experienced.

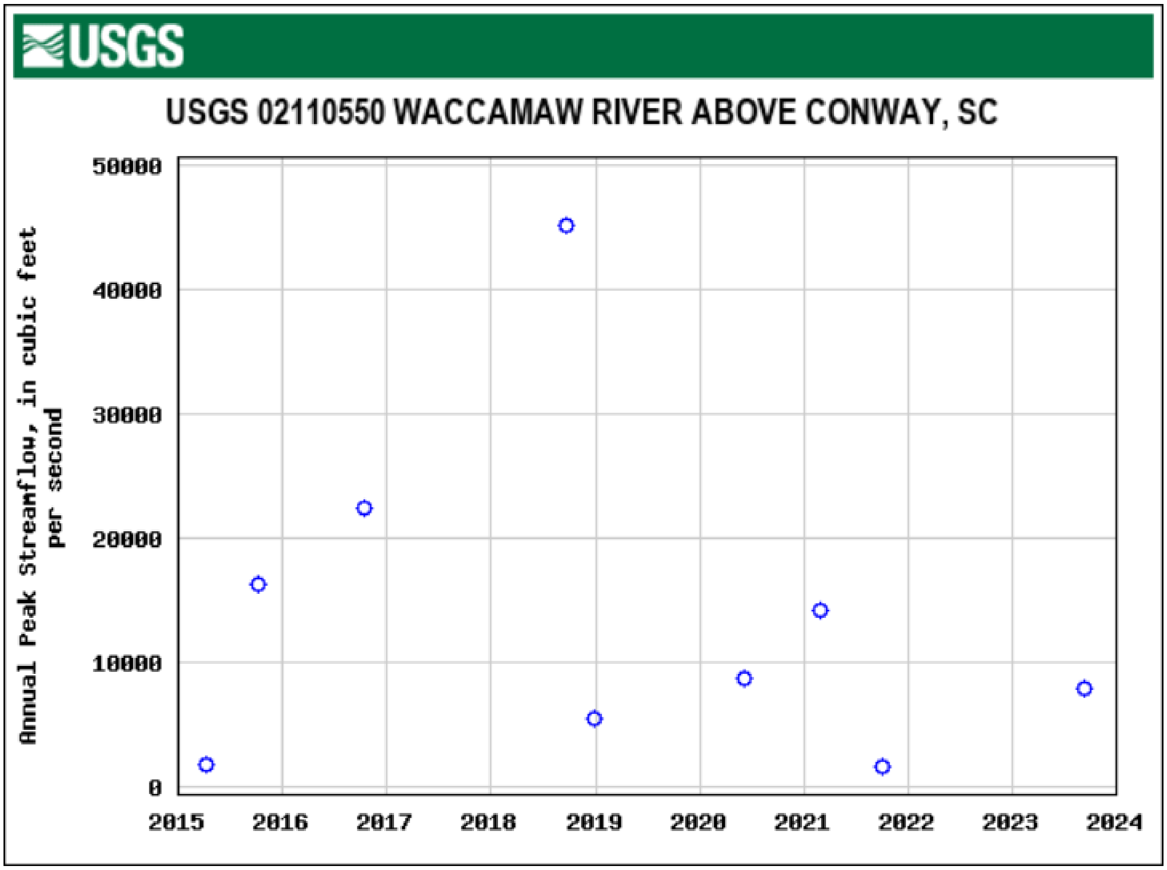


Figure 13. USGS Gage 02110550 Waccamaw River Above Conway, Annual Peak Streamflow.

Table 3. Select Floods of Record of Conway, SC near Conway Marina (02110704).

Date	Gage Height (ft)
09/26/2018*	21.16
10/18/2016*	17.89
09/30/1928	17.8
09/27/1999	17.6
10/10/2015*	16.23
10/08/2015*	16.1
02/27/2021*	15.6
09/29/1945	15.6
09/18/2018*	15.57
10/09/1924	15.5
10/02/1924	15.4
09/20/1928	15.3
10/10/2016	15.11
02/19/1998	14.8
02/12/1998	14.7
09/07/1908	14.6
03/30/1983	14.5
09/19/1996	14.4
08/10/1908	14.1
06/07/2020	14.08
10/28/1999	13.9
03/19/1983	13.8
08/27/1981	13.8
04/18/1936	13.7
01/31/1925	13.7
05/01/1918	13.7
10/20/1894	13.6
01/20/1993	13.6
07/29/1916	13.6
02/17/2016	13.56

2.9.2 Bucksport

Bucksport is the most downstream focus area community, located in southwestern Horry County and nestled between the Great Pee Dee and Waccamaw Rivers, just to the north and east of their confluence. To the west of Bucksport, these two major rivers are connected by Bull Creek, a former channel of the Great Pee Dee. This community is bordered on three sides by the expansive3 floodplain and wetlands of the Waccamaw National Wildlife Refuge. Overall, Bucksport is low-lying, particularly in developed areas where elevations rarely exceed 17 feet above sea-level.

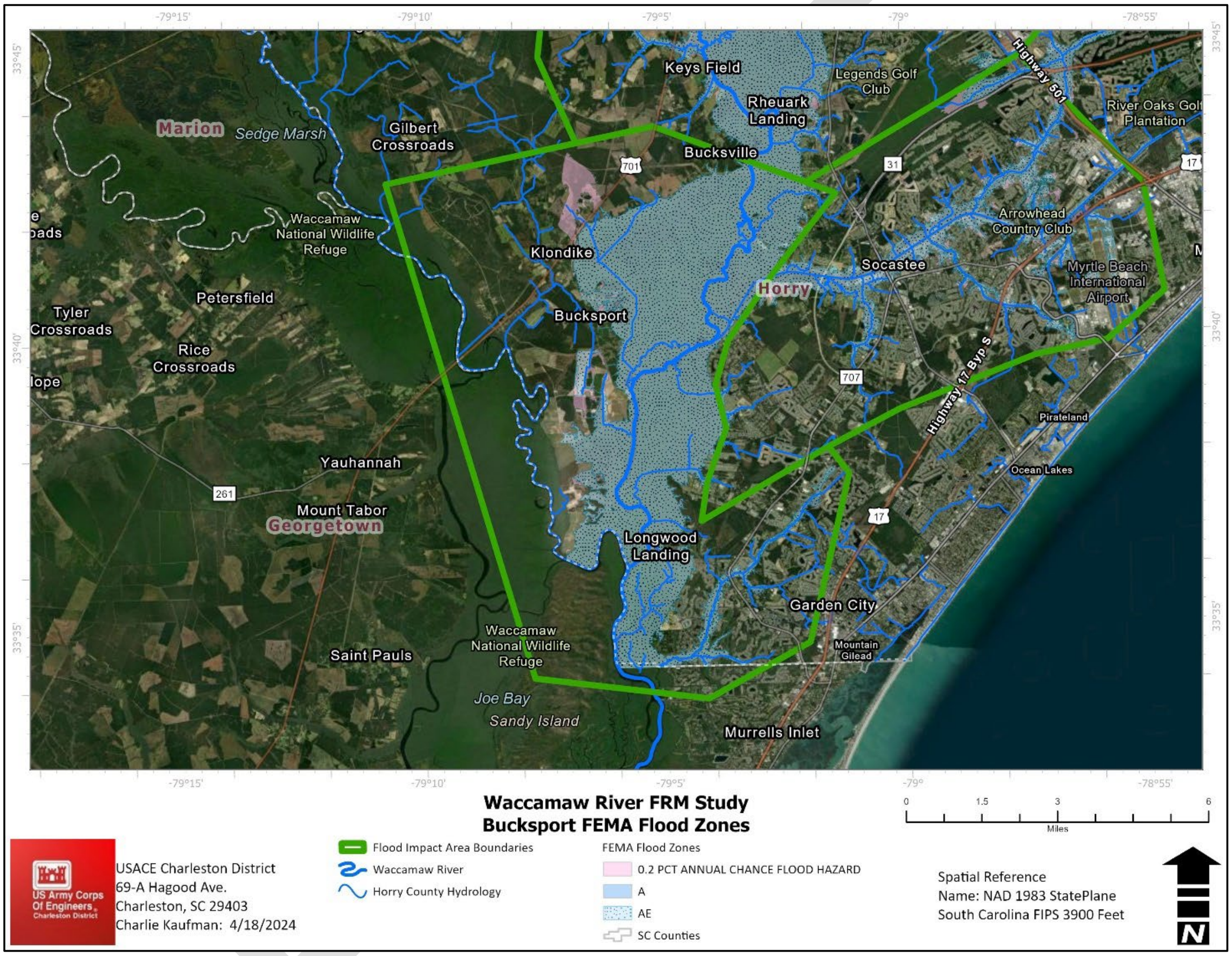


Figure 14. Focus Area of Bucksport, SC within the Waccamaw River Watershed

Like Socastee, Bucksport is also a location of repeated flooding. It is a socially close-knit community with a general reluctance on the part of residents to move to other areas of Horry County. This is both a strength and a vulnerability, as the community will be united by projects which keep their neighborhoods intact, but reluctant to accept buyouts of repetitive loss properties. Common themes that were mentioned by Bucksport residents during the public engagement meetings included elevating highways, such as SC HWY 22, SC HWY 501, Port Harrelson Road, and SC HWY 701. The residents also frequently talked about the inability to travel on the roadways, to check on their homes, orto work.

Bucksport is located on a peninsula between the Little Pee Dee and Waccamaw rivers. This area experienced flooding as water breeched Big Bull Landing Road and crept in the community through the drainage system. The community also experienced flooding from the opposite side from the Waccamaw River. The flooding from these two rivers converged and washed over Bucksport Road, the main point of access in the community. Many homes were damaged during Hurricane Florence in 2018; however, after the waters subsided, the availability of public recovery assistance was limited.

Figure 15. Flooded Road during Hurricane Florence, in Bucksport, SC (Horry County, 2021)

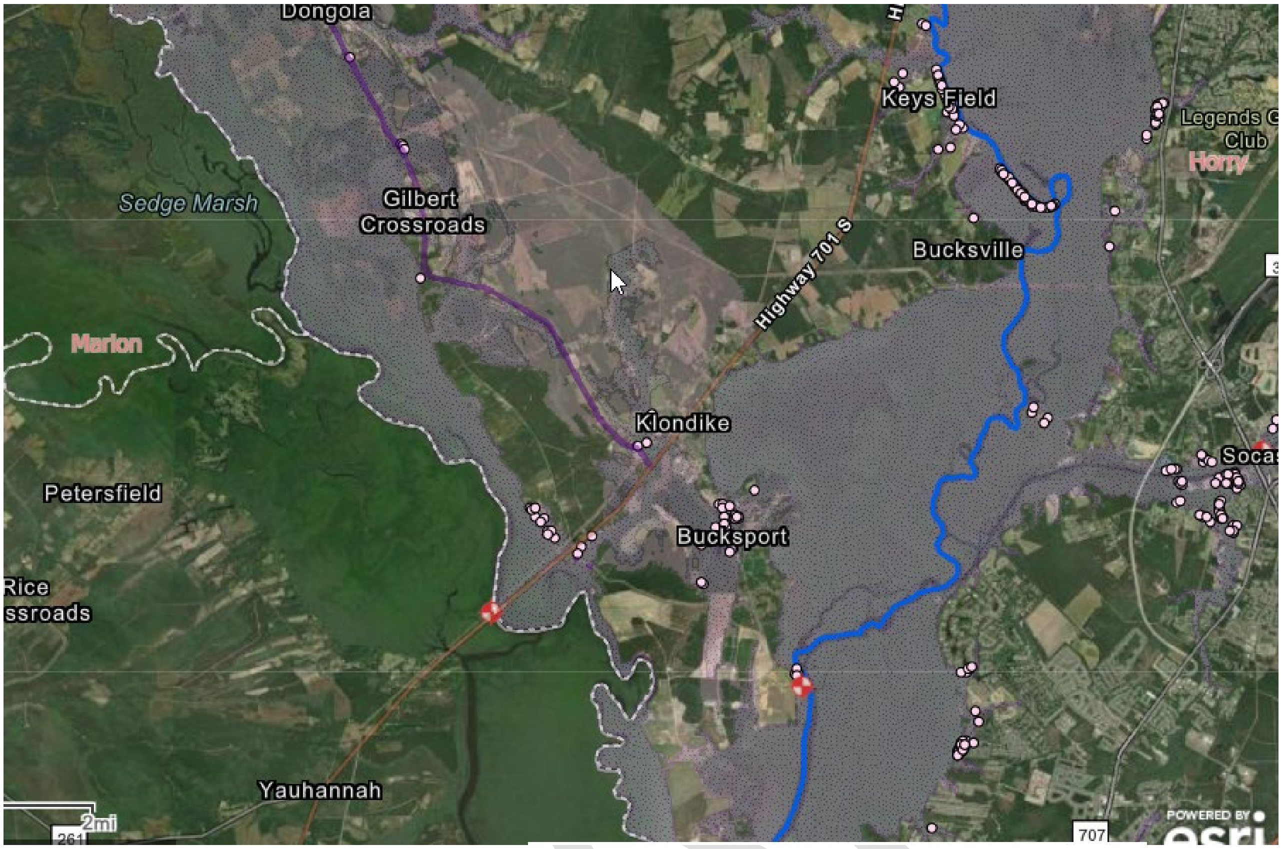


Figure 16. Properties repeatedly flooded by major Hurricanes and flooding events

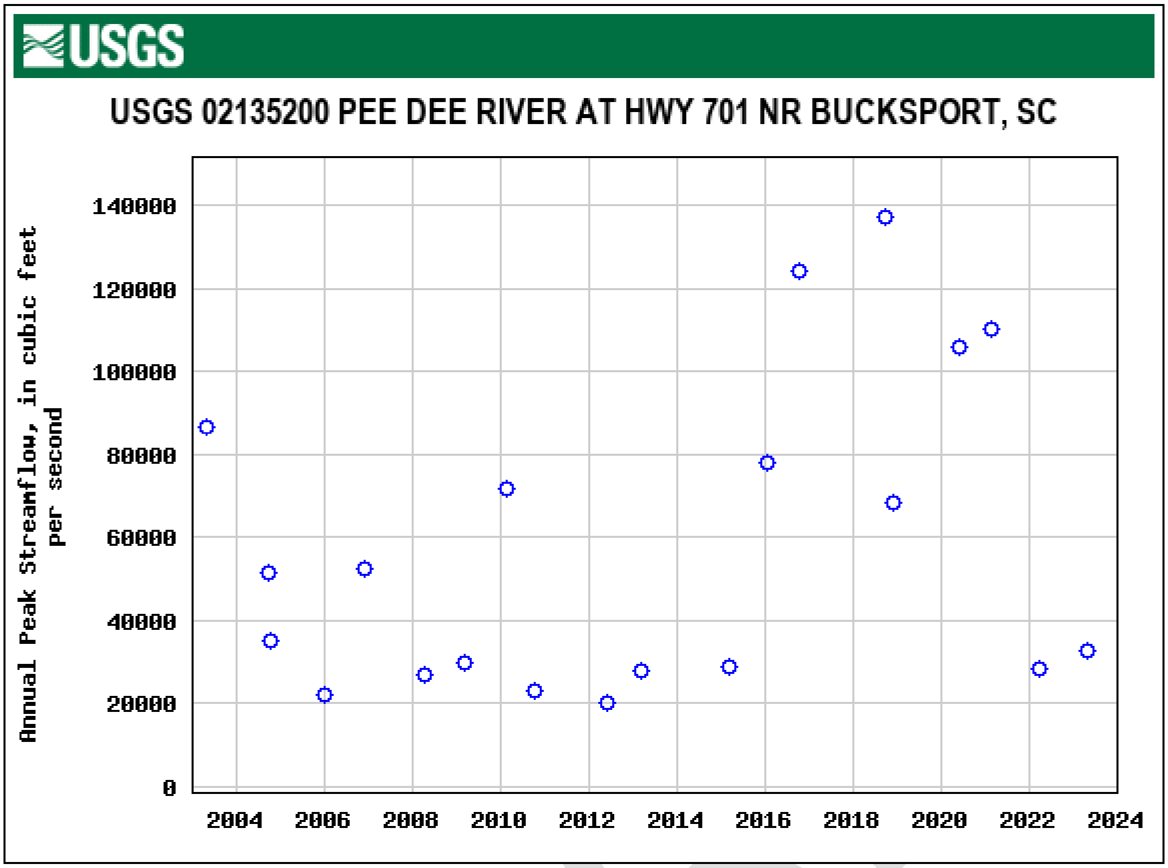


Figure 17. Floods of Record of the Pee Dee River near Bucksport, SC

Table 4. Select Floods of Record of Waccamaw River near Bucksport, USGS GAGE 02110802

Date	Gage Height (ft)
09-27-2018	26.67
02-27-2021	23.13
10-21-2016	22.74
06-07-2020	22.31
02-10-1998	21.76
10-12-2015	21.47
02-21-2020	21.24
01-10-2016	21.19
11-24-2018	20.72
11-25-2020	20.65
12-29-2018	20.31
12-31-1994	20.24
12-21-2018	20.07
09-29-1999	19.98
01-13-2021	19.79
02-16-2016	19.56
11-22-2015	19.38
03-07-2019	19.32
02-15-2010	19.29
07-19-2013	19.28
10-08-1996	19.17
09-12-2017	19.15
09-05-2019	19.14
02-06-2010	19.12
02-16-2021	19.11
10-26-2018	19.07
11-05-2020	19.05
12-19-2009	19.04
12-06-2006	19.03
09-19-1996	19.03

2.9.3 Socastee, SC

The target community of Socastee is adjacent to the Intracoastal Waterway, approximately four miles east of the confluence with the Waccamaw River (Figure 16). Socastee is an established community that consists of a mixture of older subdivisions from the twentieth century as well as new construction. Socastee is more developed than the other target communities (in the 90th percentile of population density compared to other South Carolina areas) and consists of a mixture of residential neighborhoods and subdivisions, commercial businesses, and public infrastructure, such as schools and churches. The average age of residents in the Socastee community is 38 years old, and approximately 67 percent of the homes are owner-occupied.

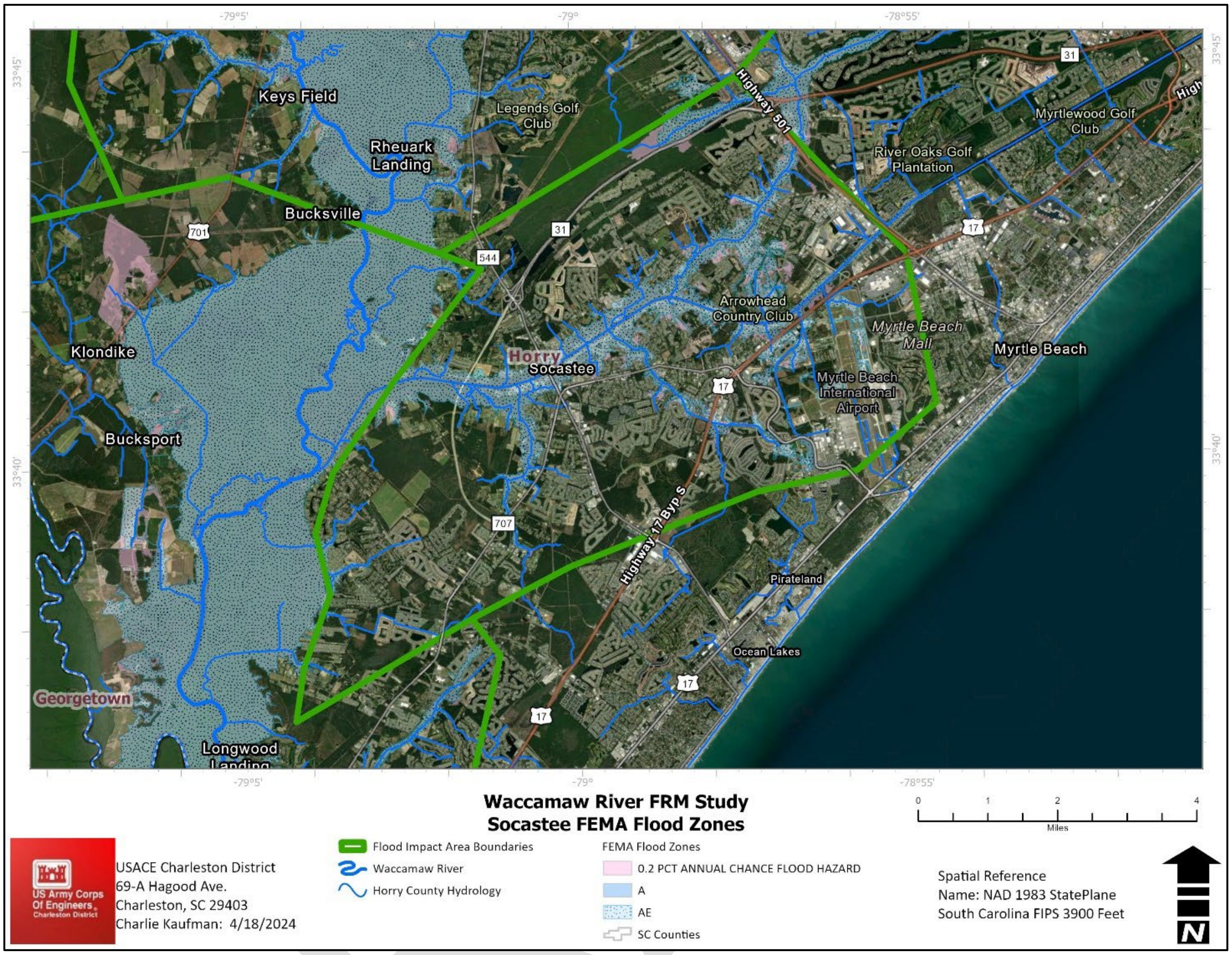


Figure 18. Socastee Focus Area within the Waccamaw River Watershed

During Florence, the river gauge along the Intracoastal Waterway in Socastee recorded a peak stage on September 27, 2018, of approximately nine feet above normal. Much of this community was built on the low-lying geomorphic floodplain of Socastee Swamp (now bisected by the AIWW), which was flooded as water backed up at the confluence with the Waccamaw River. Many buildings in the Socastee community were damaged by flooding during Hurricane Florence. Some of the worst flooding was concentrated in the Rosewood, Bridge Creek, Lawson's Landing, and Watson's Riverside neighborhoods, with water levels up to six feet in some homes. Post-storm assessments in the Socastee vicinity showed almost 565 buildings were damaged by flooding (Figure 19).

Figure 19. Flooded Homes and properties in Socastee during Hurricane Florence (Horry County (2021)

Overall, Socastee is low in elevation, with a large portion of the area less than ten feet above sea-level. When low lying land is poorly drained, it retains water for longer periods, and the ability for water to infiltrate is restricted due to the decreased void space, thus increasing the amount of surface runoff. As a result, this community had particularly high-water levels during Matthew and Florence (Table 5).

Table 5. Select Floods of Record at Socastee Creek

Date	Gage Height (ft)
9/28/2018	21.83
10/18/2016	19.23
2/27/2021	18.24
6/7/2020	17.35
10/12/2015	16.8
4/23/2003	16.03
9/30/1999	15.76
11/24/2020	15.75
12/17/2023	15.34
9/13/2017	14.59
9/22/2004	14.58
7/19/2013	14.5
2/7/2010	14.44
12/19/2009	14.43
1/14/2019	14.38
12/7/2006	14.22
1/9/2024	14.17

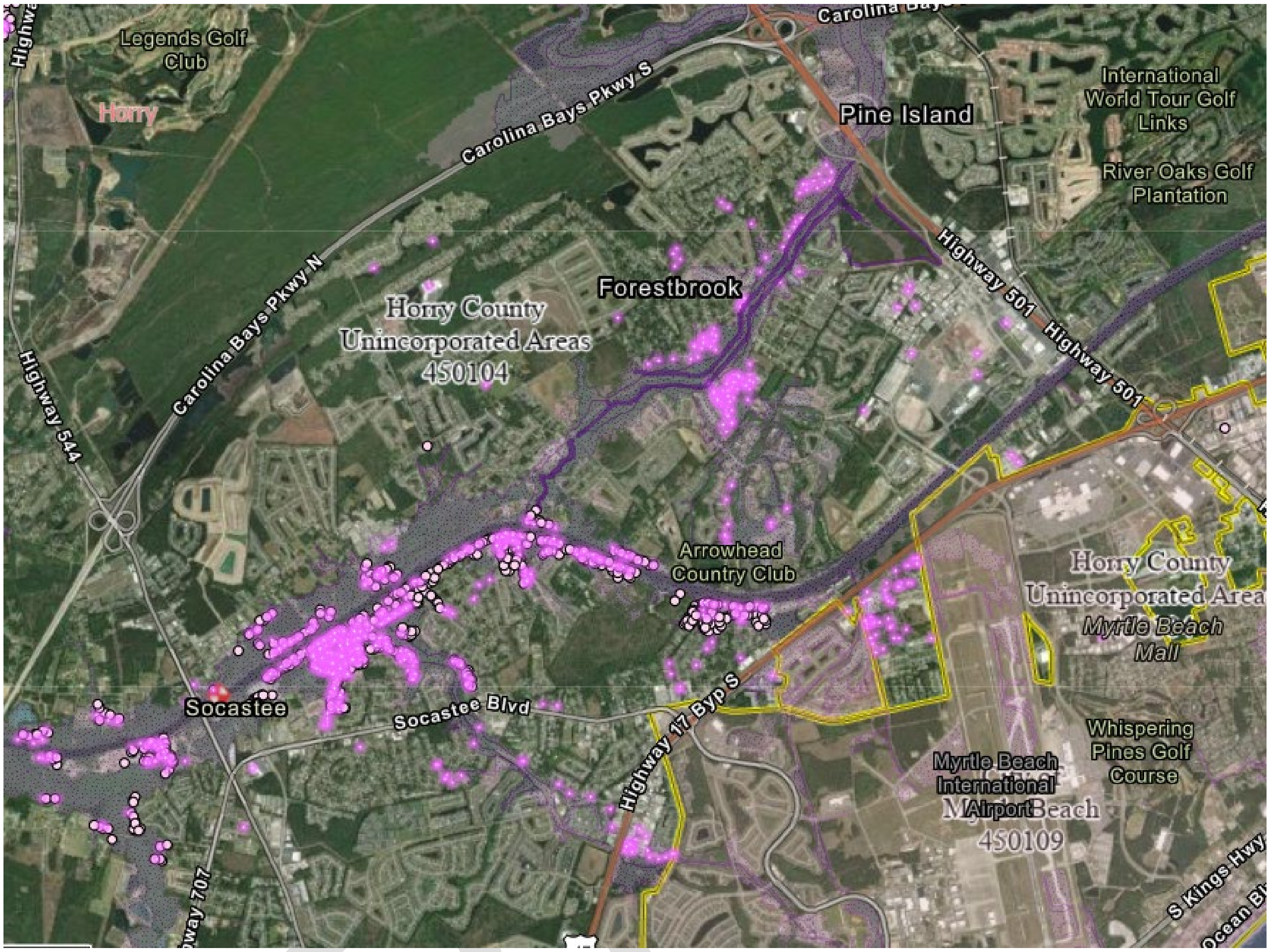


Figure 20. Properties in Socastee repeatedly flooded during major events

2.9.4 Longs/Red Bluff, SC

Longs is the northernmost impacted community targeted in this study, located north of the confluence of the Waccamaw River and Buck Creek (Longs lies a few miles southwest of the North Carolina border, near the intersection of SC HWY 9 and SC HWY 905). This small unincorporated community consists primarily of residential neighborhoods, subdivisions, small commercial businesses, and golf courses.

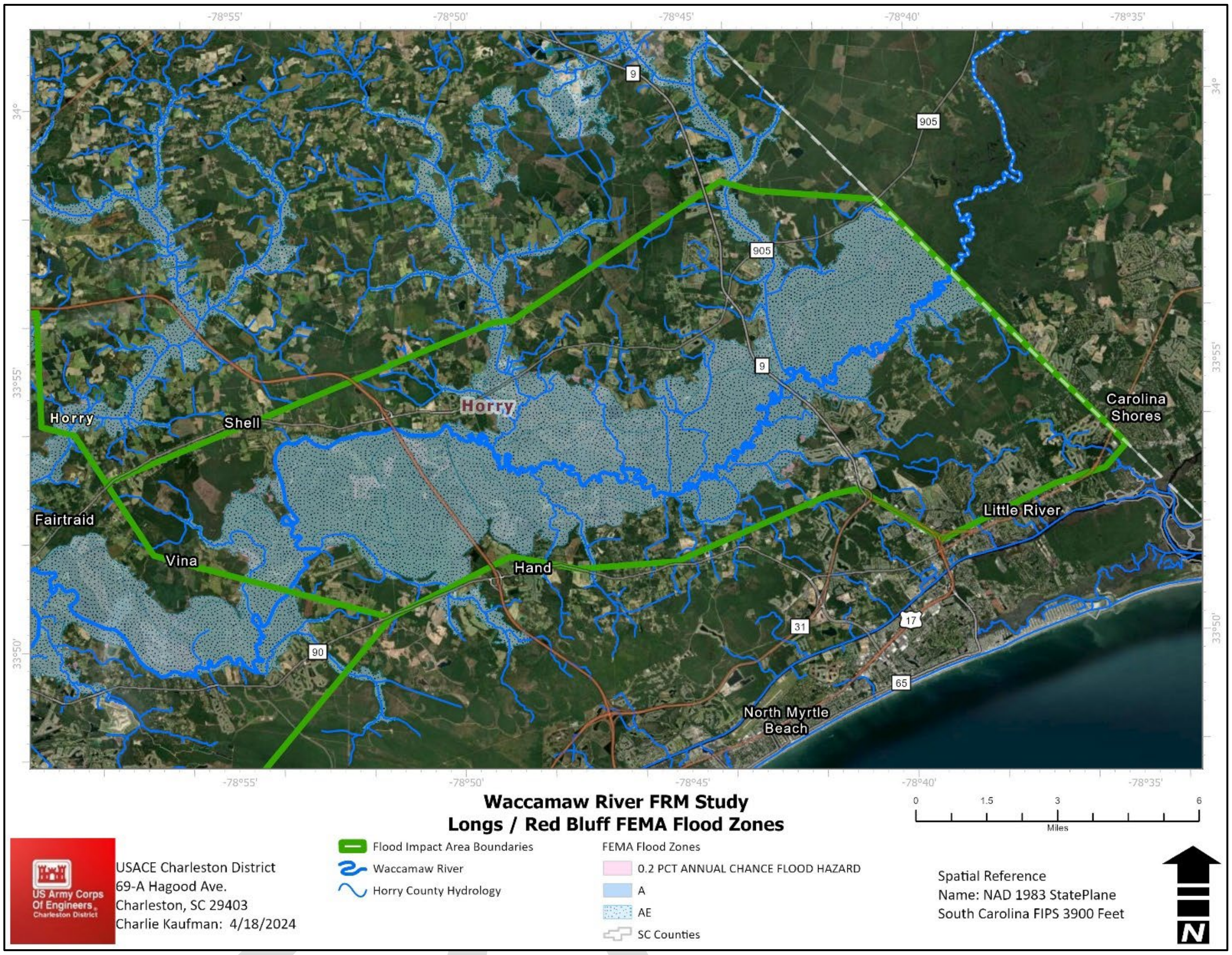


Figure 21. Focus Area of Longs/ Red Bluff

According to Horry County (2021), the average age of residents in the Longs community is 47 years old, and approximately 61 percent of the homes are owner-occupied (2010 census block data). Many homes in the area experience repeated flooding during major events (Figure 20). During Florence, the Waccamaw River gage near Longs recorded a peak stage on September 21, 2018, of approximately 18 feet above normal (Figure 23). A second stream gauge along Buck Creek also recorded a peak stage of almost 15 feet above normal. Buck Creek is a tributary of the Waccamaw River, and properties and infrastructure near the confluence of these two water bodies experienced widespread flooding. A list of floods of record at the Longs Waccamaw River USGS gage can be found in Table 6.

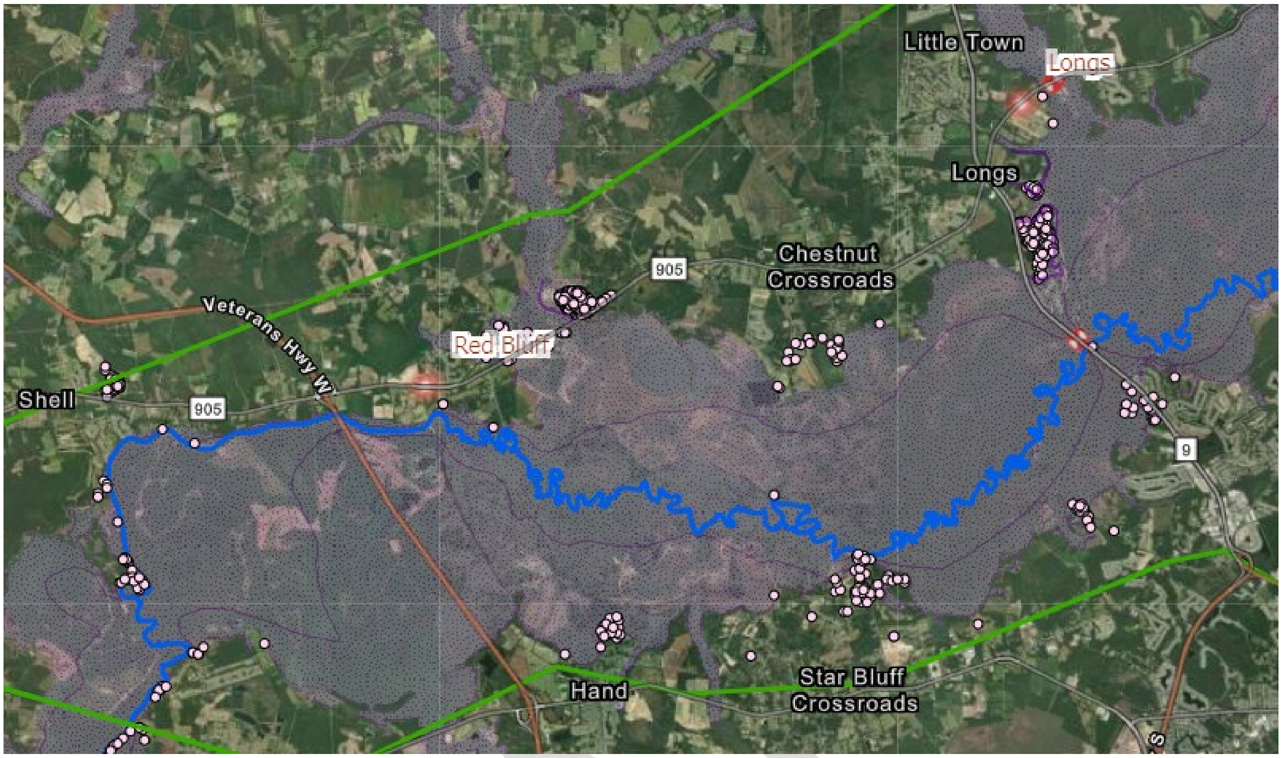


Figure 22. Repeatedly Flooded Properties in Longs/Red Bluff

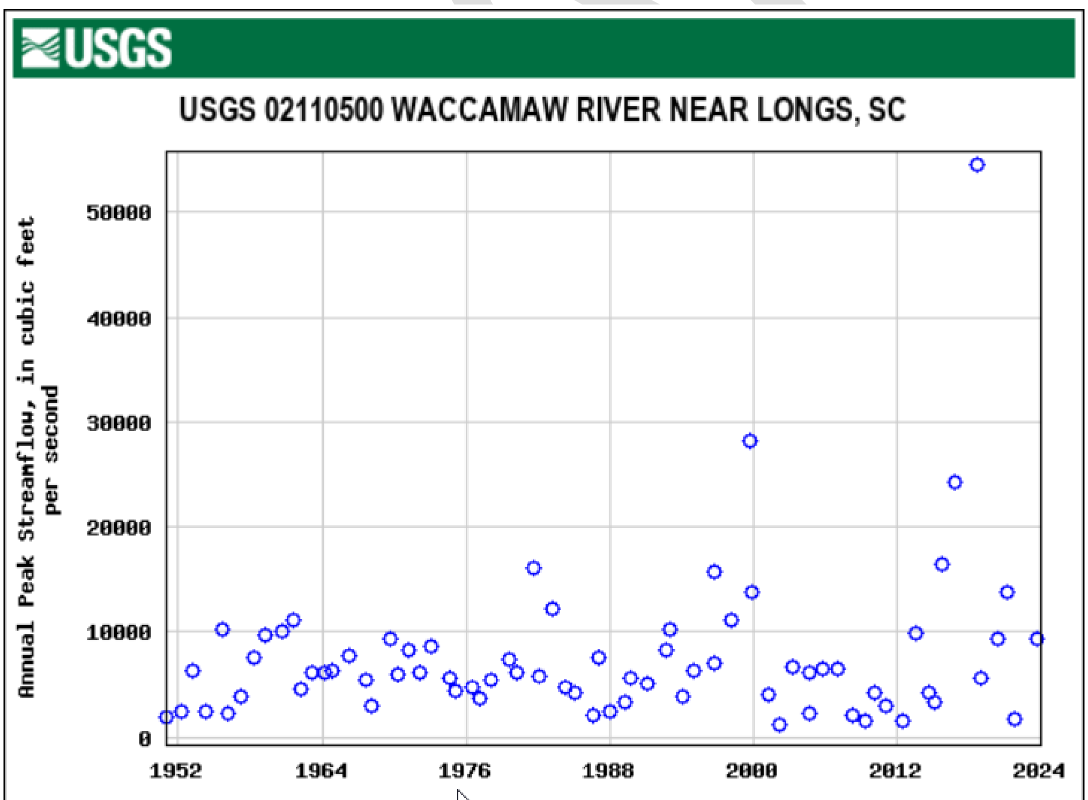


Figure 23. Annual Peak Streamflow for Waccamaw River Near Longs, SC USGS Gage 02110500

Table 6. Select Floods of Record at Longs USGS Gage 02110500

Date	Gage Height (ft)
09/22/2018	20.19
09/23/1999	17.94
10/14/2016	16.95
10/06/2015	15.17
09/15/1996	14.95
08/23/1981	14.87
10/25/1999	14.49
03/27/1983	14.4
02/23/2021	14.34
07/06/1961	13.94
02/09/1998	13.82
09/29/1955	13.82
02/19/1998	13.68
01/14/1993	13.63
08/04/1960	13.52
05/08/1999	13.49
07/09/2013	13.43
02/13/2016	13.42
03/13/1959	13.4
08/13/1969	13.26
06/02/2020	13.24
02/20/1973	13.1
08/23/1992	13.03
04/18/1961	12.95
03/12/1971	12.85
04/12/1973	12.8
03/09/1987	12.75
09/16/1979	12.72
02/24/1983	12.7
09/26/2000	12.65

2.9.5 Inundated Roads

There are numerous major transportation routes that are vulnerable to significant flooding impacts throughout the basin, especially for communities in the Coastal Plain region. Emergency management and service efforts at the Federal, State, and Local levels are among the most challenged during and following significant basin-wide flood events.

Transportation corridors in the Socastee community are also highly vulnerable to flooding. Numerous residential streets in Socastee were closed during Florence, particularly in the subdivisions with a high number of damaged homes. While flood waters do not always cause extensive physical damage to roads, the extended closures severely restrict travel, hindering residents trying to return home.

Longs and Red Bluff have several transportation corridors that are highly vulnerable to flooding. They include large portions of four major highways, including SC HWY 22, SC HWY 554, SC HWY 31, and SC HWY 905. During Florence, significant portions of these highways were closed for extended periods of time due to flooding. A portion of SC HWY 905 stretching across most of the Red Bluff and Chestnut Crossroads community was closed for nearly two weeks after Florence, and SC HWY 31 was closed across the Waccamaw River floodplain for nearly a month. Secondary roads near the damaged homes in this area (such as in Polo Farms) were also flooded for several weeks. There are also numerous bridges in this community that were closed along with the roads during Florence.

Many roads in the Bucksport community are also highly vulnerable to flooding. Bucksport Road, the primary road in the community, was flooded for almost two weeks during Florence. Almost all other residential roads in the area were also flooded to some extent. These extended road closures limited the ability of residents to return home after the storm, check on flooded homes, and even travel to work.

According to the Horry County Flood Resiliency plan (2021), in addition to building damage, over 460 road closures were attributed to Florence across Horry County, and more than 250 of these roads were either washed out or damaged by flooding (Figure 22). Some portions of primary routes were closed for up to two weeks. Routes have been designated by the magnitude of inundation, up to a scenario of >5-ft of floodwaters. Return frequency inundation scenarios were based on FEMA-related hydraulic modeling. In the weeks directly following Florence, major travel routes including SC HWY 9 and SC HWY 22 were closed due to flooding. SC HWY 501 was the only access road between land to the west of the Waccamaw River and the beach, and one lane on each side of the highway were closed to be secured with sandbags, causing commute times to be greatly increased.

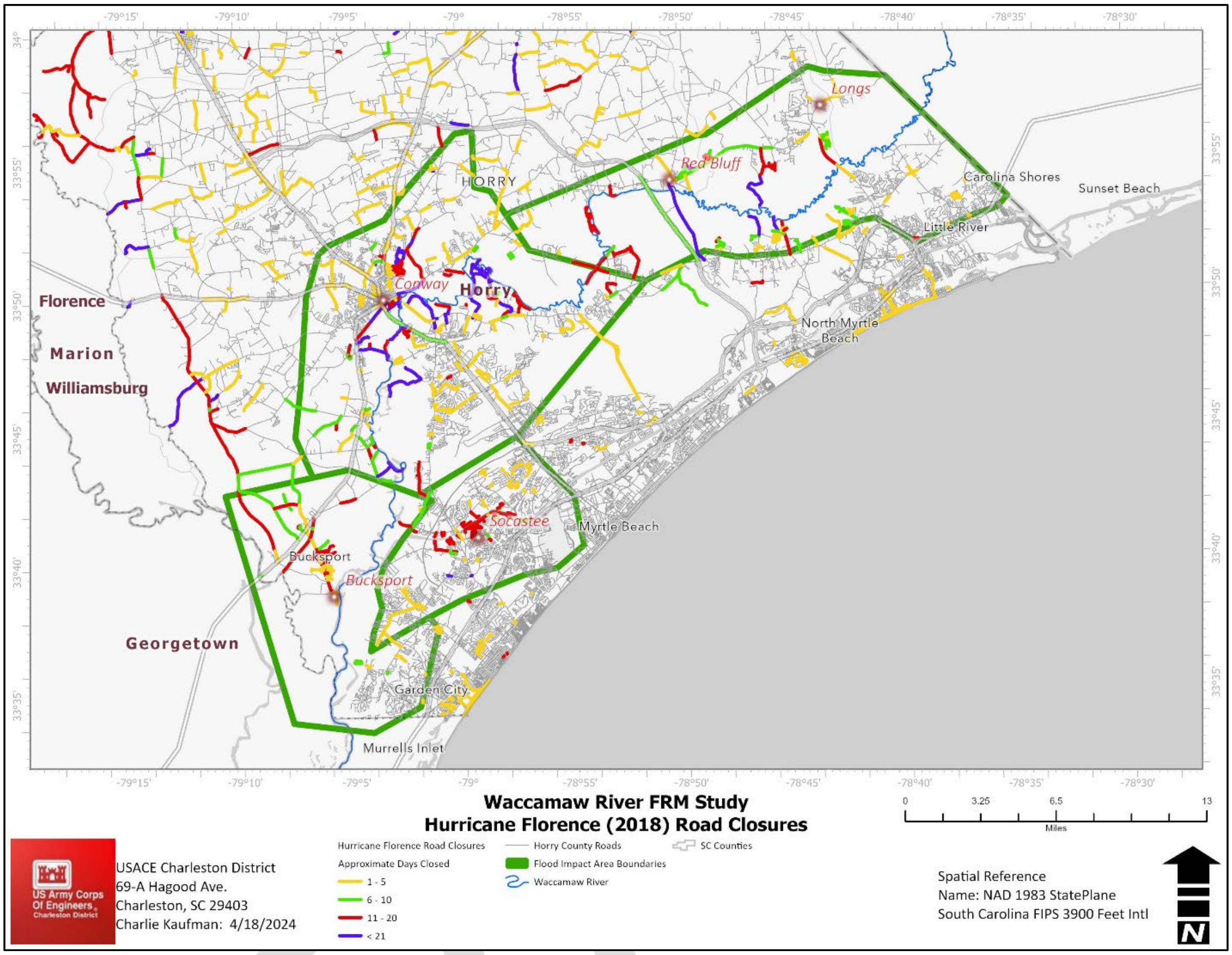


Figure 24. Hurricane Florence Road Closures

The partial closure of SC HWY 501 proved especially problematic as the highway was already a roadway with one of the highest volumes in the County, serving over 40,000 vehicles on an average day. SC HWY9 reopened October 1, 2018, although westbound lanes were still flooded, traffic was diverted in the eastbound lanes. These closures severely restricted travel in the region, limiting the ability of evacuees to return home and the trucking of supplies. A large group of residents were forced to stay in hotels for long periods of time and were unable to commute to work, compounding financial difficulties.

3.0 Data Collection

3.1 Hydrologic Data

3.1.1 Streamflow and Stage Data

The United States Geological Survey (USGS) provides extensive coverage of streamflow and stage records throughout the study area. Table 7 provides a summary of available data for select USGS sites that were utilized for the purposes of this study.

Table 7. Select USGS streamflow sites pertinent to the Waccamaw River basin study

Site ID	Description	Drainage Area (sq mi)	Period of Record (CY)	Datum (ft, NAVD88)
2109500	Waccamaw River at Freeland, NC	680	1985-2024*	14.46
2110400	Buck Creek near Longs, SC	46.9	2005-2024	5.3
2110500	Waccamaw River Near Longs, SC	1100	2007-2020	4.22
2110525	Waccamaw River at SC-22 Below Longs, SC	1230	2018-2024*	-8.76
2110550	Waccamaw River Above Conway, SC	1250	1982-2024*	0
2110760	AIW at Myrtlewood Golf Course at Myrtle Beach, SC	98.9	1996-2024*	12.07
2110725	AIW At Highway 544 at Socastee, SC	771	1999-2024	-10.88
2110704	Waccamaw River at Conway Marina	1440	1994-2024*	-6.14
2110701	Crabtree Swamp at Conway, SC	18.9	2000-2024	-9.33
2110802	Waccamaw River at Bucksport, SC	1580	2005-2024	-15.56
21108125	Waccamaw River Near Pawleys Island, SC	1620	2001-2024	-4.5
2110815	Waccamaw River NR Hagley Land, NR Pawleys, SC	1640	1989-2024*	-15.68
2135200	Pee Dee River at Highway 701 NR Bucksport, SC	14100	2001-2024*	-8.85

3.1.2 Land Use

All but one site has a gage height and peak flow period record extending through calendar year 2024. The gage sites that have both gage height and peak streamflow are indicated with the asterisk (*). Due to the consistent use of the NAVD88 vertical datum by USGS at these sites, conversion from older datums isn't a concern for integration with other modern hydrologic and hydraulic data.

Rainfall losses were computed using the Natural Resource Conservation Service (NRCS) curve number method. The curve numbers were generated using the National Land Cover Database's (NLCD) 2019 Land Cover raster and the October 2021 Soil Survey Geographic Database (SSURGO), from which the hydrologic soil group (HSG) was obtained. An abstraction ratio of 0.2 and a minimum infiltration rate of 0.001 inches/hour were used to determine rainfall losses. Table 8 provides the curve numbers for each land cover and soil type combination.

Table 8. NLCD 2019 Land Cover with Corresponding Curve Number and SSURGO data

NLCD Land Cover Description	NLCD Value	Percent Impervious	CN by SSURGO HSG						
			A	A-D	B	B-D	C	C-D	D
Open Water	11	100	100	100	100	100	100	100	100
Developed, Open Space	21	5	46	82	65	82	77	82	82
Developed, Low Intensity	22	20	61	87	75	87	83	87	87
Developed, Medium Intensity	23	50	77	92	85	92	90	92	92
Developed, High Intensity	24	80	89	95	92	95	94	95	95
Barren Land Rock-Sand-Clay	31	0	77	94	86	94	91	94	94
Deciduous Forest	41	0	36	79	60	79	73	79	79
Evergreen Forest	42	0	36	79	60	79	73	79	79
Mixed Forest	43	0	36	79	60	79	73	79	79
Shrub-Scrub	52	0	35	77	56	77	70	77	77
Grassland-Herbaceous	71	0	58	89	71	89	81	89	89
Pasture-Hay	81	0	49	84	69	84	79	84	84
Cultivated Crops	82	0	67	89	78	89	85	89	89
Woody Wetlands	90	0	45	83	65	83	73	83	82
Emergent Herbaceous Wetlands	95	0	57	87	70	87	80	87	87

3.1.3 Rainfall Data

Historical and current rainfall data was obtained and evaluated from four gages near and around the Waccamaw River. Historical data was obtained from the National Weather service gages; 0211040, 02110550, 335446079024200, and 02110701. Rainfall data gages are shown in Table 9.

Table 9. Available rainfall data at precipitation gages in the Waccamaw River watershed

Station Name	Station Number	Precip (in). POR
Buck Creek	2110400	2010-2024
Waccamaw River Above Conway	2110550	2013-2024
Meteorological Station at Conway, SC	335446079024200	2022-2024
Crabtree Swamp at Conway	2110701	2007-2024

Figure 25: Precipitation Gage Station locations

3.2 Topographic Data

Through the collaboration of various State and Federal agencies, including FEMA, SCDOT and USDA, a basin-wide Light Detection and Ranging (LiDAR) topographic dataset was available for this study. It was comprised of a multi-phased collection effort between 2014 and 2016 and is classified as Quality Level 2 (QL2). This allowed for a 30-meter post spacing collection with 8 points per meter precision.

Channel surveys from multiple sources were used to enhance study area Digital Elevation Models (DEMs). Cross sectional geometry within stream banks were obtained from FEMA hydraulic modeling and were merged with LiDAR-derived overbank floodplain. Figure 25 shows the stream, rivers and major waterbodies within the study area. Also, the Waccamaw River Bathymetry was measured by Coastal Carolina University using a 50 cm raster cell resolution. According to County Flood Insurance Studies in the study area, natural floodplain cross sections were surveyed approximately every 4,000 feet along detail study reaches to obtain geometry between bridges and culverts (FEMA, 2019). Efforts were made to georeferenced older FEMA hydraulic models, with emphasis placed on assuring accuracy at structural stream crossings. In the lower reaches of the Waccamaw River and within the AIWW, bathymetry was supplemented with Coastal Carolina University Bathymetric Measurements. Additional bathymetric measurements obtained for the upper-most part of the study were not obtained in time to include in the FWOP modeling. However, preliminary review of the data support observations of ongoing channel meander migration, which validate the assumptions of not pursuing the structural measures of the wild and scenic river. Restricting the natural channel meander, would inhibit natural stream flow.

Figure 26. Streams, Rivers and major waterbodies in Horry County

The layers necessary to develop the HEC-RAS 2D model include terrain, land cover, soil, and rainfall. Table 10 lists the data provided by USACE SAC and data gathered from outside sources used to build the model geometry and its associated reference layers.

Table 10. Model Data Sources

Data Name	Data Type	Source	Notes
2017 FEMA Regulatory Models	Various	USACE SAC	1D HEC-RAS and HMS Models with GIS Datasets
2019 Update to FEMA Regulatory Models	Various	USACE SAC	2D HEC RAS Model and Mapping Updates, including Terrain File with 4ft raster cell resolution
Waccamaw River Bathymetry	GeoTIFF	USACE SAC	Collected by Coastal Carolina University in 2010, provided by USACE SAC, 50cm raster cell resolution
2020 LiDAR, North Carolina, Hurricane Florence	GeoTIFF	USGS	Obtained via RAS Mapper Terrain Downloader, 50cm raster cell resolution
2014 LiDAR, South Carolina, Horry County	GeoTIFF	SCDNR	Obtained from https://www.dnr.sc.gov/GIS/lidar.html, 4ft cell resolution

Bridge As-Builts	PDF	USACE SAC	21 Bridges
CONUS 2019 NLCD Land Cover Raster	GeoTIFF	USGS/MRLC	CONUS clipped to Pee Dee River Watershed
Ground Corrected MRMS Gridded Precipitation	GeoTIFF	Iowa State University	CONUS precipitation rasters, 1-hour increments
SSURGO Hydrologic Soil Group Raster Dataset	GeoTIFF	USDA/NRCS	CONUS clipped to Pee Dee River Watershed
NOAA Atlas 14	Temporal Distribution	NOAA	CONUS clipped to Pee Dee River Watershed
NC USGS LIDAR	GeoTIFF	NC	Obtained from NCSpatial Data Download 4ft cell resolution

3.2.1 Coordinate System and Datum

The modeling and associated spatial files were developed in the North American Datum 1983 (NAD 83), State Plane South Carolina in US Feet (FIPS 3900). The vertical datum used was the North American Vertical Datum of 1988 (NAVD 88).

3.2.2 Terrain

The terrain file used for the project model was generated from three lidar datasets and the bathymetry dataset provided by USACE SAC. Figure 26 shows the layout of the three terrain datasets and the resulting combined terrain file used for the modeling. The combined terrain was resampled to a 4-foot raster cell resolution.

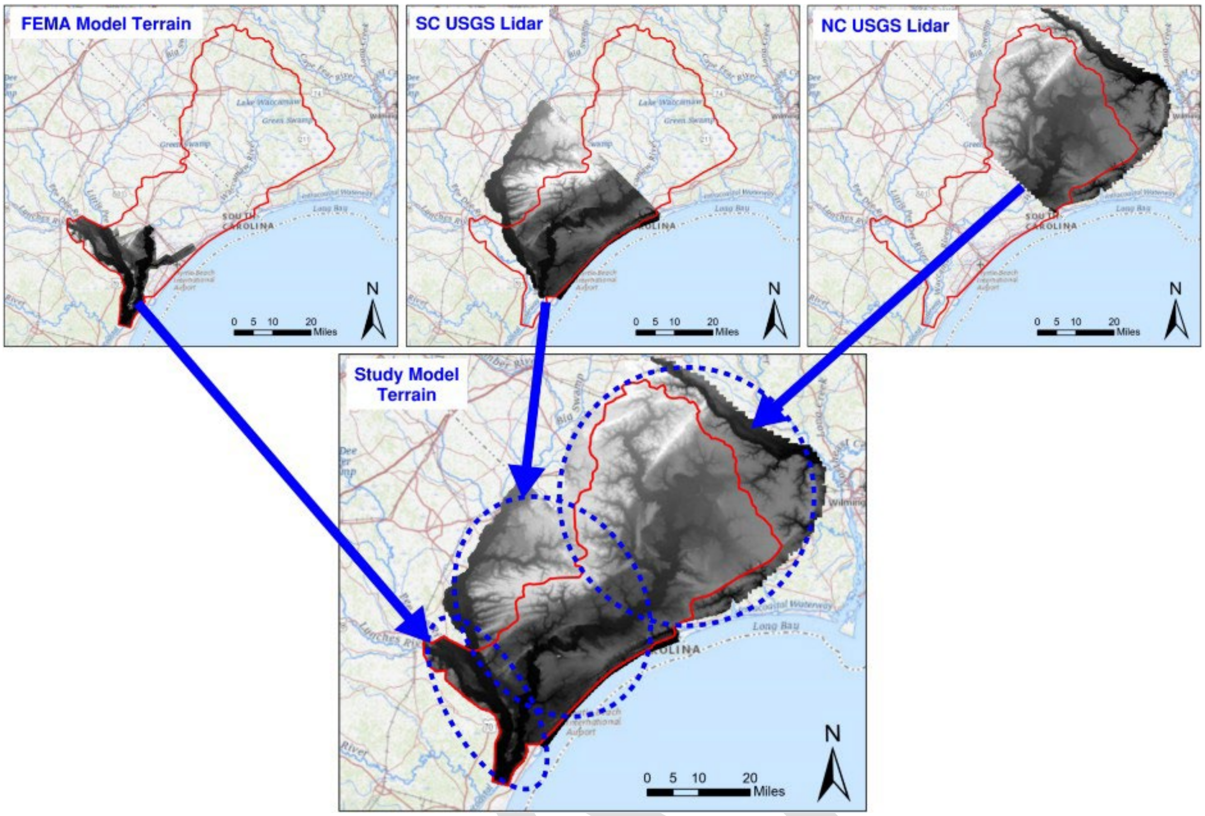


Figure 27. Lidar Data Used to Create the Model Terrain File

After combining the lidar data, bathymetric data was added to create the final model terrain file. Because the bathymetric data provided by USACE SAC only covered a portion of the Waccamaw River, supplemental bathymetry was developed to represent the approximate channel geometry below the water surface in main channel reaches that did not have bathymetry data. These terrain edits were performed using the terrain modification tools in RAS Mapper using available hydrography polygons and hand- digitized stream segments. The hydrography polygons were set to single elevations or offset from the lidar surface at estimated stream depths. The hand-digitized stream segments were sloped based on an estimated stream slope and used a trapezoidal section. Figure 27 shows the extent of each bathymetry type incorporated into the model terrain.

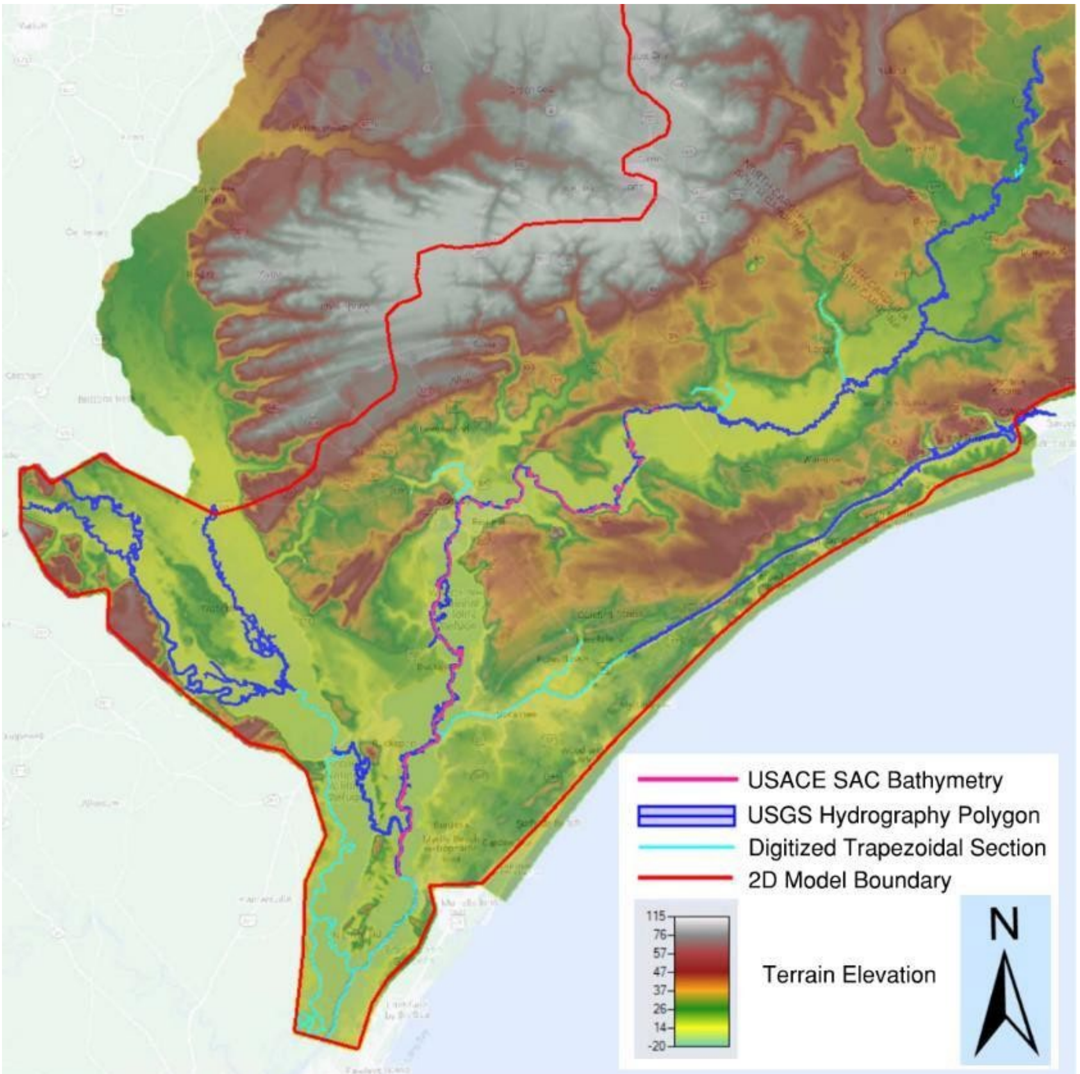


Figure 28. Bathymetric Data Extents

If a more detailed representation of the channel bathymetry is desired, additional survey would be required to collect the necessary data. Additional Bathymetry in the upper portion of the watershed measured by Coastal Carolina University was not completed in time to be used for these modeling efforts but could be incorporated in the future.

3.3 Structural Data

Most of the hydraulic structures within the study extents were based on FEMA hydraulic modeling provided by the South Carolina Floodplain Mapping Program. Hydraulic structure elevations and geometry in these models were based on detailed survey data. Other sources of bridge and culvert data were provided in structural as-builts from the South Carolina Department of Transportation and USACE Charleston District (SAC).

USACE SAC was provided the bridge as-built information for select bridges within the area of interest from SCDOT. Table 11 lists the floodplain crossings for the data provided by USACE SAC, which includes multiple bridges in some cases. The PDF file names associated with each bridge in the crossing are provided. Additionally, the bridges that were modeled and used for the bridge sensitivity check, discussed in the Model Sensitivity and Calibration section are marked with an asterisk.

Table 11. Bridge Data included in the model

Crossing	PDF File Name(s)
SC Hwy 31 over Waccamaw	S 26 31 Waccamaw River.pdf*
	S 26 31 Waccamaw Swamp.pdf
	S 26 31 Waccamaw Swamp 2.pdf
	S 26 31 Waccamaw Swamp 3.pdf
	S 26 31 Waccamaw Swamp 4.pdf
	S 26 31 Waccamaw Swamp 5.pdf
SC Hwy 105 over Waccamaw	S 26 105 Waccamaw River.pdf*
SC Hwy 616 over ICWW	S 26 616 ICWW.pdf
SC Hwy 9 over Waccamaw	SC 9 Waccamaw River and Swamp Bridges.pdf*
SC Hwy 22 over Waccamaw	SC 22 Waccamaw River.pdf*
	SC 22 Waccamaw Floodplain 1.pdf*
	SC 22 Waccamaw Floodplain 2.pdf*
	SC 22 Waccamaw Floodplain 3.pdf*
SC Hwy 31 over ICWW	SC 31 ICWW.pdf
SC Hwy 544 over ICWW	SC 544 ICWW.pdf
SC Hwy 905 over Buck Creek	SC 905 Buck Creek.pdf
SC Hwy 905 over Simpson Creek	SC 905 Simpson Creek.pdf
US BUS Hwy 501 over Waccamaw	US 501 BU Waccamaw River.pdf
	US 501 BU Waccamaw River Swamp 1.pdf*
	US 501 BU Waccamaw River Swamp 2.pdf
US Hwy 501 over Waccamaw	US 501 BY Waccamaw River.pdf*

4.0 Historic Events

4.1 Overview

NOAA's National Weather Service defines flooding as an overflowing of water onto land that is normally dry. In the Waccamaw Watershed, flooding occurs most often during and after rainstorms and hurricanes. Factors contributing to nuisance flooding include the city's location in the South Carolina Coastal Plain, 14 miles west of the Atlantic Ocean; being developed on the western banks of the Waccamaw River; and with relatively low elevations in relation to sea level. Flooding is the most frequent and costly natural hazard in the United States (EPA).

The types of flooding that Horry County generally experiences because of named storms or rain events are riverine and flash flooding. Riverine flooding is characterized by widespread rainfall across a river basin resulting in stormwater that accumulates in volume as it moves downstream (Horry County Flood Resiliency Plan). Flash flooding occurs when rainfall amounts exceed what can be absorbed or retained onsite, causing runoff that affects adjoining properties and streets. Flash flooding is felt immediately during and after a storm; however, it is seldom a multi-day event. In addition to riverine and flash flooding, compound flooding – when combined with riverine and flash flooding, increases the water table and the extent of flooding beyond what is expected from a single type of flooding. Conway is also considered to be within the coastal zone, and riverine flooding is exacerbated by tidal backwater flooding (South Atlantic Coastal Study (SACS)). A historic rain event can be described as a severe rain occurrence, whether associated with tropical storms, hurricanes or not, that results in major flooding in areas that may not have had flooding in prior years. Rain events, in combination with other factors, result in widespread flooding, drainage issues, and storm surges. The City of Conway frequently experiences flooding from rainstorms not associated with tropical storms or hurricanes. These storms occasionally result in structural damage; more often, road and park closures.

Table 13 provides a list of historic flooding events prior to 2015 in the Waccamaw River basin adapted from a recent SCDNR publication: South Carolina Extreme Events Timeline.

Table 12. List of Historic Flood Events compiled by SCDNR

Event Date	Quantified Impacts (state-wide)	Description
September 1752	95 deaths	Two Hurricanes Strike the Coast, with an estimated five-foot stage storm surge and 15 later in northeastern coast of SC.
June 6, 1903	$146 million damages and 65 deaths statewide	Major Flooding in the Santee River basin caused by flash flooding and waters rose to 40ft within an hour.
August 25, 1908		Statewide Flood; All major rivers in the state rose above the flood stage between 9 and 22 ft. Rainfall amounts from 10-13 inches recorded
September 1928		Okechobee Hurricane caused riverine flooding in the Pee Dee aggravated by extraordinary rain fall and high floods from tropical storm.
October 15, 1954	1 life lost in Horry County, 95 deaths in total path, $50 million damages	Hurricane Hazel, Category 4 at Cherry Grove landfall 130mph winds 14 ft-15 storm surge
September 1959	Damage $58 million	Hurricane Gracie rainfall, primarily affected southern portion of the state but riverine induced flooding.
September 1989	Lives lost, 25; damages, $2.4 billion	Hurricane Hugo, causing extensive damage in Charleston but rainfall induced riverine flooding

4.2 October 2015 Flooding (Hurricane Joaquin)

A record setting and historic rainfall event occurred October 1st through 5th, 2015, producing widespread and significant flooding across much of South Carolina. All–time precipitation records were shattered from the midlands to the coast, with totals ranging from 10 to over 26 inches of rain (Figure 28). Streams and creeks swelled out of their banks with 17 U.S. Geological Survey (USGS) gages reaching record peaks. The event was the worst flooding most residents had ever experienced. Emergency responders worked tirelessly with over 1,500 water rescues. The flooding displaced over 20,000 citizens, closed over 500 roads and bridges, resulted in 47 dam failures, disrupted drinking water supply to over 40,000 residents and tragically took the lives of 19 people.

Rainfall Totals (Sep 30 – Oct 7)	Station	County
27.19"	Charleston 6.4 NE	Charleston
23.88"	Georgetown County Airport	Georgetown
23.68"	Kingstree 9.5 NW	Williamsburg
22.59"	Sumter	Sumter
20.97"	Moncks Corner 3.6 E	Berkeley
19.74"	Summerton 8.4 SE	Clarendon
18.17"	Coward 5.1 NNW	Florence

Figure 29. Rainfall data and flooded properties from October 2015 Flood (Hurricane Joaquin) (SCDNR)

The rainfall amounts and distributions across the State were similar in pattern to those normally produced by hurricanes making landfall; however, although the moisture drawn over the State was from deep in the tropics, the synoptic features, or mechanism, that produced the heavy rainfall was of a mid-latitude nature rather than that of a tropical cyclone. Mid-latitude features include surface and upper level high- and low-pressure features, warm fronts, and cold fronts, as well as ridges and troughs that exist due to differences in temperature and moisture content. The heavy rains and subsequent catastrophic flooding occurred a week after heavy rainfall across the state. On October 1, a cold front swept across the state and stalled offshore for the next five days. This boundary tapped into deep tropical moisture over the Gulf of Mexico as it sat offshore in the Low Country. At the same time, Hurricane Joaquin rapidly deepened over the Bahamas and interacted with the stalled coastal front, providing additional moisture into the region.

All-time precipitation records were shattered with rainfall totals ranging from 10 to over 26 inches from the Midlands of SC to the coast, with 12-24 inches of precipitation over the Waccamaw River Watershed (Figure 31).

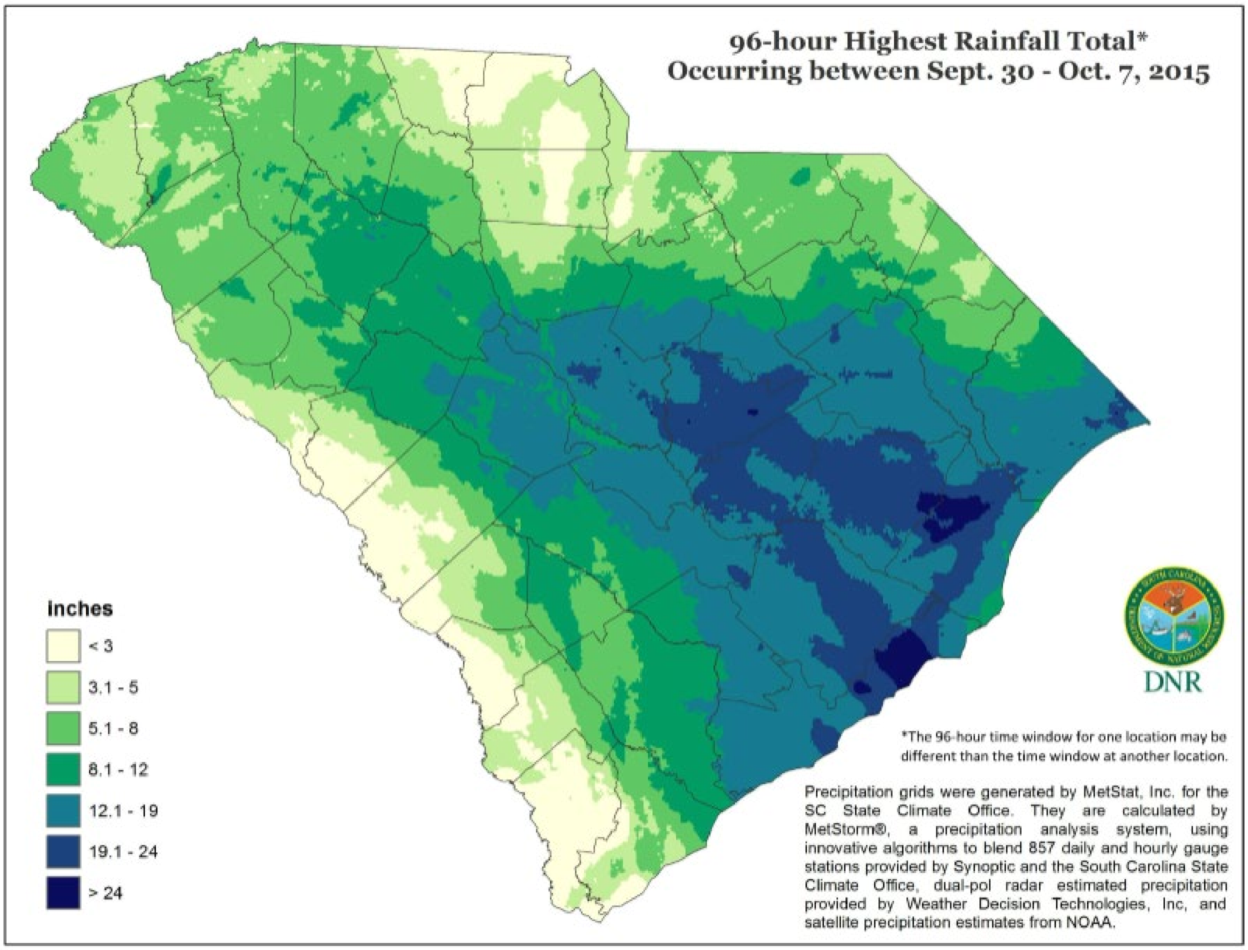


Figure 30. 96-hour Highest Rainfall Totals, Sept. 30 - Oct. 7, 2015 (SCDNR)

Streams and rivers swelled out of their banks and 17 USGS gages reached record peaks including the Black River at Kingstree (Table 13) and Conway Marina (Figure 32).

Table 13. October 2015 Peak Flows at Selected Gages and Compared to Historical Record

Gage	Peak Stage (ft)	Peak Flow (cfs)	Record Stage and Flow
Black River at Kingstree	22.65	83,700	1973 (19.77 ft; 58,000 cfs)
Waccamaw River near Longs	15.17	16,900	1999 (17.94ft; 28,200 cfs)
Pee Dee River	22.81	30,100	1945 (33.3 ft; 220,000 cfs)

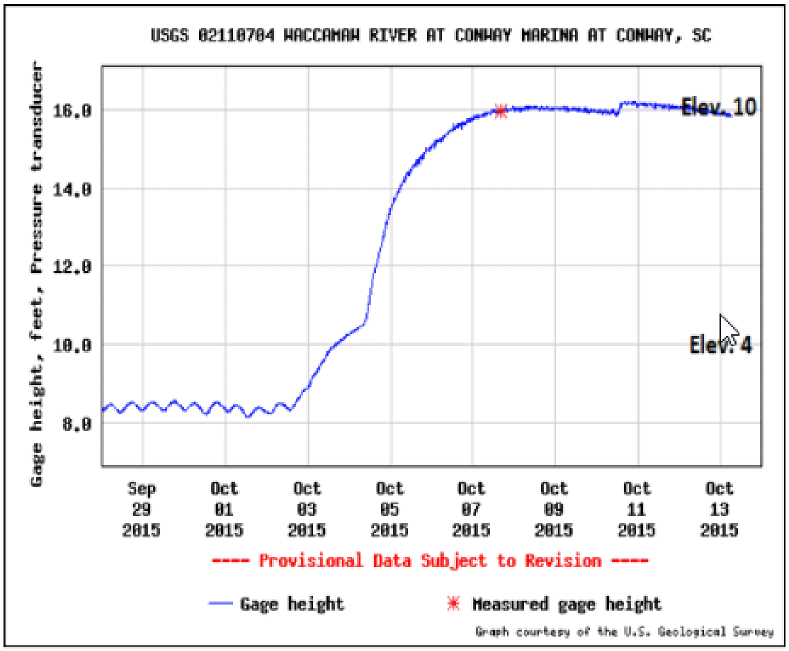


Figure 31. Gage number 02110704 during Hurricane Joaquin

4.3 Hurricane Matthew

In the fall of 2016, Hurricane Matthew caused significant damage to the States of North Carolina, South Carolina, and Georgia, both in economic and life-safety terms. On October 8, Hurricane Matthew made landfall near McClellanville, SC as a Category 1 hurricane. Matthew caused severe beach erosion, and hurricane-force gusts downed thousands of trees along the coast and well inland. The remnants of Matthew dumped 10-17 inches of rain from Savannah, Georgia through Florence, South Carolina, and into a wide area of eastern North Carolina. The most widespread heavy rain fell in the Pee Dee Basin and into North Carolina, where significant flooding occurred. Rainfall totals across portions of the Pee Dee surpassed the record rainfalls in the basin, including "Bulls Bay Hurricane" in 1916 and Hazel in 1954 (Figure 31).

Rainfall Totals (Oct 8 - 10)	Station	County
17.11"	Dillon 3.8 NW	Dillon
15.06"	Mullins	Marion
14.10"	Daufuskie Island 1.7 SW	Beaufort
13.56"	Kingstree 7.9 NW	Williamsburg
13.05"	Reevesville 1.0 SSE	Dorchester
12.39"	NWS Charleston	Charleston
11.32"	Yemassee 1 N	Hampton

Figure 32. Flooding due to Hurricane Matthew with rainfall data during the event (SCDNR)

On October 9, the Lumber, Little Pee Dee and Waccamaw rivers had swelled to a “Major Flood Stage” and were rising. On October 12, the Little Pee Dee River at Galivants Ferry rose to 17.10 ft. The town of Nichols was submerged under the adjacent Lumber River floodwaters. Non elevated property along the Waccamaw River near and below Conway had to be abandoned. The Waccamaw River near Conway reached a record stage of 17.89 ft on October 18 surpassing the flood of September 1928. Many riverside docks and decks, private or state owned had been swept away. On November 2, after 25 days above its flood stage (11ft) the Waccamaw River near Conway subsided to normal levels (Figure 33).

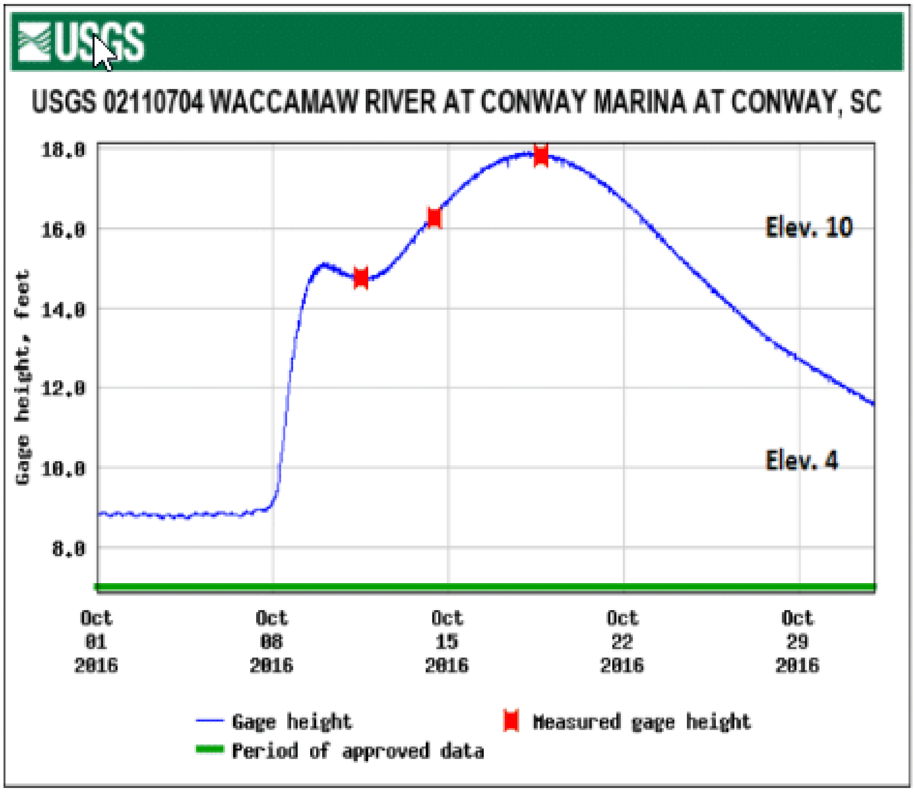


Figure 33. USGS gage 02110704 during Hurricane Matthew

The event resulted in damage estimates in South Carolina and North Carolina that exceeded $1.5 billion and nearly 30 deaths were attributed to the hurricane (SC Keystone Flooding Event). A roughly 15-year period of quiet tropical storm activity in much of the Waccamaw River basin, following the devastating 1999 Hurricane Floyd event, was abruptly ended in October of 2016. Figure 35 shows the rainfall accumulation from Hurricane Matthew along the southeastern portion of the United States.

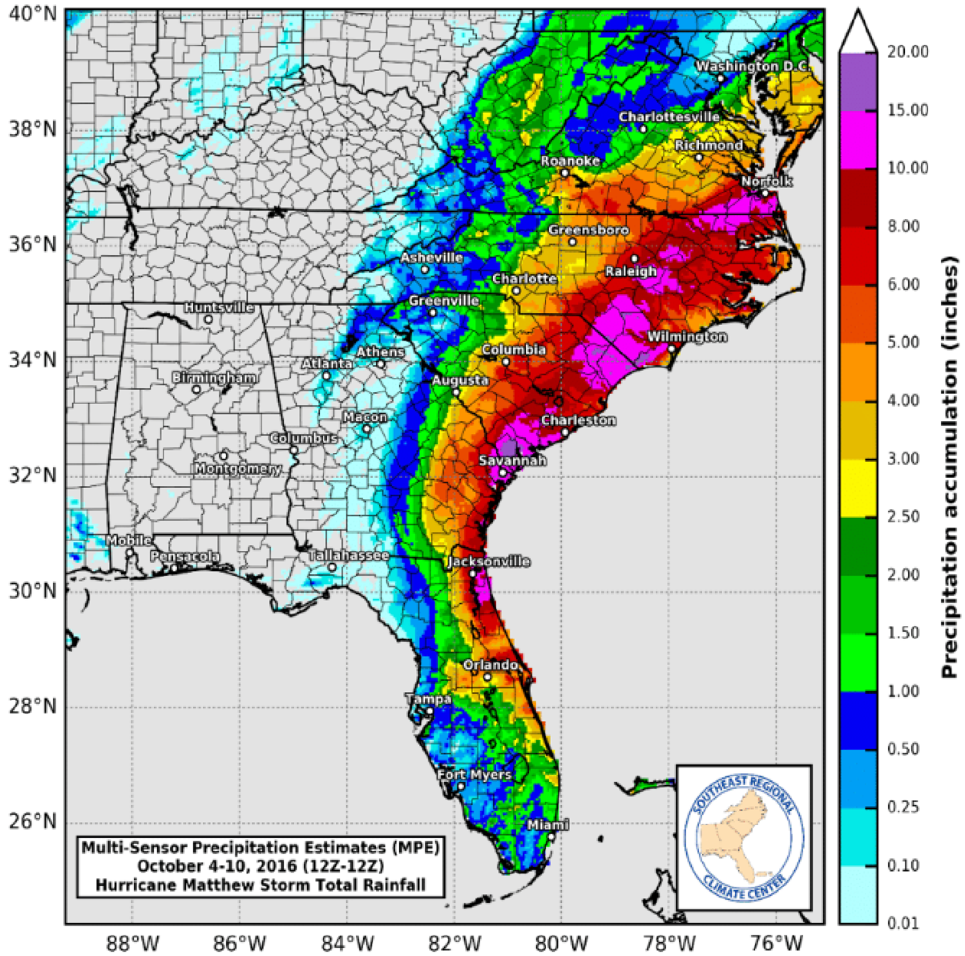


Figure 34. Precipitation Estimates during Hurricane Matthew

4.4 Hurricane Florence

Hurricane Florence slowly approached the coast of South Carolina after periods of rapid intensification and weakening that had allowed it to strengthen to a category 4 storm on September 12, 2018. Outer rain bands initially reached the lower portions of the Waccamaw River basin with consistent wind gusts near 40 to 50 mph and gusts of 60 to 70 mph measured over the Pamlico Sound. Tornado warnings were issued for the lower basin. While Florence did weaken to a category 1 storm when it made landfall on September 14, 2018, along the southeastern coast of North Carolina, threats from its forecast was not necessarily based on intensity but on overall storm size. The storm’s large circulation caused a significant storm surge despite its low category strength, especially when combined with heavy rainfall due to its slow movement. The overall character of the hurricane had a well-defined eye but with only a partial eyewall on its western side due the storm’s large size. The storm’s path had a stair-stepping pattern near the coast due to the wobbling inner eye trying to center within a broader outer band. This pattern caused the storm to stall at intervals as it traveled west which produced prolonged precipitation over the basin.

The storm’s direction shifted in a southerly direction once it made landfall which further increased the rainfall totals across its northwest outer bands. The New Bern, NC airport reported a 5-day total rainfall of over 17 inches between 12-September and 17- September. 5-day total rainfall in the Kinston, Farmville, and Raleigh-Durham areas were reported at approximately 19, 13.5, and 9 inches, respectively (SC ACIS, 2022). Hurricane Florence observed precipitation is shown in Figure 36.

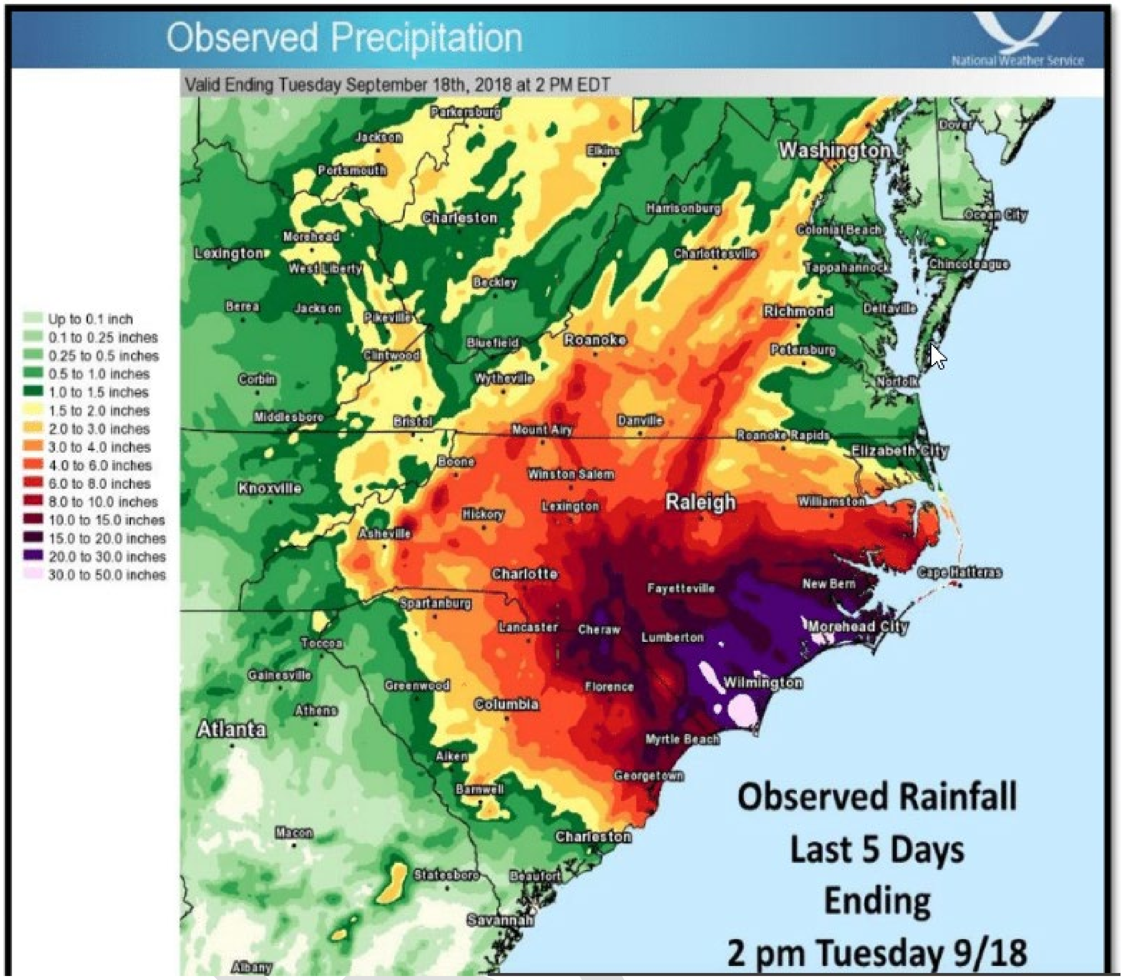


Figure 35. National Weather Service - Hurricane Florence Observed Precipitation

Florence was a Category 1 Hurricane when it made landfall near Wrightsville Beach, North Carolina, on September 14. It proceeded to stall and remain nearly stationary for an entire day before it began a slow turn to the southwest, which is not a typical movement for tropical cyclones. It traveled across South Carolina at a speed of 2-3 mph. The storm continued to weaken during the 15th and accelerated to the north-northeast and out of the state on September 16. The slow-moving system dropped more than 30 inches of rain across portions of eastern North Carolina and over 20 inches in Chesterfield and Horry counties.

While Florence was a coastal storm, the severe impacts felt by Horry County were primarily from inland flooding that took place in the days and weeks after the hurricane made landfall. Storm surge was relatively minor along the Grand Strand in Myrtle Beach, with minimal surge inundation reported. However, roughly 80,000 residents were without power across the Grand Strand area during the storm. The maximum storm tide was measured at Surfside Beach and was approximately 6.4 feet above mean (average) sea level. The significant levels of rainfall in both North and South Carolina from the storm that landed upstream of Horry County, slowly flowed down the drainage basins, merging with already flooded rivers and streams. While streams in the County began to rise just after Florence made landfall, the Pee Dee and Waccamaw Rivers in Horry County did not crest until September 26, twelve days after landfall and eight days after the storm had dissipated over New England. Rivers continued to crest downstream over the next several days. The Waccamaw River crested at its upstream gauge near Longs on September 21, near Conway on September 26, and at its downstream gauge near Bucksport on September 27. Similarly, the Little Pee Dee River crested upstream at Galivants Ferry on September 21, and downstream on the Pee Dee River near Bucksport on September 27. Table 14 lists flood crests from Florence compared to previous flood crests (SCDNR).

Table 14. Florence vs Historical Crests at Selected USGS Gages (Horry, 2019)

River Gauge	Florence Crest (ft.)	Previous Crest (ft.)	Previous Crest Data/Event
Waccamaw at Longs	20.22	17.94	9/22/1999 Hurricane Floyd
Waccamaw above Conway	19.82	15.77	10/16/2016 Hurricane Matthew
Waccamaw at Conway	21.16	17.87	10/18/2016 Hurricane Matthew
Pee Dee at Bennettsville	94.25	89.94	04/12/2003
Black Creek Near Quinby	17.37	16.81	10/05/2015 October Floods
Little Pee Dee at Galivants Ferry	17.21	17.10	10/12/2016 Hurricane Matthew

Historic peak gage height (ft) data shows that Hurricanes Florence (2018), Matthew (2016), Joaquin (2015), and Floyd (1999) resulted in four of the highest five crests recorded in the area. Many stream gauges in the region set new records for flood elevation, exceeding those set by Hurricane Matthew in 2016. Record flooding was documented at several USGS stream gauge locations in Horry County, including the Little Pee Dee River at Galivants Ferry, the Pee Dee River at Bucksport, and the Waccamaw River at Longs and Conway Marina. The gages along the Little Pee Dee/Pee Dee Rivers recorded peak water-level rises approximately 14 to 16 feet above normal and gages on the Waccamaw River recorded rises of around 13 to 19 feet above normal. Along the Intracoastal Waterway (near the confluence with the Waccamaw River at Socastee), gauges recorded peak water-level rises of approximately 9 to 10 feet above normal (USGS, 2016).

The extensive and prolonged flooding in Horry County during Florence was due to a combination of widespread unprecedented rainfall across the entire Pee Dee drainage basin that was further exacerbated by the low elevation and relief of the landscape (flat land near sea level) and the fact that the outfall to the Atlantic Ocean is more than 30 miles further south (at Winyah Bay). As a result, the stream channels were unable to accommodate and quickly drain the excessive rainfall.

For the inland communities in Horry County, such as Loris, flash flooding caused by the storm's record rainfall was the primary issue during Florence. The community of Dongola in western Horry County was isolated by flooding for ten days. Flood levels of up to eight feet were registered in communities south of Myrtle Beach near the Intracoastal Waterway. Trees were blown down by high winds across the northern portion of Horry County. Flooding from Florence caused major damage to infrastructure. The Horry County post-storm assessment documented approximately 2,000 buildings with flood damage. The total market value of properties (parcels) with flood-damaged buildings has been estimated at $400 million. While approximately 2,000 buildings were damaged during Florence, just under 400 Florence related permits have been received (including residential and commercial buildings) in the unincorporated area of the County, with 34 of these permits to elevate the building and 40 to demolish. There are numerous properties that remain in disrepair.

Unprecedented flooding occurred in Florence's wake, as a portion of the excessive amount of rainfall measured in North Carolina fell in the Yadkin-Pee Dee River watershed. For weeks after the initial landfall, flooding plagued most of the Pee Dee Region, with significant impacts along the Pee Dee, Little Pee Dee, Lumber, Lynches, and Waccamaw rivers and their tributaries. Many of these river gauges reached crest values that fell within the top five highest measured crests at their locations, while several of the rivers set new record crest values. The Pee Dee River at Pee Dee reached a height of 31.83 ft. during the flooding, which was 1.5 ft. lower than the historic crest of 33.3 ft. in 1945. Gauges along Waccamaw exceeded previous record crests by three or more feet during this event. Figure 36 shows USGS gage 02110704 for the Waccamaw River at Conway. Notice the second peak was almost 1.6 times the initial peak. This effect was caused by the additional riverine flooding from the Pee Dee Diver with backwater effects.

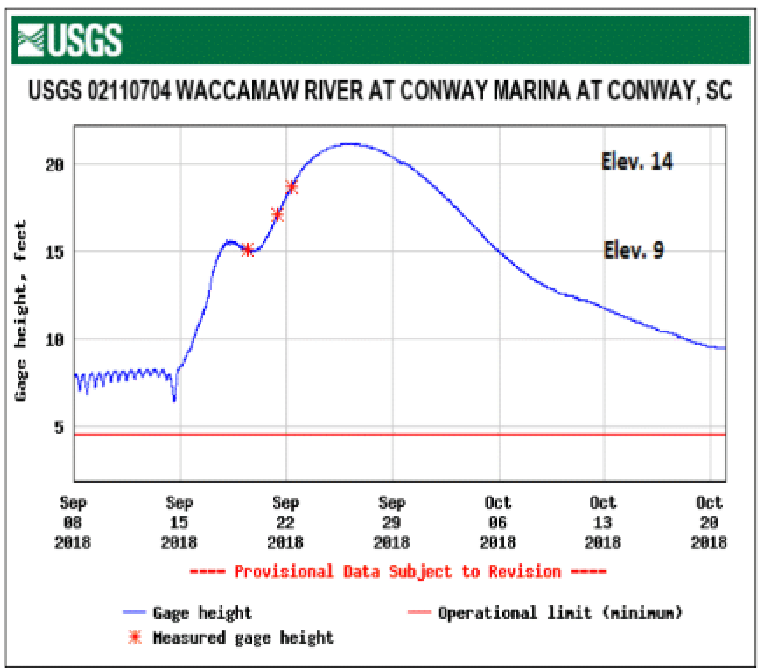


Figure 36. USGS Gage 02110704 Waccamaw River at Conway gage height during Hurricane Florence

4.4 Hurricane Debby

Hurricane Debby was a slow-moving and erratic Category 1 hurricane that caused widespread flooding across the Southeastern United States in early August 2024. The fourth named storm and second hurricane of the 2024 Atlantic hurricane season, Debby developed from a tropical wave that was first noted by the National Hurricane Center (NHC) on July 26. After crossing the Greater Antilles, the system began to organize over Cuba and was designated a potential tropical cyclone on August 2. After exiting off the southern coast of Cuba, the disturbance organized into a tropical depression early on August 3. Later that day, it became a tropical storm in the Florida Straits, being named Debby. It moved northwards and gradually intensified into a Category 1 hurricane before making landfall near Steinhatchee, Florida, early on August 5. Debby weakened once inland and began to slow down over the Southeastern United States, causing widespread flooding from heavy rain. It re-emerged in the Atlantic on August 7 before slowly moving northwards again, making landfall in South Carolina early on August 8 before weakening and becoming post-tropical the next day (NOAA 2024).

States of emergency were declared for the states of Florida, Georgia, and North and South Carolina ahead of the storm. Heavy rains fell as a result of the storm moving slowly, with accumulations peaking near 20 inches (51 cm) of rain near Sarasota, Florida as of August 7. Two dozen tornadoes were confirmed as the storm also moved up the East coast of the United States. Ten fatalities have been attributed to the storm, and preliminary damage reports are estimated to be up to $2 billion. In Horry County, flooding was a result of the extensive rainfall in the northernmost portion of the study area. Accumulated rainfall totals as of August 8th are shown in table 15.

Table 15. Rainfall Totals for Hurricane/ TS Debby in Horry County

Location	Rainfall Totals (in)
Loris	15.89
North Myrtle Beach	14.25
Little River	13.06
Horry County Police	12.57
	85
Garden City	10.35
	10.21
Allsbrook	9.52
iffe A	9
Galivants Ferry Land	9.07
ols	
Red Hill	8.65
Swa	
North Conway	8.19
tion	
Central Horry County	7.3
Socastee	6.02
Longs	4.72

The effects of the flooding are still being experienced in the Socastee, Conway and Bucksport areas as gages are showing a major flood stage is in the Pee Dee River, Socastee along the AIWW and Conway river gages.

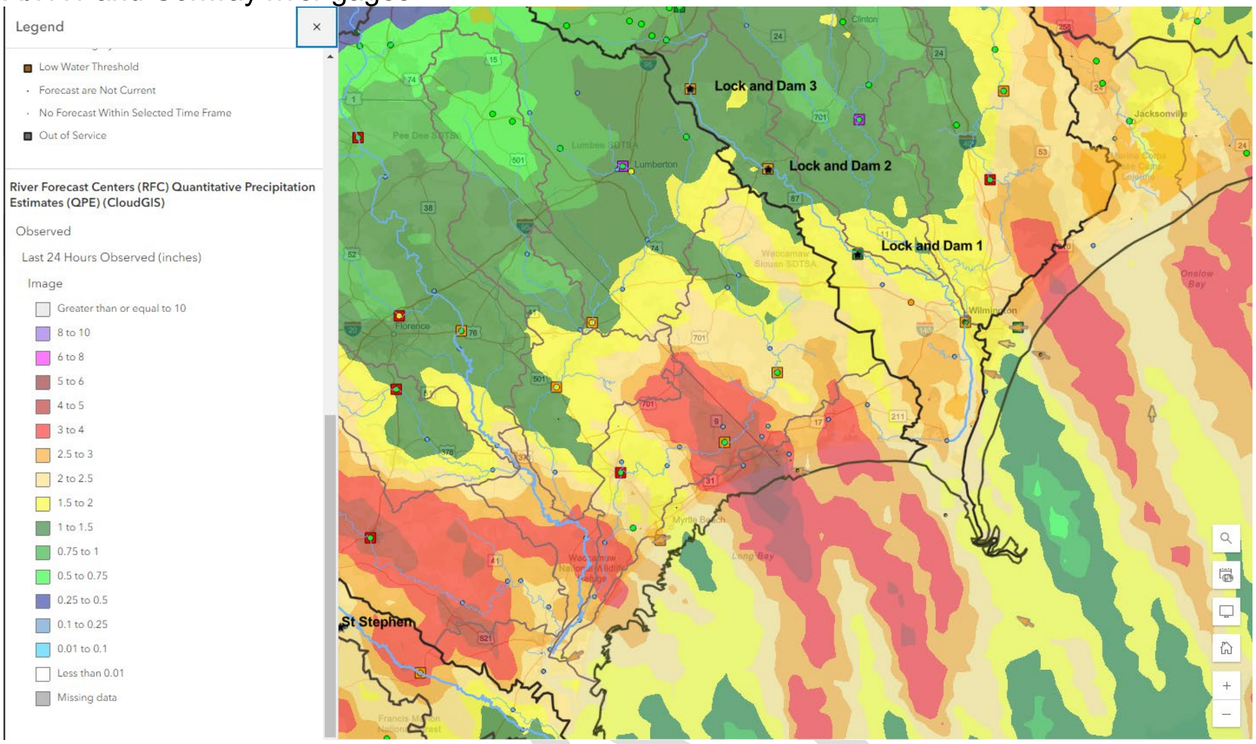


Figure 37. 24 Hour Quantitative Precipitation Estimate (Aug. 6th, 2024)

As of August 20, 2024 the Waccamaw River near Conway is showing a peak of 14.90 ft, which is a Major flood stage as shown in Figure 37.

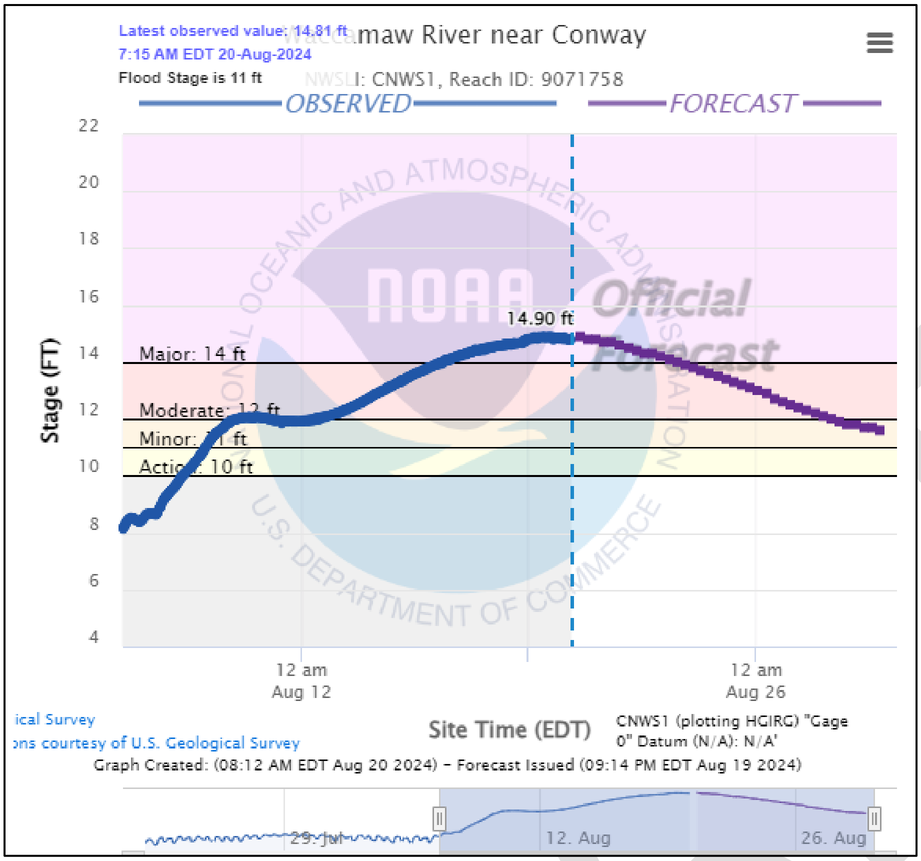


Figure 38. River Forecasting Center report for USGS Gage Waccamaw River near Conway as of August 20, 2024

Similar to Hurricane Florence and Matthew an additional and larger peak was observed on August 10, at 12.06ft and then receded and another peak was measured at 14.90 ft on August 19th. The gage information is shown in Figure 38. Flooding effects are still being experienced at the time this report was written.

4.5 Summary of Historic Events

The historic flooding events affecting the Waccamaw River have proven to be a severe threat to the residents of Horry County. The flooded and closed roadways and days it took for the storms to recede negatively impacted the livelihood of most people. The three events that occurred since 2015, Joaquin, Matthew and Florence were three events that were validated with the Hydraulic model. Each one of these events were unique with intensity, duration and impact. The second peak that is observed in both Hurricane Matthew, Debby, and Florence are indicative of the flooding from the Pee Dee River and its effect on the Waccamaw River. The second peak is significantly higher than the initial peak. This can be observed in Figure 33, Figure 36 and Figure 38. Hurricane Joaquin was a unique storm because of the rainfall that led to the saturated soils, and that there was not a second peak from the backwater effect from the Pee Dee because this event was a "firehose" to the coast and midlands of South Carolina, causing riverine flooding and dam failures in the midlands. The calibration modeling results of these events is in the Hydraulic Engineering modeling section of this appendix.

5.0 Existing Conditions

5.1 Hydrology

The Waccamaw River, the primary water body in the watershed, is a slow-moving blackwater river that meanders through the landscape. Its flow is influenced by precipitation, tides, and groundwater inputs. During periods of heavy rainfall, the river can experience significant increases in water levels, leading to flooding in low-lying areas. The five main hydrologic features of the watershed area are; wetlands and swamps, diverse ecology, human impacts and urbanization, and recreational potential. The watershed contains extensive wetlands and swamps, which play crucial roles in regulating water flow and quality. These wetlands act as natural sponges, absorbing excess water during storms and releasing it slowly over time, thereby reducing the risk of flooding downstream. Additionally, they filter pollutants and nutrients from the water, improving water quality.

The Waccamaw River Watershed is home to a diverse array of plant and animal species, many of which depend on the unique hydrological conditions provided by the wetlands and rivers. These habitats support rare and endangered species, including various fish, birds, and reptiles. Like many watersheds, the Waccamaw River Watershed faces threats from human activities, including urbanization, agriculture, and industrial development. These activities can lead to habitat loss, water pollution, and altered hydrological patterns. Conservation efforts, such as land preservation, restoration projects, and water quality monitoring, are essential for protecting the health and integrity of the watershed. The Waccamaw River Watershed provides numerous recreational opportunities for residents and visitors, including boating, fishing, birdwatching, and hiking. These activities rely on the health of the watershed and its waterways, highlighting the importance of sustainable management practices.

Overall, the hydrological aspects of the Waccamaw River Watershed are integral to its ecological health, biodiversity, and the well-being of surrounding communities. Protecting and managing these resources effectively is essential for maintaining the watershed's resilience in the face of environmental challenges

The Waccamaw River watershed includes 1,640 square miles within North and South Carolina. Its headwaters are in North Carolina and the river originates at Lake Waccamaw, a permanently inundated Carolina Bay managed as Lake Waccamaw State Park. The Waccamaw River is a coastal plain river with extensive wetlands that leach pigments, such as tannins, causing its dark coloration and description as a blackwater river. This blackwater river flows over 140 miles through North and South Carolina. Along the way, the Waccamaw joins with the Atlantic Intracoastal Waterway in South Carolina, then with the Pee Dee River before it empties into the Winyah Bay estuary at Georgetown, SC. The Waccamaw River watershed (hydrologic unit code 03040206, area=311,685 ha) is on the lower Coastal Plain of eastern North and South Carolina. The watershed has little topographic gradient (99% is <5% slope), wide floodplains, and complex groundwater characteristics due to poorly drained soils, a shallow water table, and extensive wetlands. Elevation ranges from 6 to 46 m above mean sea level. The watershed is in a humid sub-tropical climate with hot summers and mild winters. Precipitation in the basin falls almost exclusively as rainfall, with an annual average of 1,309 mm during the study period (2003-2007). Streamflow data from two U.S. Geological Survey (USGS) gaging stations, at Freeland (34°05042N, 78°32054W) and Longs (33°54045N, 78°42055W), were used as sub watershed outlets.

Waccamaw land use information was obtained from USGS National Land Cover Data portal on September 13,2022 (http://viewer.nationalmap.gov/viewer/). NLCD became available in July of 2023. NLCD 2019 was incorporated into the model. NLCD 2021 was not used because it became available mid-study in July of 2023. Forested wetlands were the dominant land use, occupying approximately 28% of the watershed. Agricultural uses were 26% and developed uses (residential, commercial, and industrial) were 5%. Approximately, 90.5% of the soils are one of four series, all of which are either hydrologic groups B, D, or B/D. Only 9.5% of the soils are hydrologic group A; there are no group C soils (Table 2). Hydrologic group D soils (poorly drained) are adjacent to the main channel and hydrologic groups B and B/D. The Waccamaw River Watershed, located in the southeastern United States, encompasses a diverse range of hydrological features and processes. The watershed covers parts of North and South Carolina and is characterized by its unique mix of wetlands, swamps, and rivers, making it an ecologically significant area.

5.1.1 Hydrology Model Background

A hydrologic model was developed to assess existing conditions in the Waccamaw River basin, using the USACE Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) software, version 4.11. Given the Waccamaw River basin's large size and number of tributaries, as well as variety in urban landscape, it was decided that the rain-on grid feature in HEC-RAS would best serve the intent in formulating local flood risk management measures. A hydrologic sensitivity analysis was conducted to see which hydrologic model would replicate the hydrologic features of the Waccamaw River Watershed. One comprehensive basin model was developed for hydrologic assessment along the mainstem of the Waccamaw River as well as the following headwaters and major tributaries: Pee Dee River, Little Pee Dee River, Buck Creek, Socastee Creek, Simpson Creek, Crabtree Swamp, and Atlantic Intracoastal Water Way. The large footprint of this model would provide the ability to evaluate basin-wide flooding concerns and associated opportunities. Its development priority would also help direct future modeling needs as plan formulation progressed through the feasibility process.

For this study, the Pee Dee effective FEMA HMS models were utilized. The rainfall parameters (depth, distribution, duration, ARF) were adjusted and the models were re-run with the new inputs. Calibration and validation were not performed for the existing HMS models as part of this study. A FEMA HMS model development report was not available for review of their approach, but the data and parameters in the model were relatively straightforward and appeared to be reasonable based on our cursory review. The purpose of the original HMS model was determining effective flows for development of the regulatory floodplain.

Based on sponsor and community input at the onset of this feasibility study, as well as recently completed/ongoing related basin studies, several specific locations within the study area were highlighted. The availability of existing subbasin modeling also provided either a good starting point or in one instance, a significant modeling effort that already detailed existing and future without project conditions. Furthermore, the highly urban characteristics of some of these subbasins created inconsistencies in the modeling approach assumed for the larger basin-wide model. Complex watersheds such as Crabtree Swamp required much smaller subbasin delineations in area to account for the high density of streams, impoundments, and confluences. A basin wide HEC-HMS model was developed in parallel with the rain on grid approach encompassing the four areas of interest: Socastee, Longs/ Red Bluff, Conway, and Bucksport.

The Rain on Grid approach, also known as the Rainfall-Runoff Grid approach, is a method used in hydrological modeling to simulate rainfall and its resulting runoff within a specific area. This approach is often implemented using HEC-RAS. In the Rain on Grid approach, the study area is divided into a grid of smaller cells, with each cell representing a portion of the watershed. Rainfall data, typically obtained from rain gauges or radar, is applied to each grid cell individually. This allows for spatially distributed rainfall inputs, accounting for variations in precipitation across the watershed.

HEC-RAS utilizes the Rain on Grid approach to simulate how rainfall is transformed into runoff, considering factors such as infiltration, surface runoff, and channel flow. The software calculates runoff volumes and flow rates for each grid cell, accounting for factors such as land use, soil type, topography, and vegetation cover.

By simulating rainfall and runoff at a high spatial resolution, the Rain on Grid approach provides more detailed and accurate representations of hydrological processes compared to traditional lumped models. This allows for a better understanding of how rainfall events impact the flow of water through the Waccamaw River watershed, including potential flooding risks and the effectiveness of mitigation measures. Overall, the Rain on Grid approach using HEC-RAS is a powerful tool for hydrological modeling, offering insights into watershed dynamics and informing decision-making for water resource management, flood forecasting, and infrastructure planning.

5.1.2 Model Overview

HEC-RAS version 6.4 was used to assess the Waccamaw River watershed hydrology and hydraulics. HEC-HMS version 4.11 was used to develop hydrologic inputs for the HEC-RAS boundary conditions associated with the Pee Dee River watershed. The FEMA HEC-HMS model provided by USACE SAC was used to compute the Pee Dee River inflow boundaries. Hydrology computations for the Waccamaw watershed were performed using the HEC-RAS 6.4 2D rain-on-grid approach. The 2D rain-on-grid approach was chosen to consolidate hydrology and hydraulics into one model for the Waccamaw River basin. Furthermore, the single model approach facilitates streamlined model calibration and flexibility when performing future hindcast simulations.

Rainfall losses were computed using the Natural Resource Conservation Service (NRCS) curve number method. The curve numbers were generated using the National Land Cover Database's (NLCD) 2019 Land Cover raster and the October 2021 Soil Survey Geographic Database (SSURGO), from which the hydrologic soil group (HSG) was obtained. An abstraction ratio of 0.2 and a minimum infiltration rate of 0.001 inches/hour were used to determine rainfall losses. Table 2 in Section 2.4 provides the curve numbers for each land cover and soil type combination.

Synthetic rainfall events were developed to assess watershed's response for the 50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.2% annual exceedance probabilities (AEPs), also known as the 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year storm events, respectively. The rainfall depths used to develop the rainfall hyetographs were calculated in HEC-HMS using the Volume 2 (Ohio River Basin and Surrounding States) NOAA Atlas 14 GIS grid atlas, which contains gridded datasets for each AEP. The annual maximum precipitation values calculated from these grids within HEC-HMS is shown for each basin in Table 4 in Section 2.5. Early coordination with the HHC PCX, guided the PDT to analyze and determine the duration of storm within the region. The charts of accumulated precipitation, shown in figure 37 and 38, for Hurricane Florence and Joaquin, for events used for calibration seem to indicate that a typical storm is around a 3 or 4-day duration. The results of the existing model indicates that the results were highly sensitive to the initial flow assumptions, which also indicates a 24 hour duration is not sufficient, therefore a 96 hour storm was selected for this region to accurately depict the events.

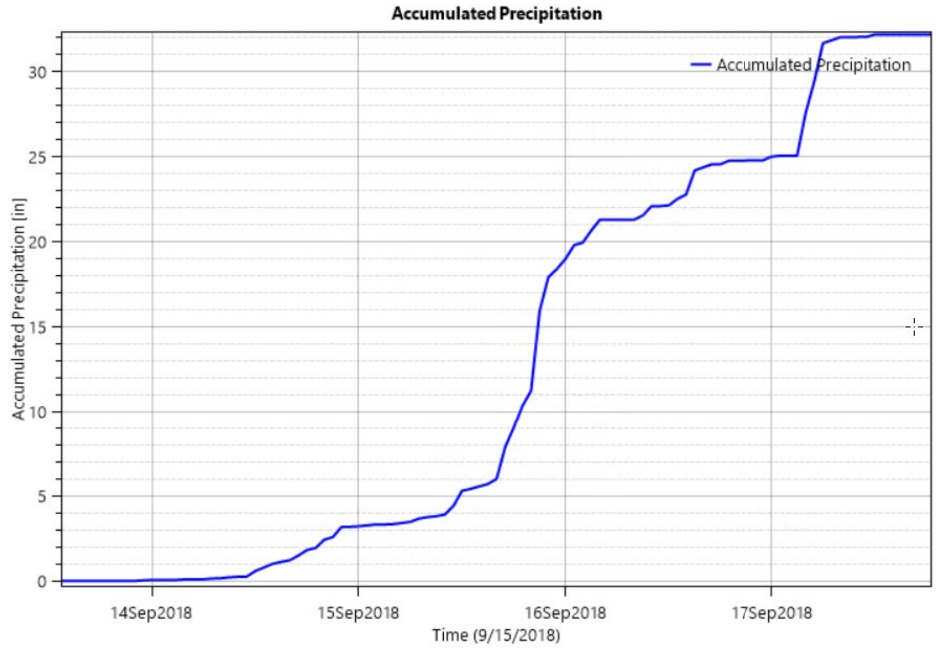


Figure 39:Approximate Maximum Accumulated Precipitation Point for Hurricane Florence

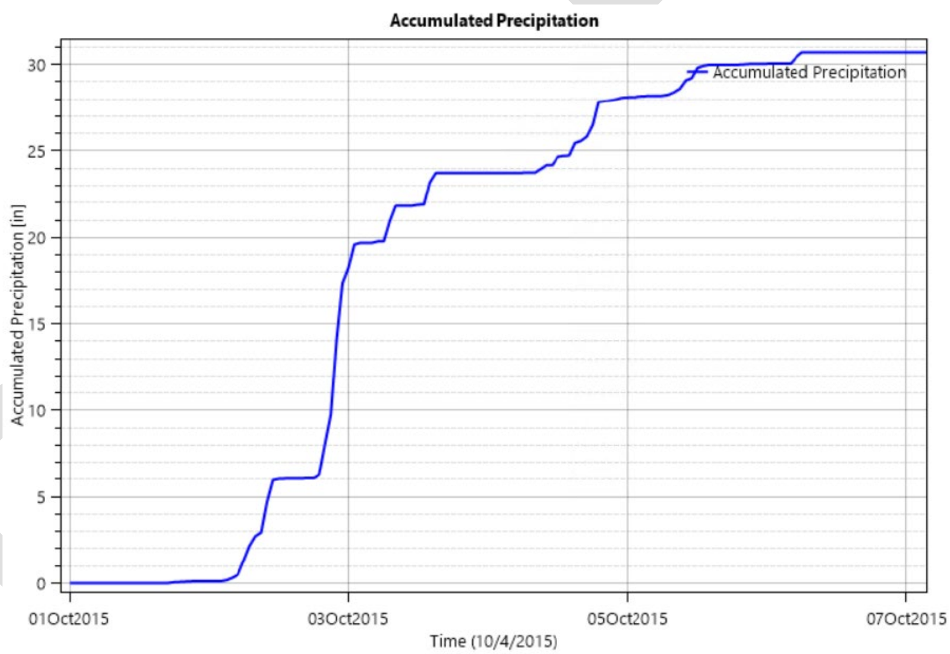


Figure 40: Approximate Maximum Accumulated Precipitation Point for Hurricane Joaquin

A 96-hour storm duration with a NOAA Atlas 14 Quartile 4, 90% decile rainfall distribution was utilized to generate the rainfall hyetographs for each sub-basin in the HEC-HMS model for the synthetic event simulations. The unit hyetograph used in HEC-HMS is shown in Figure 36.

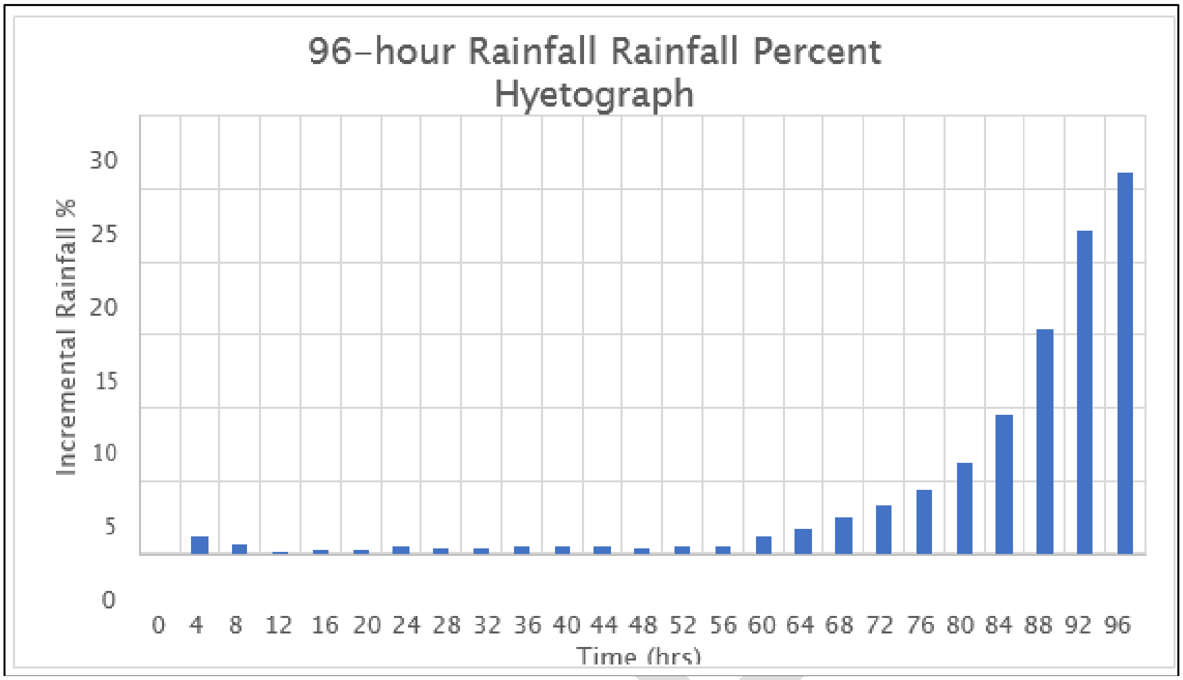


Figure 41. Rainfall Percent Hyetograph for 96 Hour NOAA Atlas 14, Quartile 4 90% Decile Event

Hydrology computations for the Waccamaw watershed were performed using the HEC- RAS 6.4 2D rain-on-grid approach. The 2D rain-on-grid approach was chosen to consolidate hydrology and hydraulics into one model for the Waccamaw River basin. Furthermore, the single model approach facilitates streamlined model calibration and flexibility when performing future hindcast simulations.

Rainfall losses were computed using the Natural Resource Conservation Service (NRCS) curve number method. The curve numbers were generated using the National Land Cover Database's (NLCD) 2019 Land Cover raster and the October 2021 Soil Survey Geographic Database (SSURGO), from which the hydrologic soil group (HSG) was obtained. An abstraction ratio of 0.2 and a minimum infiltration rate of 0.001 inches/hour were used to determine rainfall losses. Land cover classifications can be seen in Figure 24 below and Table 2 in Section 2.4 provides the curve numbers for each land cover and soil type combination. It is a USACE requirement for FRM feasibility studies to use the Annual Maximum Series (AMS) rainfall dataset as opposed to the Partial Duration Series (PDS) dataset. The AMS rainfall dataset was used for the H&H modeling.

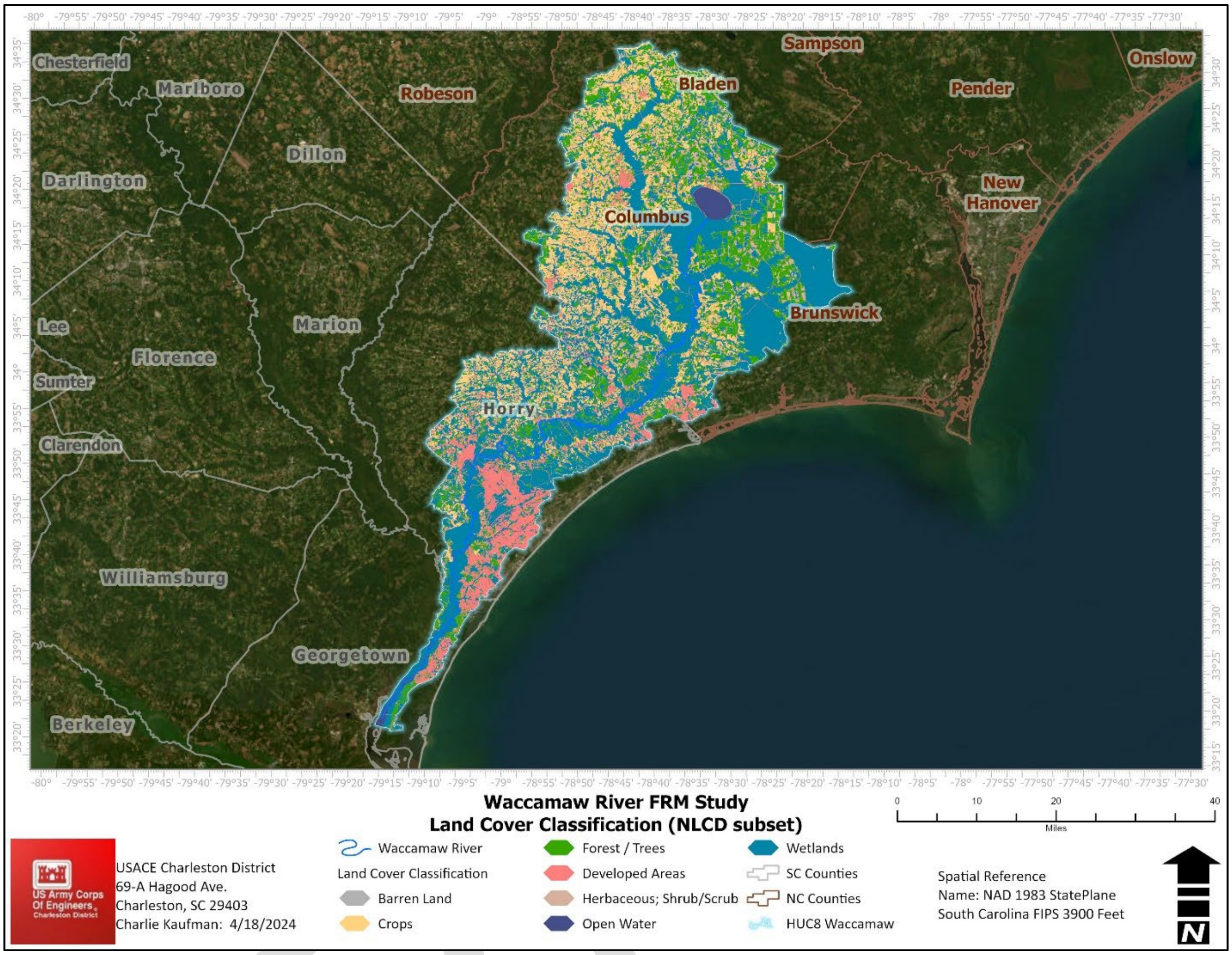


Figure 42. Land cover classifications from NLCD2019 for the Waccamaw River Basin

Sensitivity tests were performed on the storm duration, distribution, and areal reduction. The 100-year, NOAA Atlas 14, 24-hour and 96-hour storm depths and distributions in HEC- HMS were simulated to check the critical storm duration. The 96-hour distribution resulted in a larger peak flow for the area of interest (lower Pee Dee and Waccamaw). Based on this, the 96-hour duration was chosen along with the NOAA Atlas 14 Quartile 4, 90% rainfall distribution. For additional information, see Section 5.1 – Hydrology.

5.1.3 Rainfall Losses

For HEC-HMS models, the Soil Conservation Service (SCS) Curve Number methodology contained within Natural Resources Conservation Service (NRCS) Technical Report (TR)-55 was used to estimate for losses from a precipitation event occurring over the study areas (USDA, 1986). This method was chosen due to the desire for consistency with existing calibrated modeling, its accepted usage across both urban and rural hydrologic landscapes, and its ability to efficiently assess both historic and future watershed conditions.

The 2019 National Land Cover Database (NLCD) was utilized to generate land use classifications for subbasin areas. Geospatial analyses within ArcGIS software were used to determine weighted curve numbers based on the NLCD and the USDA Soil Survey Geographic Database (SSURGO) at the subbasin-level. Impervious surface area is also a parameter in the SCS Curve Number modeling. Impervious areas were estimated with the 2019 NLCD Urban Imperviousness dataset. Similar to the curve number methodology described above, a subbasin area-weighted impervious area percentage was determined for all subbasins. Initial abstraction values were automatically computed within HEC-HMS as 0.2 times the potential retention, which was calculated from the curve number (Figure 37).

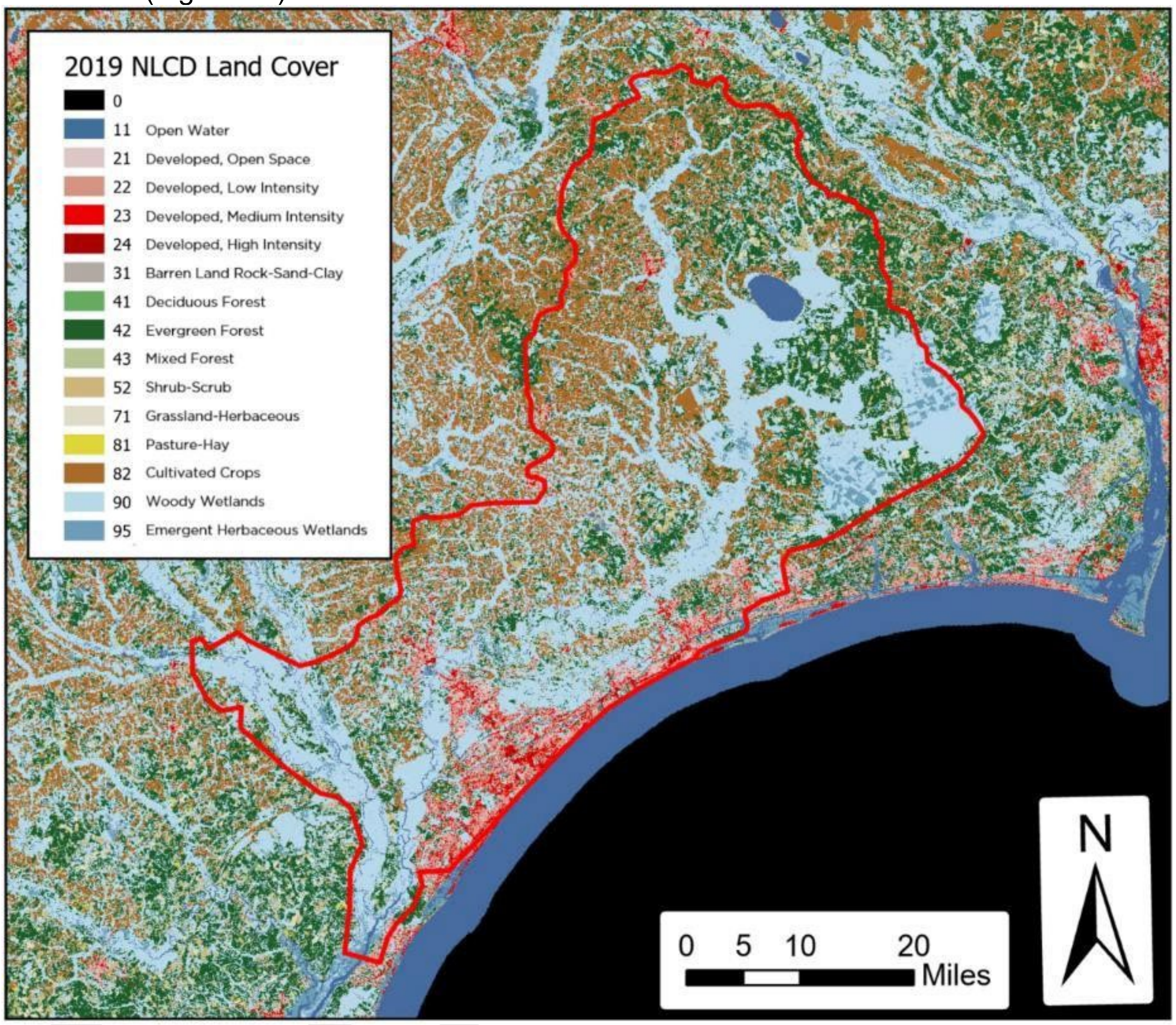


Figure 43. NLCD (2019) for the project area

The initial subbasin curve numbers that resulted from the geospatial analysis were adjusted during calibration to best fit observed data. Adjustments were also made in consideration of antecedent moisture conditions associated with the historic events.

Synthetic rainfall events were developed to assess watershed's response for the 50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.2% annual exceedance probabilities (AEPs), also known as the 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year storm events, respectively. The rainfall depths used to develop the rainfall hyetographs were calculated in HEC-HMS using the Volume 2 (Ohio River Basin and Surrounding States) NOAA Atlas 14 GIS grid atlas, which contains gridded datasets for each AEP. These gridded datasets account for the spatial variation in rainfall probability across each region, an example is shown in Figure 6. The precipitation values calculated from these grids within HEC-HMS is shown for each basin in Table 4.

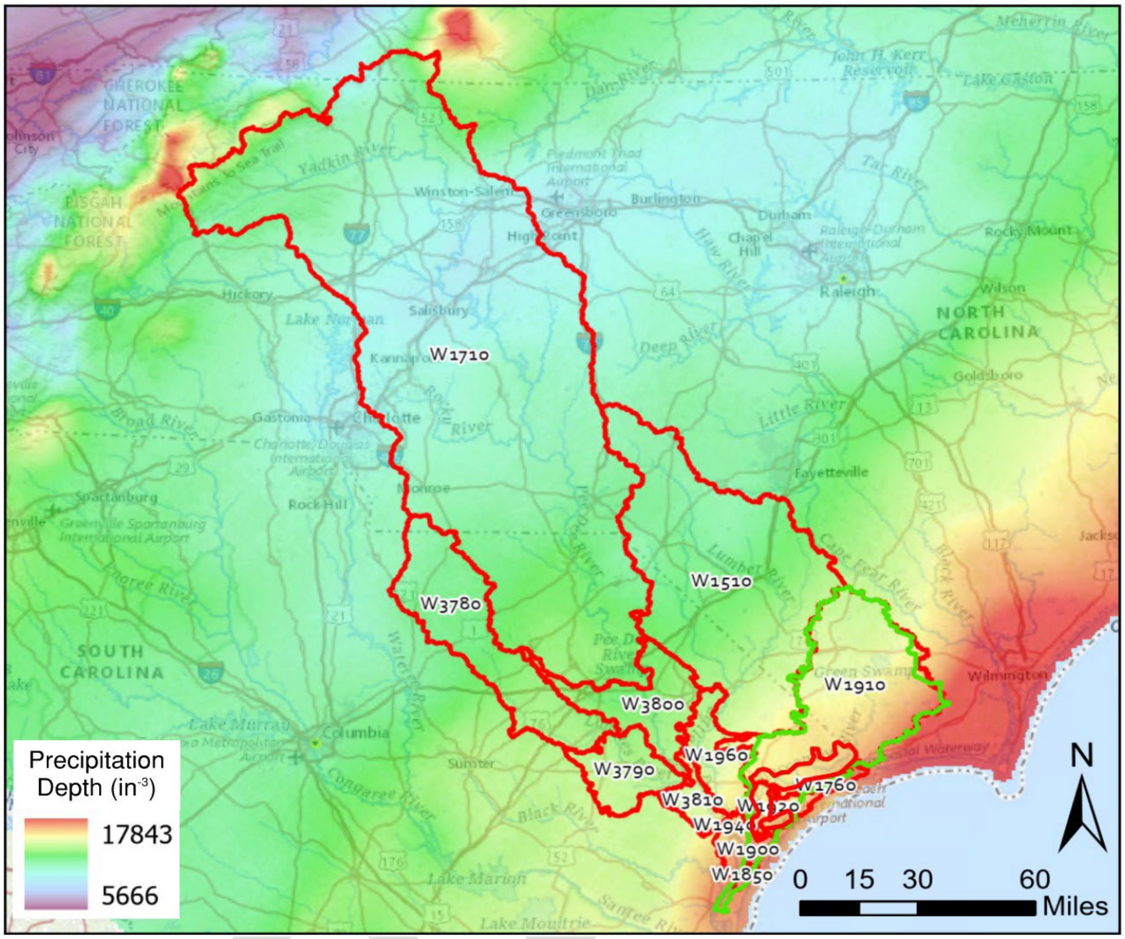


Figure 44. Example of NOAA Atlas 14 GIS Precipitation Frequency Estimate Grid, 1% AEP

Table 16. Rainfall depths for each synthetic rainfall event for sub watersheds in Waccamaw River Basin

HEC-HMS Basin	Rainfall Depth (in.)						
	2yr	5yr	10yr	25yr	50yr	100yr	500yr
W1510	4.22	5.62	6.9	8.33	9.47	10.68	13.82
W1710	4.04	5.31	6.47	7.72	8.71	9.73	12.35
W1760	4.91	6.59	8.13	9.91	11.35	12.89	17
W1850	5.06	6.78	8.35	10.13	11.55	13.06	17.01
W1900	5.14	6.89	8.49	10.29	11.73	13.26	17.26
W1910	4.6	6.18	7.65	9.38	10.81	12.36	16.6
W1920	4.84	6.49	8	9.71	11.09	12.56	16.43
W1940	4.84	6.49	8	9.72	11.11	12.59	16.51
W1960	4.5	6.03	7.44	9.06	10.38	11.8	15.59
W3780	4.08	5.38	6.61	8.06	9.25	10.56	14.2
W3790	4.26	5.67	7.01	8.6	9.92	11.37	15.39
W3800	4.14	5.53	6.82	8.34	9.59	10.94	14.67
W3810	4.54	6.07	7.48	9.12	10.45	11.87	15.69
Waccamaw 2D	4.86	6.52	7.76	9.5	10.92	12.43	16.65

A 96-hour storm duration with a NOAA Atlas 14 Quartile 4, 90% decile rainfall distribution was utilized to generate the rainfall hyetographs for each sub-basin in the HEC-HMS model for the synthetic event simulations.

Sensitivity tests were performed on the storm duration, distribution, and areal reduction (Results in Sensitivity Analysis Section). The 100-year, NOAA Atlas 14, 24-hour and 96-hour storm depths and distributions in HEC- HMS were simulated to check the critical storm duration. The 96-hour distribution resulted in a larger peak flow for the area of interest (lower Pee Dee and Waccamaw). Based on the results, the 96-hour duration was chosen along with the NOAA Atlas 14 Quartile 4, 90% rainfall distribution.

5.1.3.1 Aerial Reduction Factor

A design storm was used in the Waccamaw River mainstem basin HEC-HMS model to create rainfall events that captured the high variability in subbasin response throughout the large study area. Its intent was to simulate a more objective and homogenous rainfall pattern that can be used for engineering purposes. NOAA Atlas 14 Annual Maximum Series point precipitation values were used to develop design storms for the following annual exceedance probabilities (AEP): 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, 0.005, and 0.002.

Sensitivity tests were performed on the storm duration, distribution, and areal reduction. The 100-year, NOAA Atlas 14, 24-hour and 96-hour storm depths and distributions in HEC- HMS were simulated to check the critical storm duration. The 96-hour distribution resulted in a larger peak flow for the area of interest (lower Pee Dee and Waccamaw). Based on this, the 96-hour duration was chosen along with the NOAA Atlas 14 Quartile 4, 90% rainfall distribution.

Due to the large size of the Waccamaw River basin, Aerial Reduction Factors (ARF) were applied to frequency point precipitation values to represent the reduction in point rainfall depths moving away from the center of the storm. Figure 80 shows a comparison of the runoff hydrographs for the sensitivity scenarios at the Highway 701 bridge on the Pee Dee River. The dark blue (top) line on the graph is from the FEMA model, which used a single rainfall depth with the 24hr, Type III distribution. That produces much higher peak flows than the runs for this study because of the difference in rainfall depths. The FEMA study used 11.2 inches for all subbasins, and the basin weighted average is closer to 8.62 inches based on NOAA Atlas 14 depth values for a 24-hour event. This is due to the upstream basins of the Pee Dee being well inland from the coast and having much lower 100-year 24-hour rainfall depths. The orange (without areal reduction) and gray (with maximum TP-40/49 areal reduction) lines show the 24- hour results using the basin-averaged NOAA Atlas 14 rainfall depths and a Quartile 4, 90% rainfall distribution. The yellow (without areal reduction) and light blue (with maximum TP-40/49 areal reduction) lines show the 96-hour results using the basin-averaged NOAA Atlas 14 rainfall depths and a Quartile 4, 90% rainfall distribution. The 100-year peak flow based on the AECOM study for FEMA was 129,000 cfs (from “USGS Bulletin 17b” stream gage analysis), which falls between the two 96-hour peaks. The TP-40/49 depth-area- duration (DAD) chart was then used to adjust the HEC-HMS model results to as close to the 100-year FEMA study value of 129,000 cfs as possible.

A storm size of 25 square miles was utilized for the areal reduction within HMS. The ARF associated with the 25 sq mi storm area results in a value that approximately matches the Bulletin 17B values that were calculated for the FEMA model. Because we were changing the duration and distribution of the rainfall for this study, we felt it was necessary to adjust the results of peak flow to approximately match the FEMA flow at that location. Keep in mind the ARF has an inverse relationship to the average precipitation intensity across the watershed. The larger the ARF storm size, the smaller the average precipitation intensity that gets applied across the entire watershed during the simulation. In this case, 25 sq mi is a very small storm size, so it doesn't reduce the average precipitation intensity by very much. As previously stated, the difference for with and without the areal reduction factor for the 100-year rainfall for the Waccamaw basin was approximately 0.22 inches, or 1.7%. We felt this was a conservative ARF value to adjust the point precipitation values for this watershed. Additional study would be necessary to determine a more accurate storm size for use in updating the ARF. The result of such a study would show a larger storm size, which would decrease the average precipitation intensity and thereby reduce the peak flow rates in the model. We don't feel it's necessary because we are quasi-calibrating the model to the Bulletin 17B data from the FEMA study as indicated.

The areal reduction factor used it to quasi-calibrate the Pee Dee HMS model to FEMA's 100-year effective peak flows since our storm was adjusted to a 96-hour NOAA Atlas 14 temporal distribution (instead of the 24-hour SCS Type 3 distribution). It was necessary to match FEMA's effective peak flows for consistency with existing regulatory models. In this case, the TP40/49 areal reduction factor was an accessible calibration parameter within HEC-HMS that we could use to adjust the new runoff hydrographs to approximately match FEMA's effective peak flows at our boundary conditions (keeping in mind we weren't scoped to do a full update of the Pee Dee model). This reduction was in place for the Waccamaw basin to be consistent with the other basins (the difference for with and without the areal reduction factor for the Waccamaw basin was approximately 0.22 inches, or 1.7%). Figure 82 in the report shows the modeled differences for with and without the maximum areal reduction factors. We adjusted the factor until we got close to the FEMA flow value of 129,000 cfs at the Hwy 701 bridge.

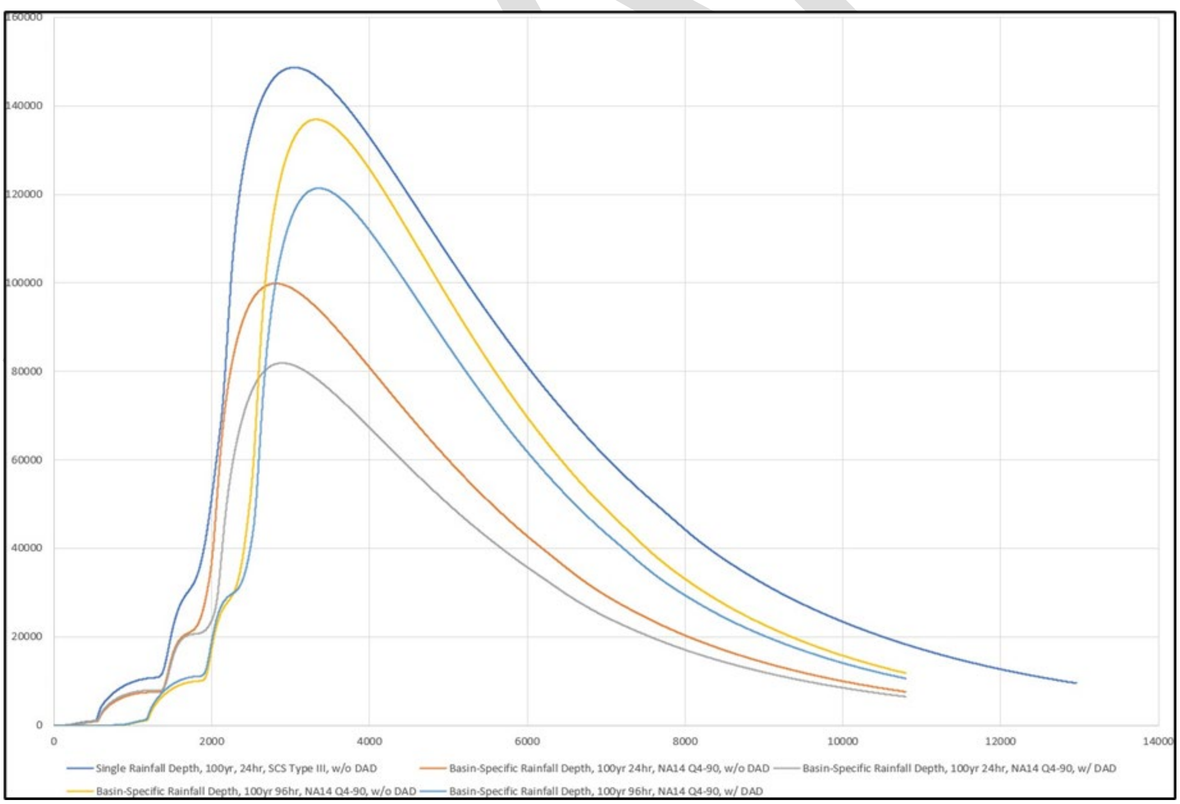


Figure 45. Comparison of 1% AEP flow estimates with varying hydrologic assumptions

Figure 81 shows a comparison of the FEMA/AECOM "USGS Bulletin 17B" stream gage analysis results and this study's sensitivity checks on TP-40/49 depth-area-duration reduction for the 96-

hour Quartile 4, 90% distribution (for the 10-, 25-, 50-, 100-, and 500-year events). The 25 square mile storm size was selected because it approximately produced the 129,000 cfs value from the FEMA/AECOM study for the 100-year event.

Note that the HEC-HMS model underestimates flow for the more frequent events compared to FEMA's Bulletin 17b results, while it overestimates flow for the less frequent events.

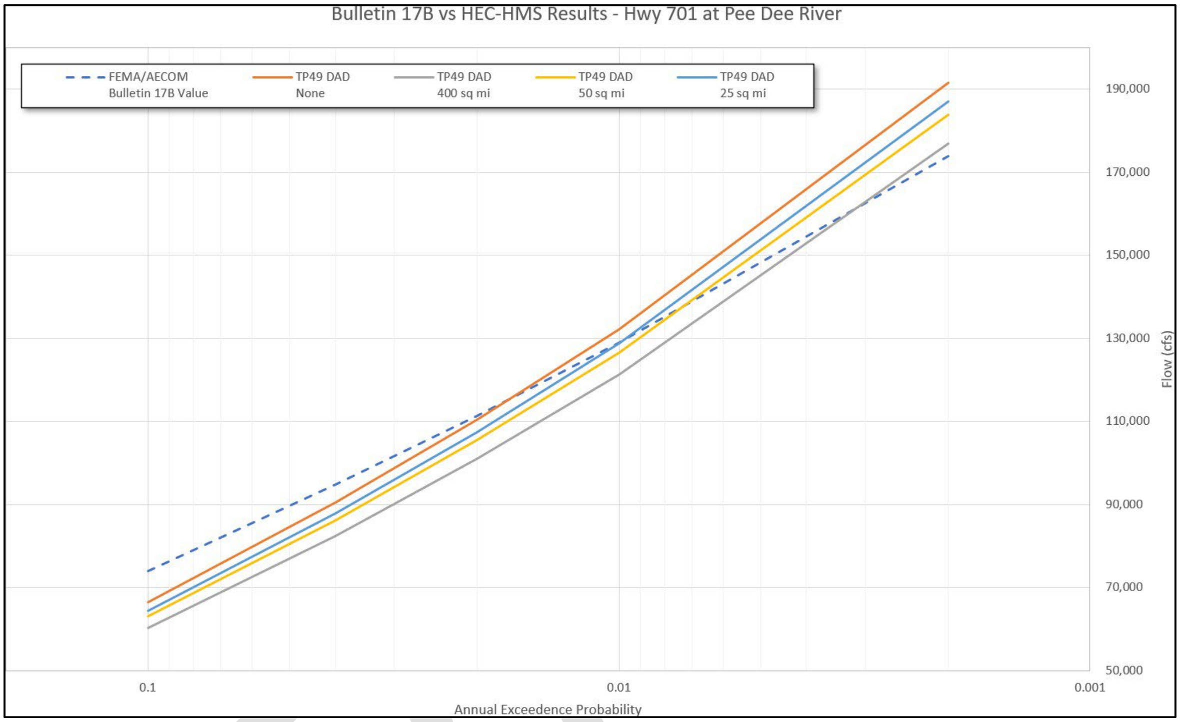


Figure 46. Comparison of Bulletin 17B Stream Gage Analysis Results vs. HEC-HMS Results for Various Areal Reduction Storm Sizes (25 to 400 sq mil)

5.2 Hydraulics

5.2.1 Model Overview

As discussed in the background hydrology section, a tiered modeling approach was used to create the 2D mesh and roughness value refinements within the HEC-RAS existing conditions geometry file. This approach reduces the model run times and provides the necessary mesh and roughness detail within the floodplains and the area of interest. The base mesh comprised the upland areas, or overland flow areas, which covered most of the modeled area. The floodplains were defined with calibration regions and breaklines, and the channels were further refined with additional calibration regions and breaklines. The base mesh and floodplain areas consisted of hexagonal cells, and the channel mesh consisted of rectangular cells where breaklines were implemented. The range of cell spacing used for these three tiers is provided in Table 16.

Table 17. Refinement Region Cell Spacing Table

Region	Cell Spacing (ft)	Notes
Base/Overland Flow Areas	1000	Any area outside of the refinement regions
Floodplain Flow Areas	500-1000	Flow areas within the floodplain, including breaklines where necessary for more detail
Channel Flow Areas	100-250	Top of bank width of each channel within the area of interest, including breaklines where necessary for more detail

Breaklines were utilized to represent hydraulic restrictions in the floodplains, such as roadway embankments or dams. Generally, the cell spacing for the breaklines was set to the same spacing as the adjacent mesh. Sometimes, a finer mesh sizing was utilized for breaklines where more detail and definition were desired. Figure 40 shows an example of the mesh layout for a location within the area of interest. The computation interval for the hydraulic modeling was 2 minutes.

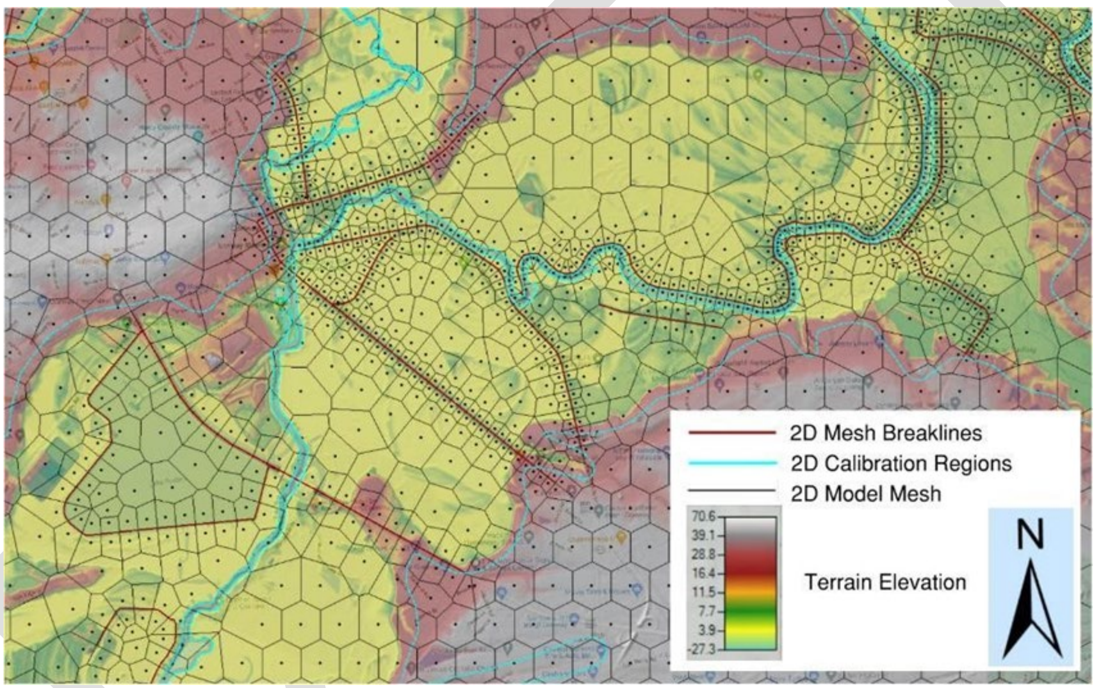


Figure 47. HEC-RAS 2D Mesh Layout Example

Once the mesh was generated, bridges were added to the model as “2D connections”. The cells surrounding the 2D connection were aligned perpendicular to the bridge, which helps create a more uniform flow through the bridge opening and improves model computation stability. An example of a bridge incorporated into the 2D mesh is shown in Figure 39. Sensitivity testing was performed to understand the impact of bridges on water surface elevations and flows. The bridges were ultimately removed from the model because they only created localized effects on water surface elevation and velocity. The sensitivity of the model results due to bridges is discussed more in the Model Sensitivity and Calibration section of this report.

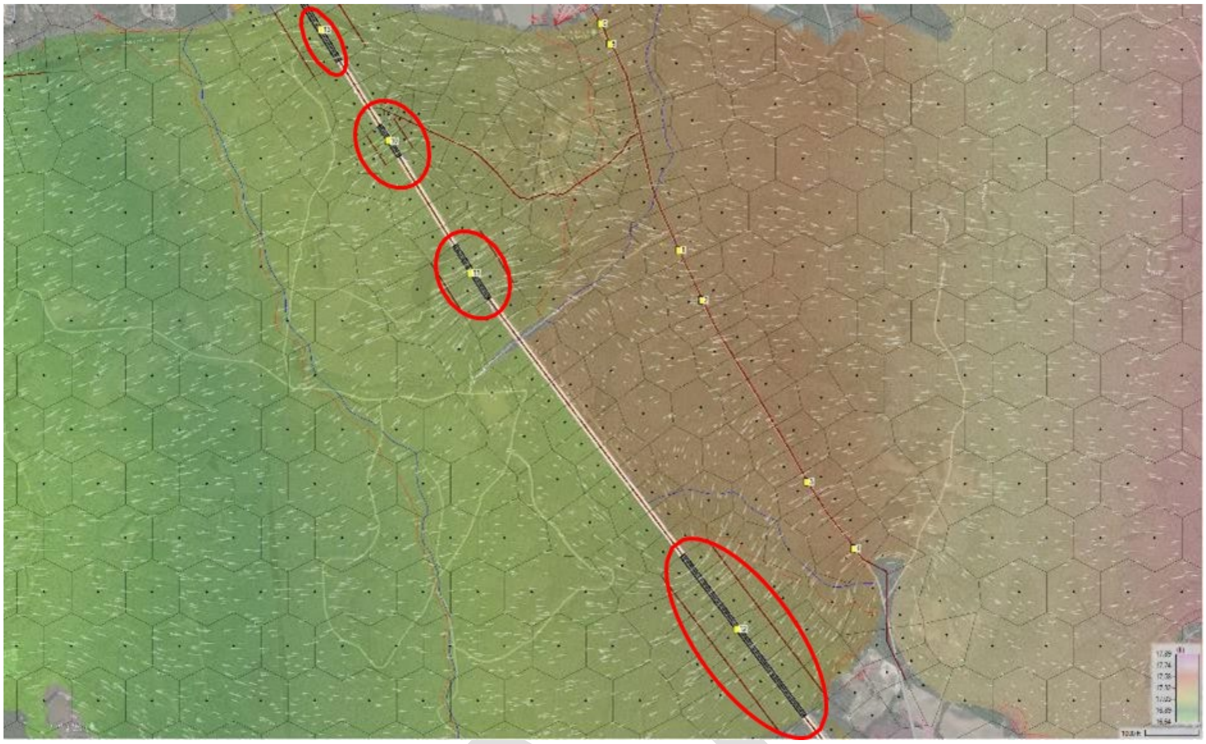


Figure 48. HEC-RAS 2D Bridge Layout Example, Highway 22 Crossing

Finally, the inflow and outflow boundaries were added to the mesh's exterior perimeter, as shown in Figure 40, with the major inflow and outflow boundaries labeled. The major outflow conditions include ICWW Outflow and Pee Dee River Outflow were set up as stage hydrograph.

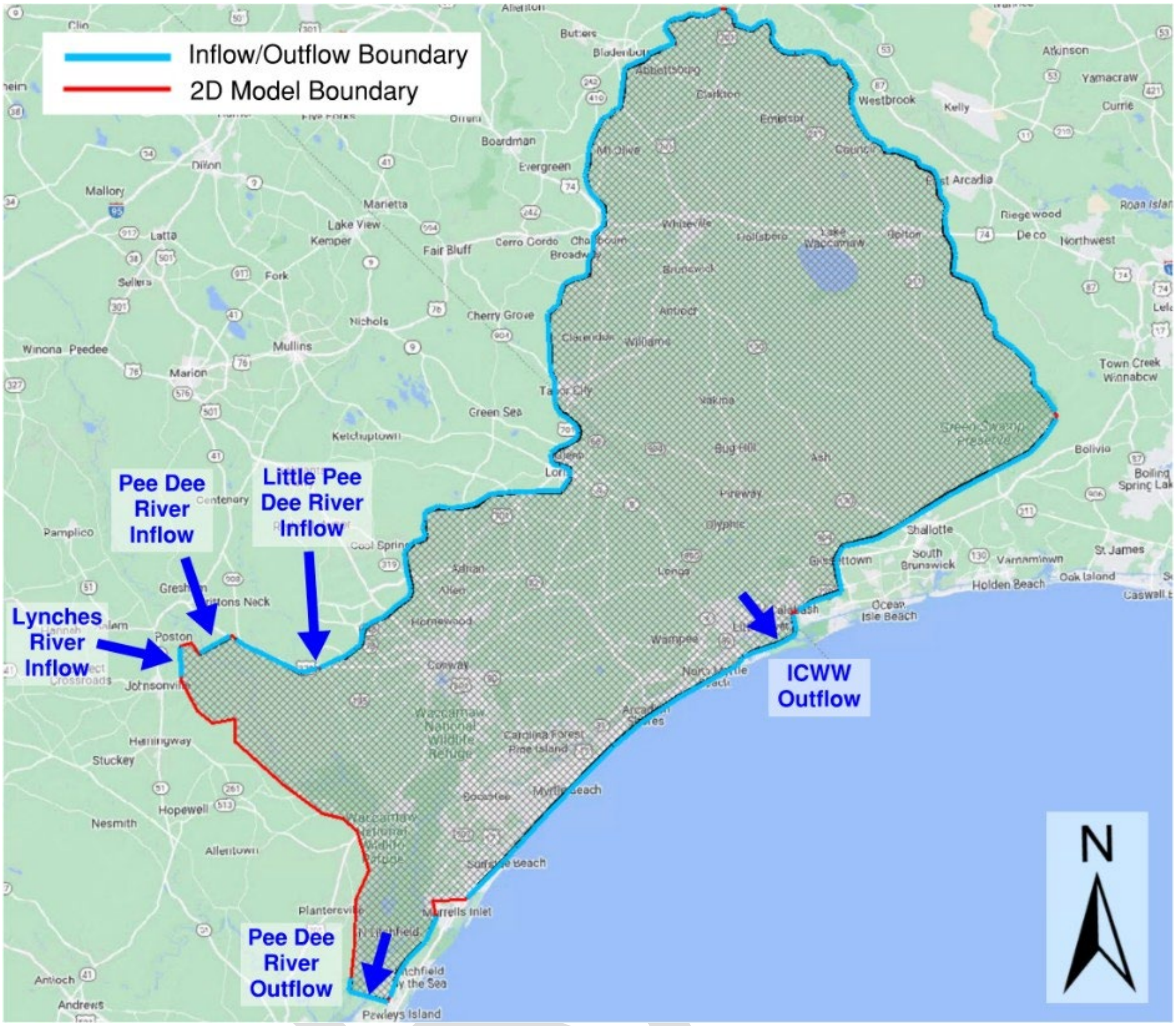


Figure 49. HEC-RAS 2D Model Boundary Condition Lines

5.3 Calibration and Validation

Three rainfall events were chosen for the Waccamaw River Mainstem basin rain on grid model calibration and validation. One event was used for calibration and two for validation. One calibration scenario included Hurricane Florence (2018) and validation for Hurricane Matthew (2016) and the October flood of 2015 caused by Hurricane Joaquin (2015). The Florence run was the true calibration run, where parameters such as roughness and terrain were changed in the model to achieve the results discussed in the Model Sensitivity and Calibration Results section of the report. Matthew and Joaquin were used as validation events to check the accuracy of the previously calibrated parameters using different events. Selection of calibration events were primarily based on availability of gridded precipitation, ground-based precipitation gages, rainfall footprint, and completeness of streamflow gage records in the basin.

Model calibration was performed to validate the water surface elevation and flow output results. Three events were used to calibrate and validate the model based on conversations with USACE SAC. These events were Hurricane Florence, which calibrated the model, while Hurricane Matthew and Hurricane Joaquin rainfall events validated the model. It was a unique rainfall event, that captured a different flooding event with no second peak as observed in Hurricane Florence and Matthew. It was paramount to validate the event primarily since it was a “firehouse event”. One-hour Multi-Radar Multi-Sensor (MRMS) rasterized rainfall data was obtained from Iowa State

University's Iowa Environmental Mesonet website. The MRMS datasets were imported into HEC-RAS to reflect the spatial and temporal variation in rainfall across the Waccamaw River watershed. No comparisons to ground rainfall gages were performed for this study. However, MRMS data incorporates rainfall gages to correct the radar data, so it is considered "ground corrected".

Gridded rainfall datasets provide much better calibration results than point rainfall data from precipitation gauges. This is due to the large spatial and temporal variation in rainfall across large basins like the Waccamaw River watershed. Figure 37 through Figure 38 show the accumulated precipitation for each of the three calibration events and a point rainfall accumulation graph associated with the approximate maximum rainfall located within the Waccamaw River watershed for each event.

The USGS gage locations are shown in Figure 41 and the corresponding names and gage locations are shown in Table 17. In addition to the rainfall for the calibration events, USGS stream gages were used to set the inflow and outflow boundaries of the Pee Dee River. This data was pulled directly from the USGS website, and the boundaries were input as water surface elevation hydrographs. The green line indicates the modeling extents of the project, to capture the pertinent USGS gage data.

Calibration with observed data was based on selection of widespread rainfall events as described above. Overall, comprehensive event coverage for the entire Waccamaw River basin was limited due to its large area. For Hurricanes Matthew and Florence, there were inconsistences in rainfall amounts across the different geographic regions in the basin. Outside of these major tropical events, the varying intensity associated with frontal-based rainfall events meant that out-of-bank flooding for large portions of the Waccamaw River mainstem was difficult to capture in a single, historical scenario. There were some High water mark (HWM) data in the 2019 FEMA study documentation that we could use to compare the model results to, however it's unclear what vertical datum was used in the survey, therefore it was used as a spot check in lieu of calibration effort. We spot checked some of the locations around Conway, and the results vary with some being higher and some being lower than the modeled water surface elevations. The bulk of the water surface elevation show the model being higher, on the order of a quarter of a foot.

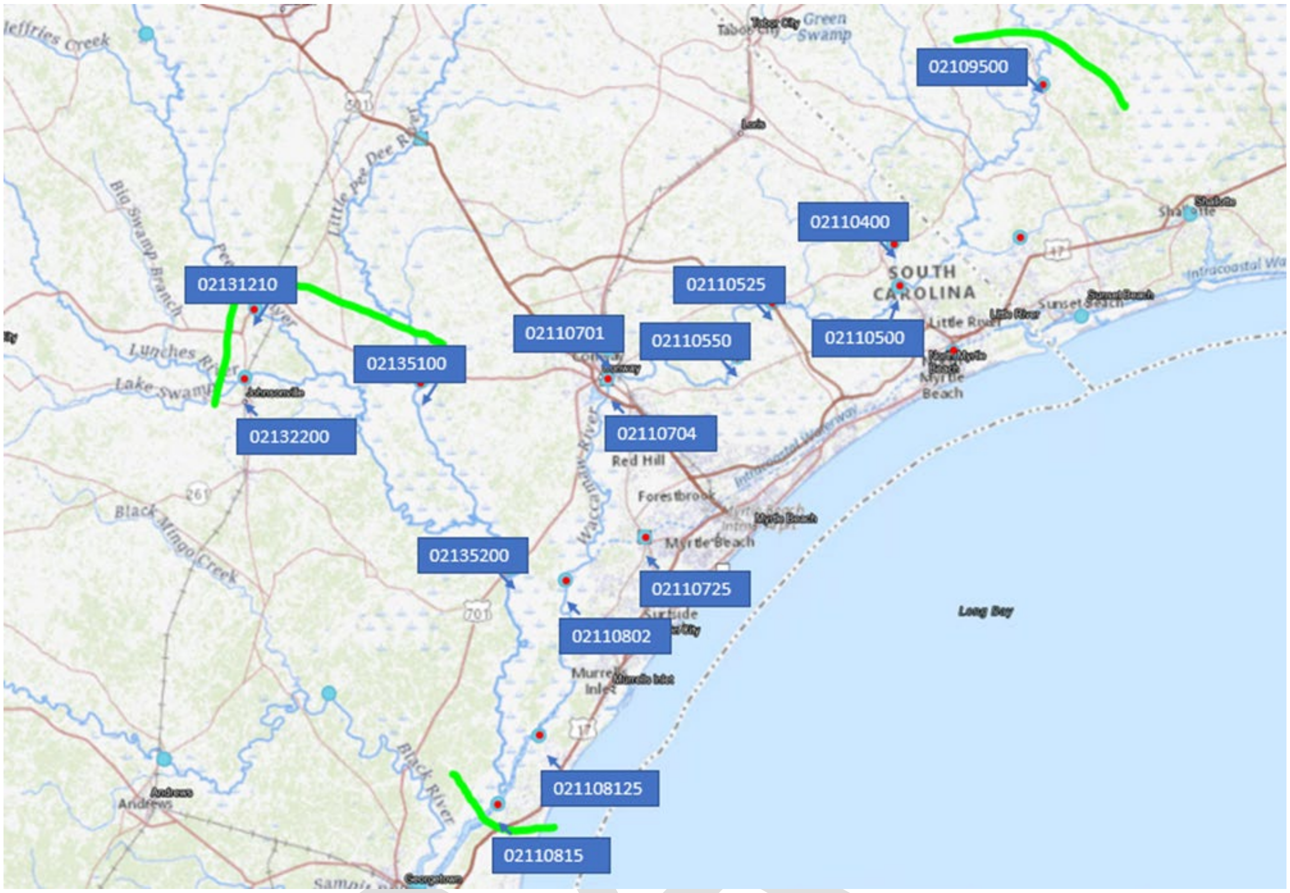


Figure 50. Streamflow gages in Waccamaw River Watershed

Table 18. Streamflow Gages Used in Calibration Efforts

Station Number	Station Name
2110815	Waccamaw Near HagleyLndg
21108125	Waccamaw at Pawleys
2135200	PeeDee at Hwy701
2110802	Waccamaw at Bucksport
2135100	Little Pee Dee at Conway
2131210	PeeDee at Hwy378
2132200	Lynches at Johnsonville
2110725	AIW at Hwy544
2110704	Waccamaw at Conway Marina
2110701	Crabtree Swamp at Conway
2110550	Waccamaw bv Conway
2110500	Waccamaw near Longs
2110400	Buck Creek near Longs
2109500	Waccamaw at Freeland
NOAA only	Caw Caw Swamp

5.3.1 Hurricane Florence Calibration

Model calibration was performed to validate the water surface elevation and flow output results for Hurricane Florence. Figure 51 shows total rainfall accumulation across the project area and Figure 52shows the approximate maximum point rainfall accumulation timeseries. It’s located at the approximate maximum precipitation depth for the event. See the point on the map below for the approximate location.

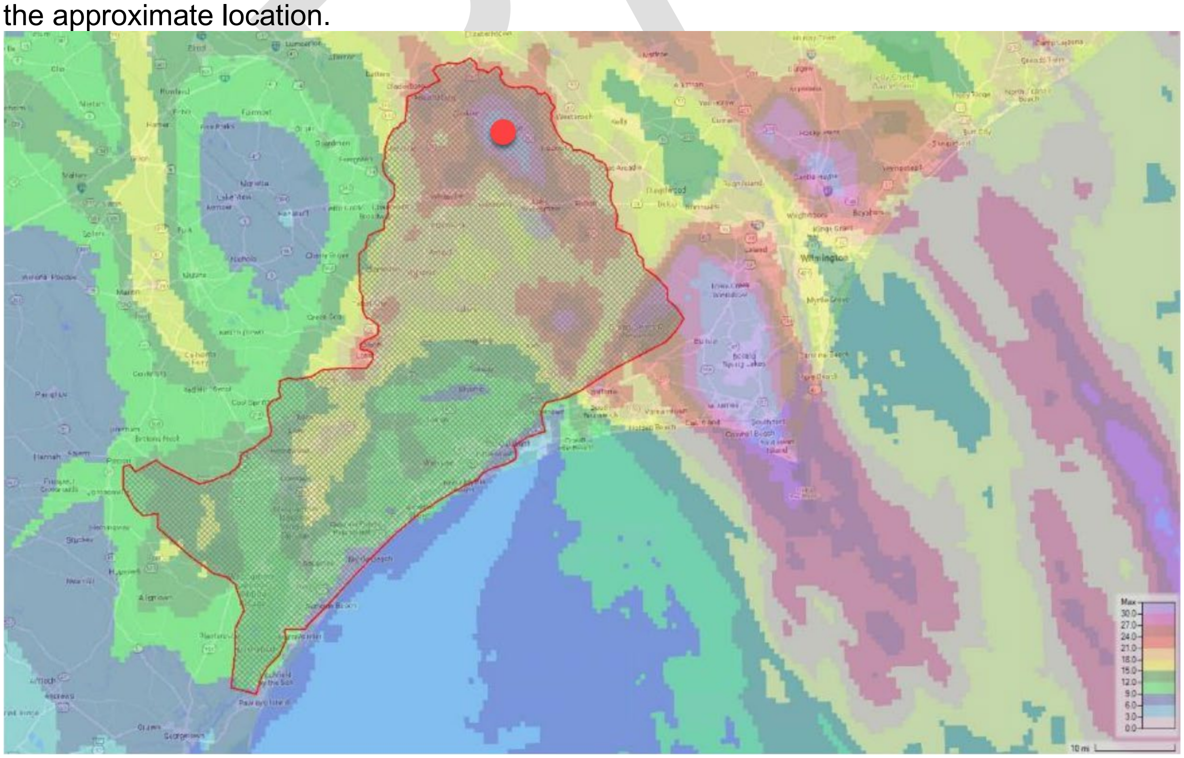


Figure 51. Total Rainfall Accumulation Map for Hurricane Florence (9/13-9/20/2018)

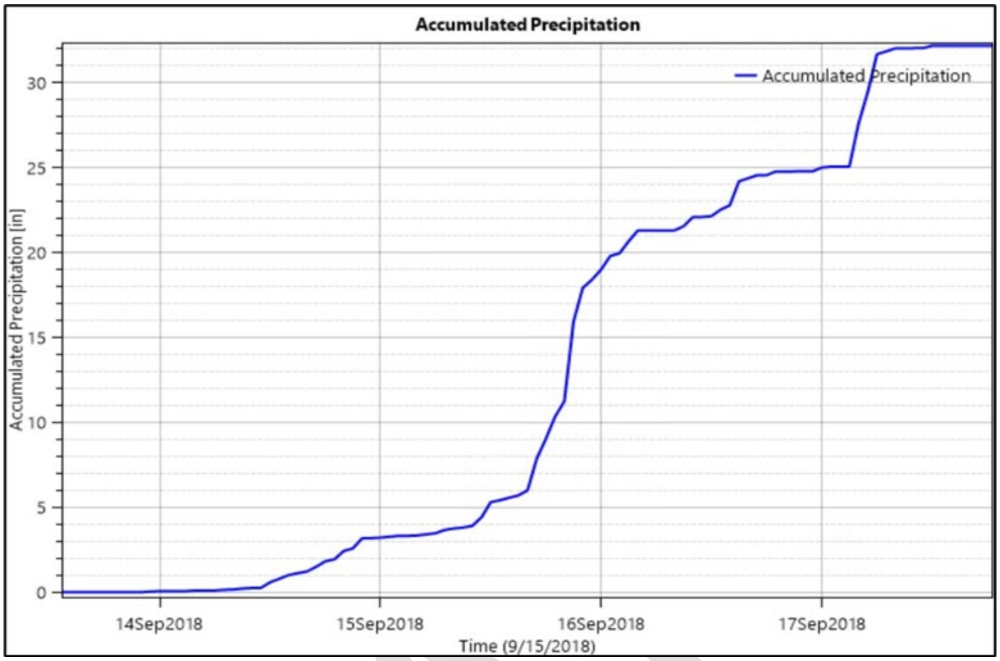


Figure 52. Approximate Maximum Accumulated Precipitation Point for Hurricane Florence. The location of the hyetograph is indicated in previous figure.

Results for the Hurricane Florence calibration event at select USGS gages are shown in Figure 53 through Figure 59.

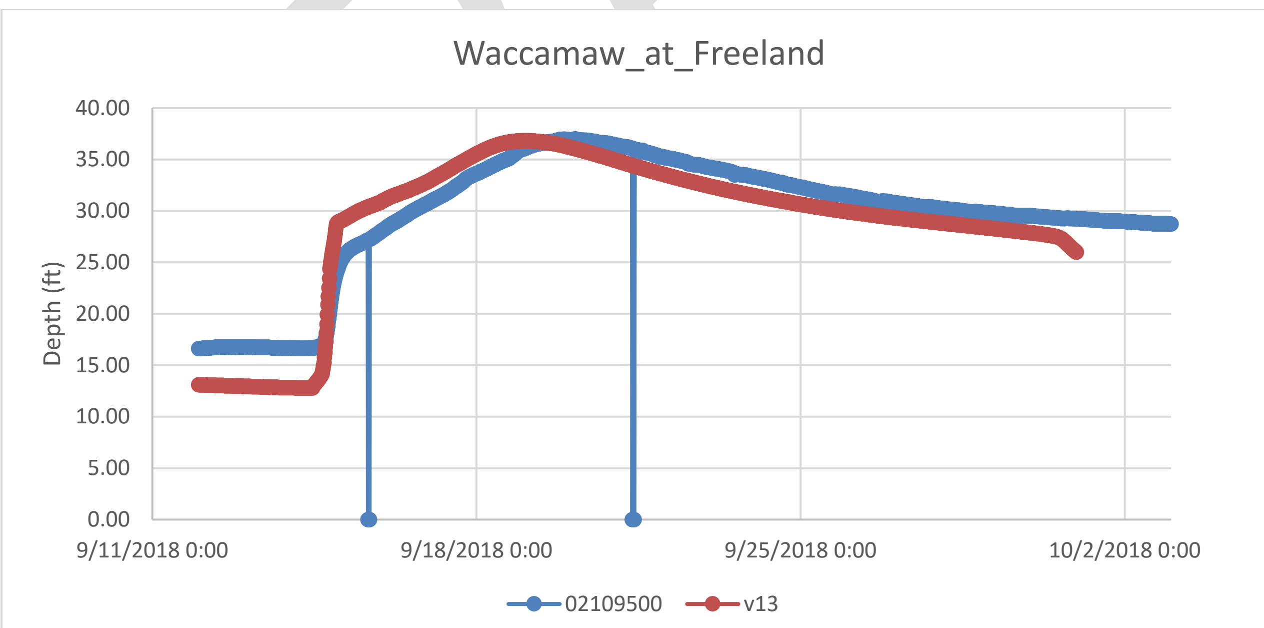


Figure 53. Calibration Hurricane Florence 02109500

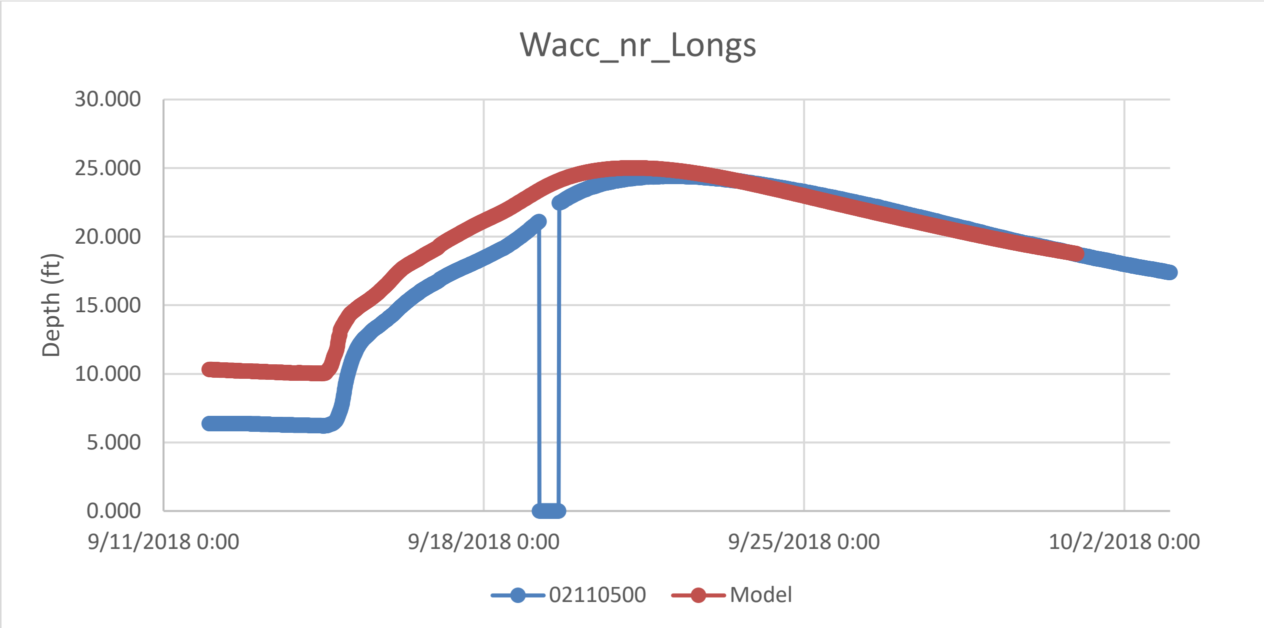


Figure 54. Calibration Hurricane Florence 021010500

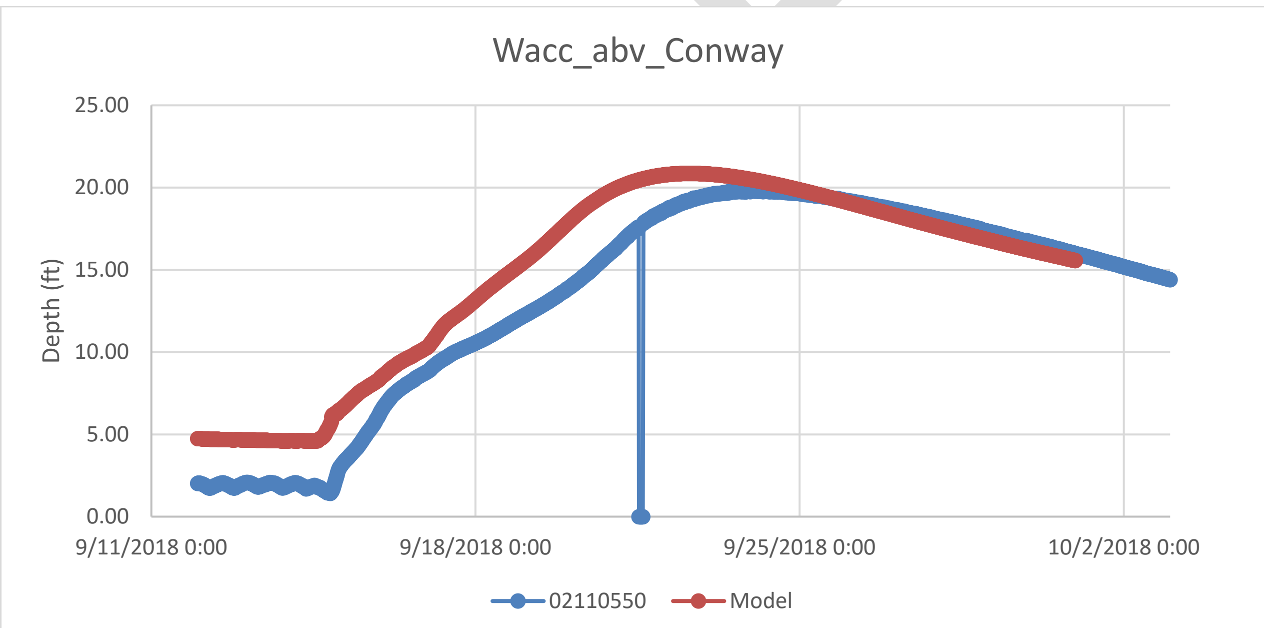


Figure 55. Calibration Hurricane Florence 02110550

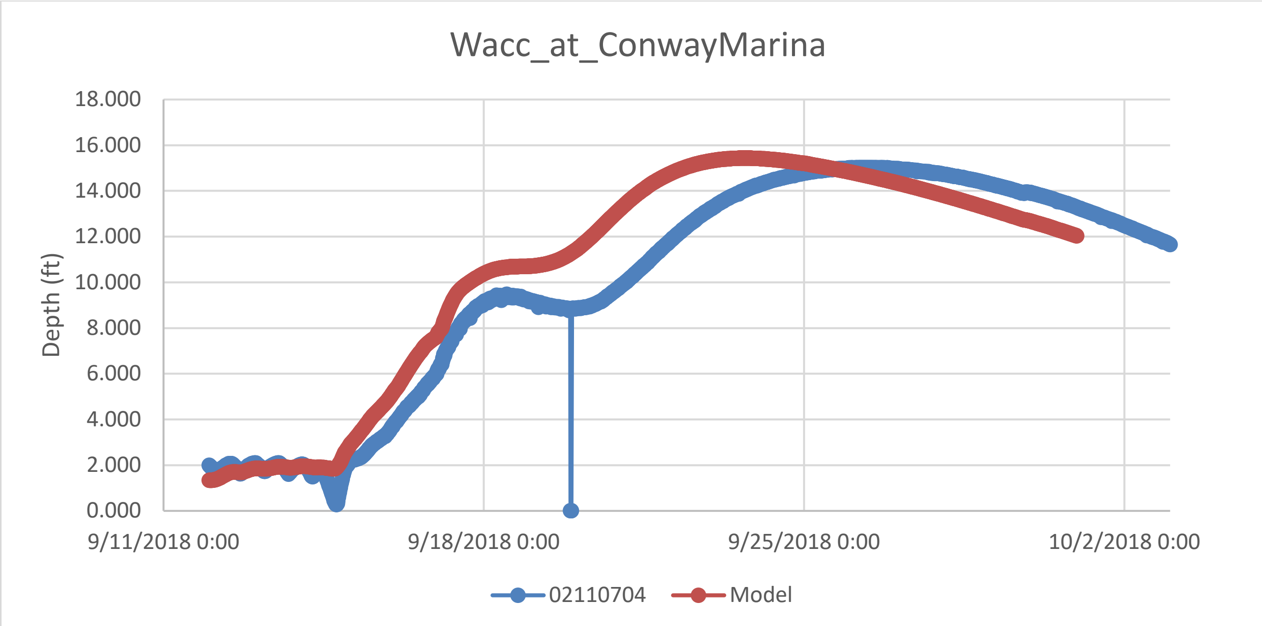


Figure 56. Calibration Hurricane Florence 02110704

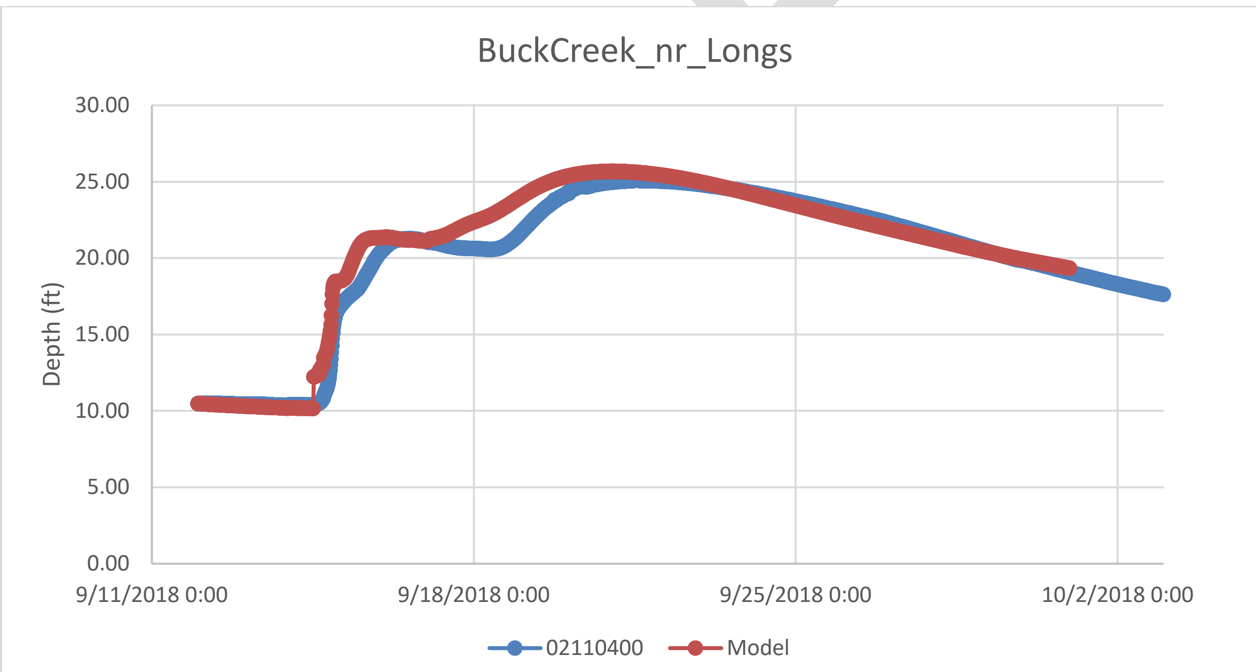


Figure 57. Calibration Hurricane Florence 02110400

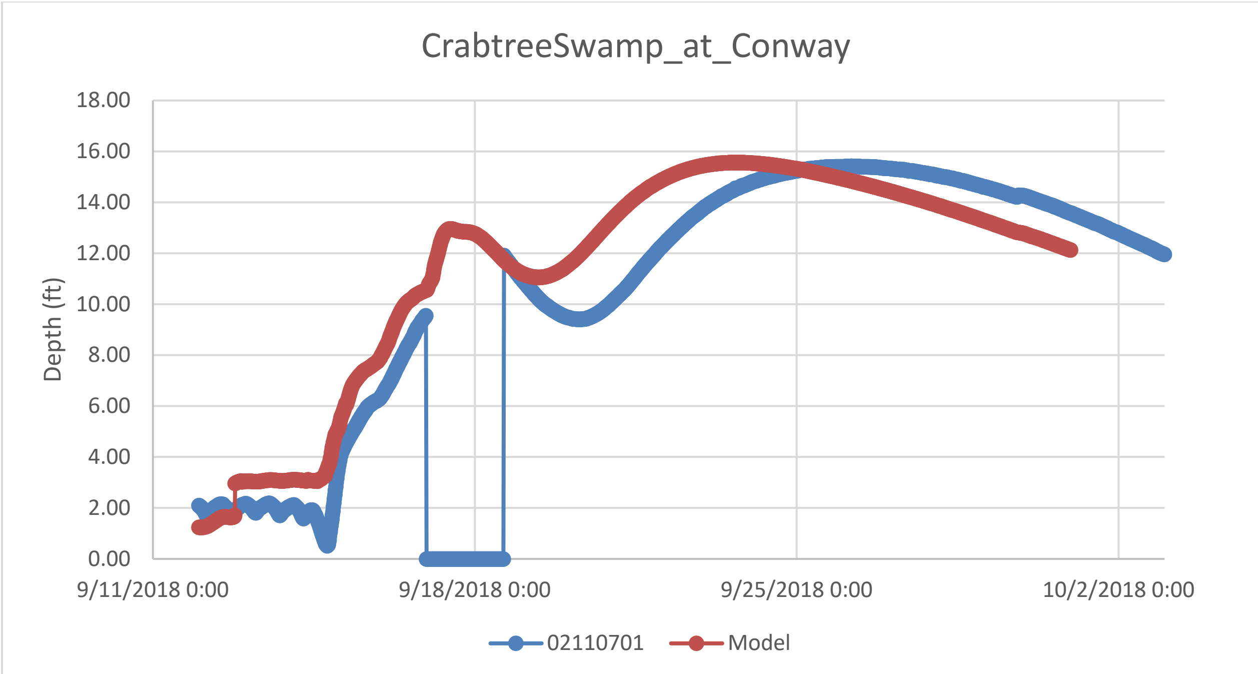


Figure 58. Calibration Hurricane Florence 02110701

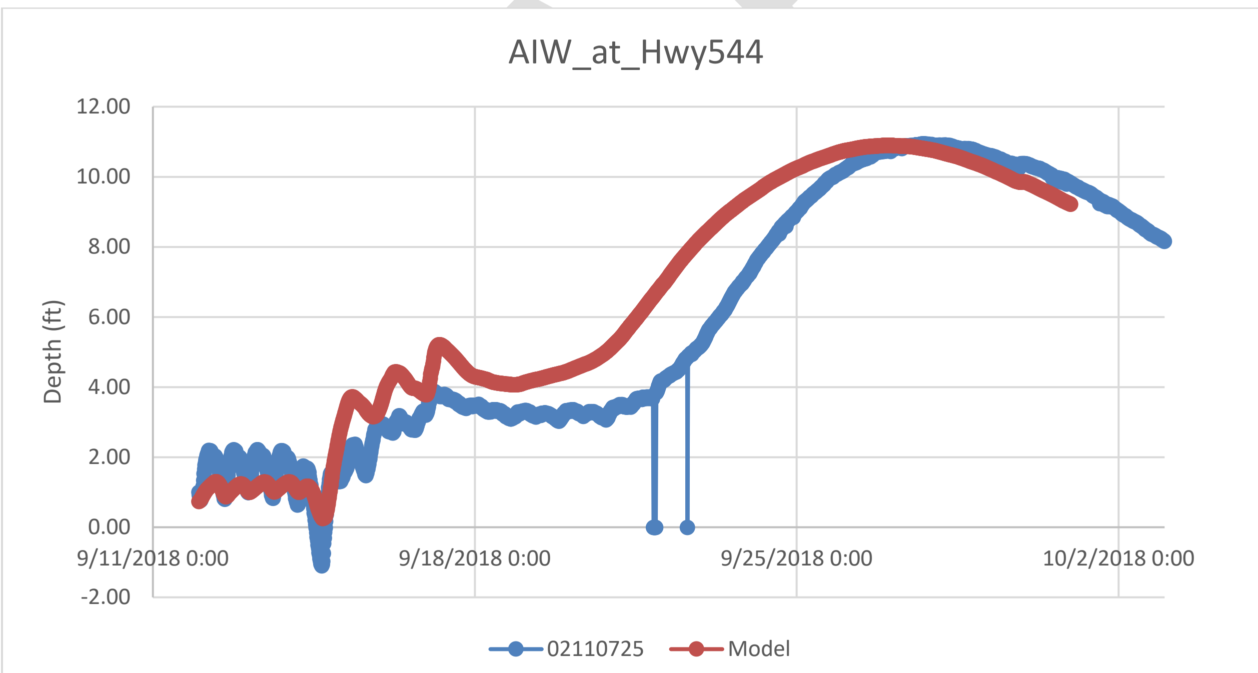


Figure 59. Calibration Hurricane Florence 02110725

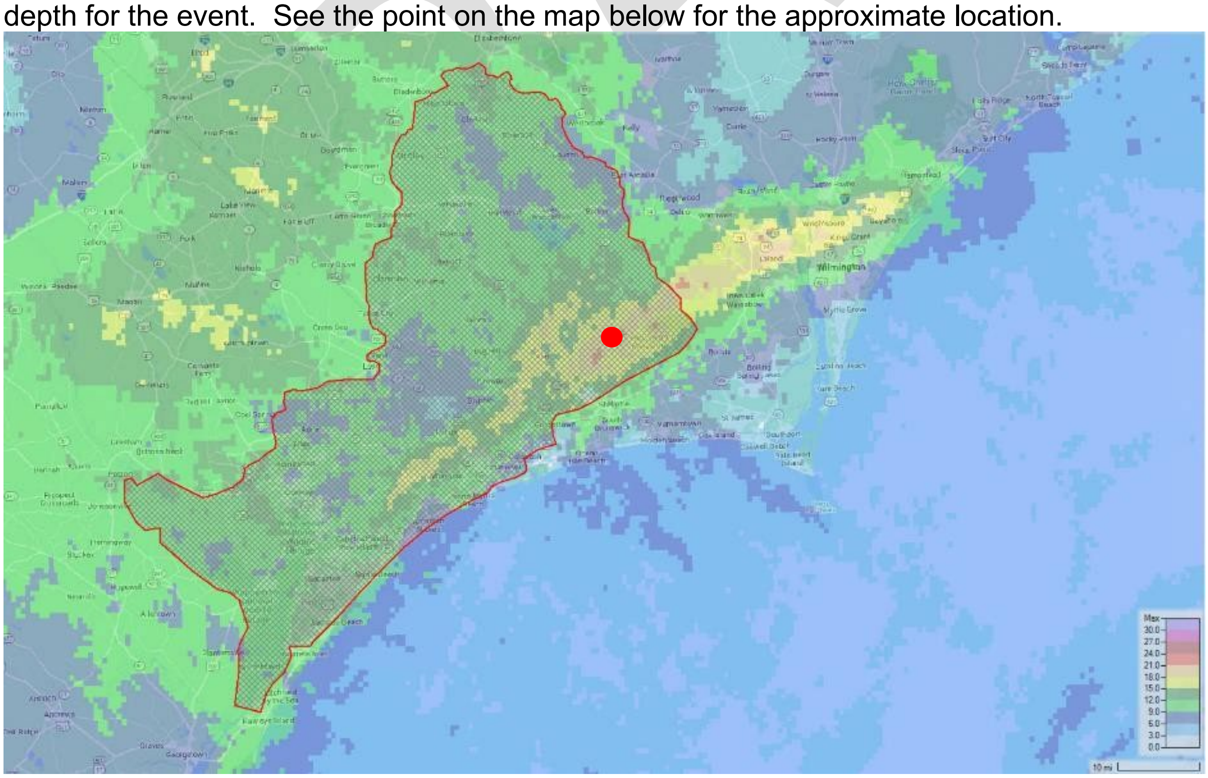
Table 18 shows results from the calibrated Hurricane Florence model run. Additional discussion of sensitivity analysis and parameter adjustment can be found in Section 5.3.4.

Table 19. Summarized Results of Hurricane Florence Calibration

Gage Location	Gage ID	Observed Flow (cfs)	Computed Flow (cfs)	Std. Dev.	Variance (%)
Waccamaw_at_Freeland	2109500	37.01	36.835	0.12	0.47%
Wacc_nr_Longs	2110500	24.41	25.012	0.43	2.47%
BuckCreek_nr_Longs	2110400	25.25	25.675	0.3	1.68%
Wacc_abv_Conway	2110550	19.82	20.858	0.73	5.24%
CrabtreeSwamp_at_Conway	2110701	15.42	15.561	0.1	0.91%
Wacc_at_ConwayMarina	2110704	15.02	15.433	0.29	2.75%
AIW_at_Hwy544	2110725	10.95	10.901	0.03	0.45%
Wacc_at_Buksport	2110802	11.319	10.875	0.31	3.92%
Wacc_at_Pawleys	21108125	6.85	6.786	0.05	0.93%
PeeDee_at_Hwy701	02135200 **	129000	125655	2364.82	2.59%

5.3.2 Hurricane Matthew Validation

Model validation was performed to validate the water surface elevation and flow output results for Hurricane Matthew. Figure 60 shows total rainfall accumulation across the project area and Figure 61 shows the approximate maximum point rainfall accumulation timeseries. Similarly to Hurricane Florence the hyetograph data is pulled from location at the approximate maximum precipitation depth for the event. See the point on the map below for the approximate location.



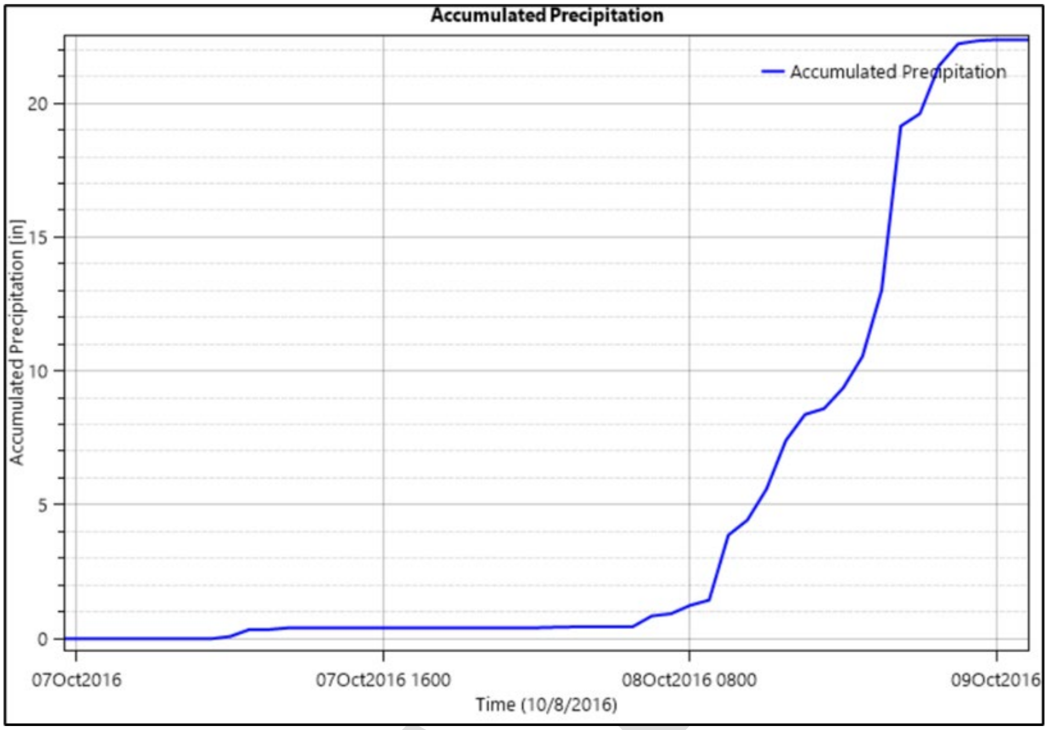


Figure 61. Approximate Maximum Accumulated Precipitation Point for Hurricane Matthew

Results for the Hurricane Matthew calibration event at select USGS gages are shown in Figure 62 through Figure 65.

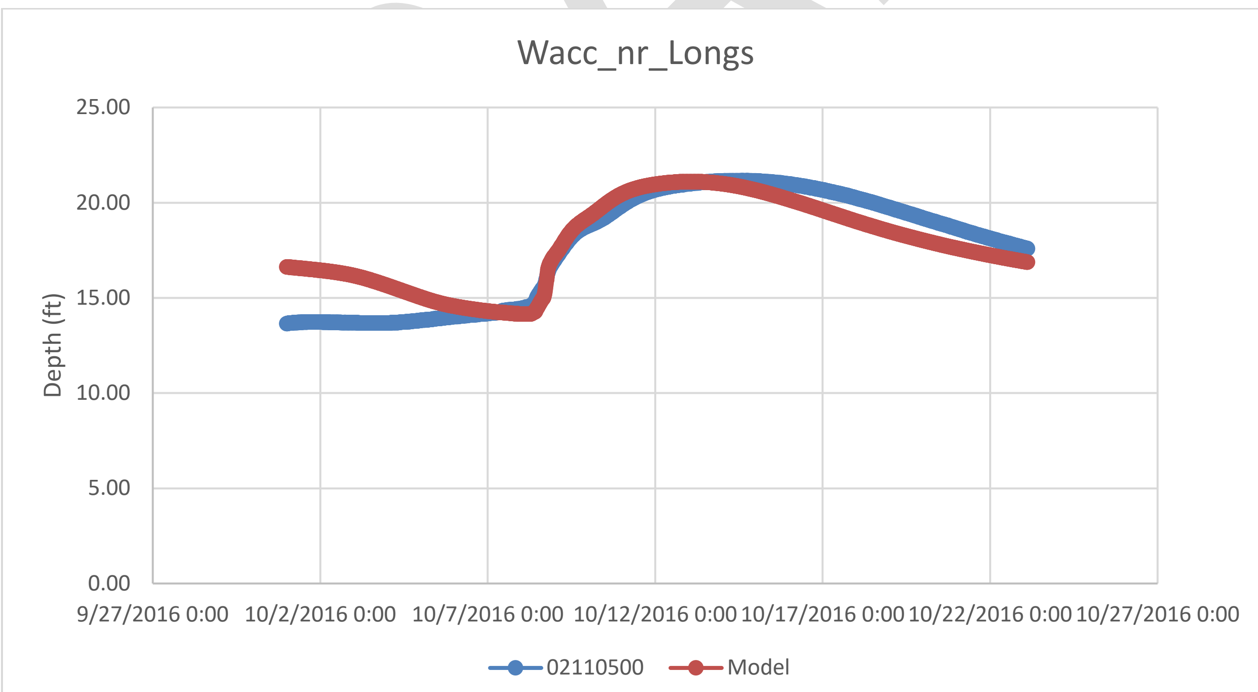


Figure 62. Validation for Hurricane Matthew Gage 02110500

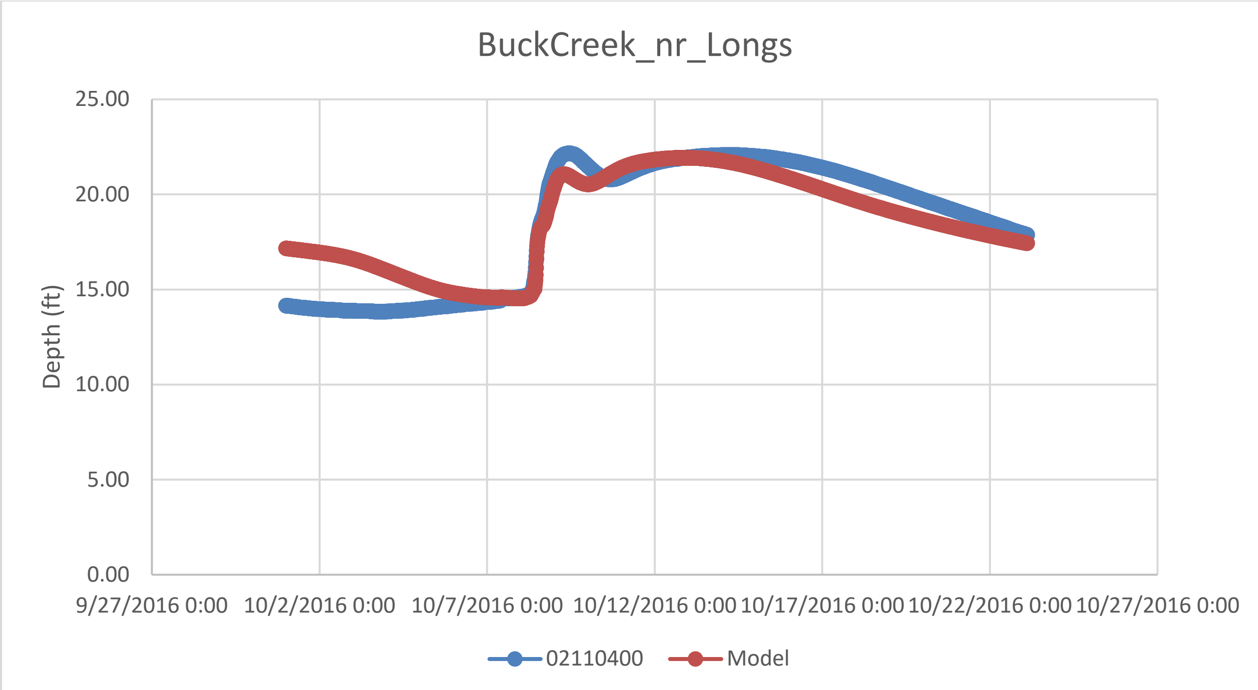


Figure 63. Validation for Hurricane Matthew Gage 02110400

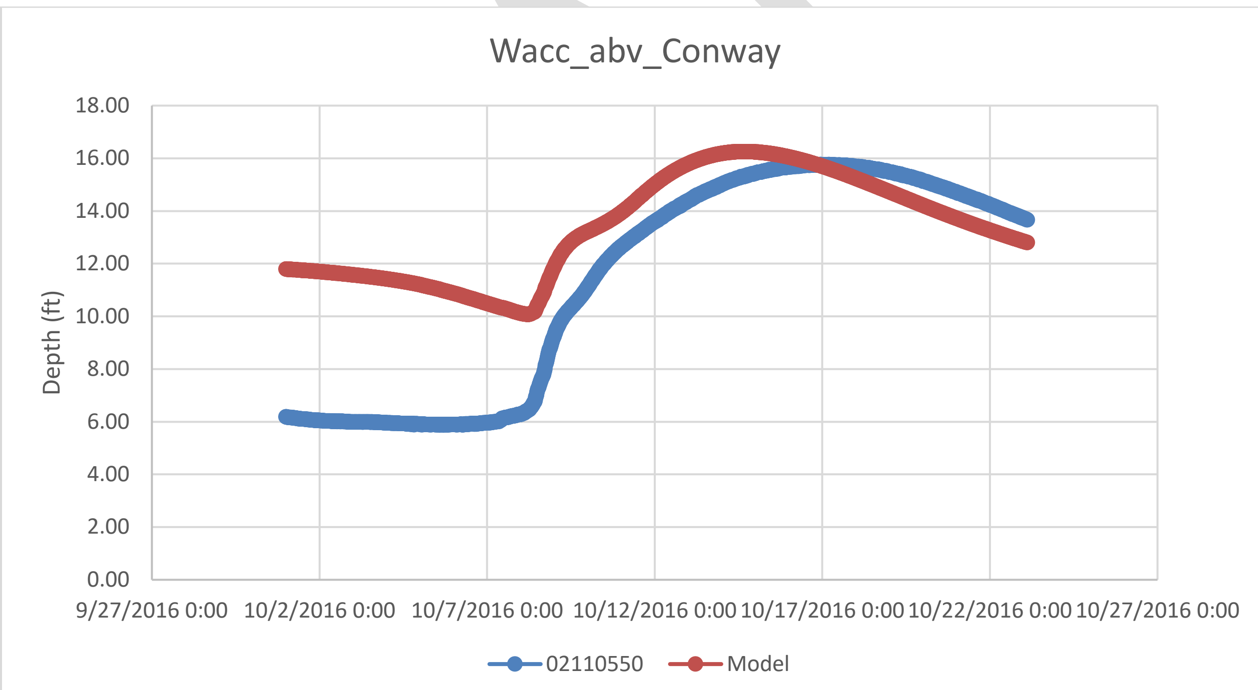


Figure 64. Validation for Hurricane Matthew Gage 02110550

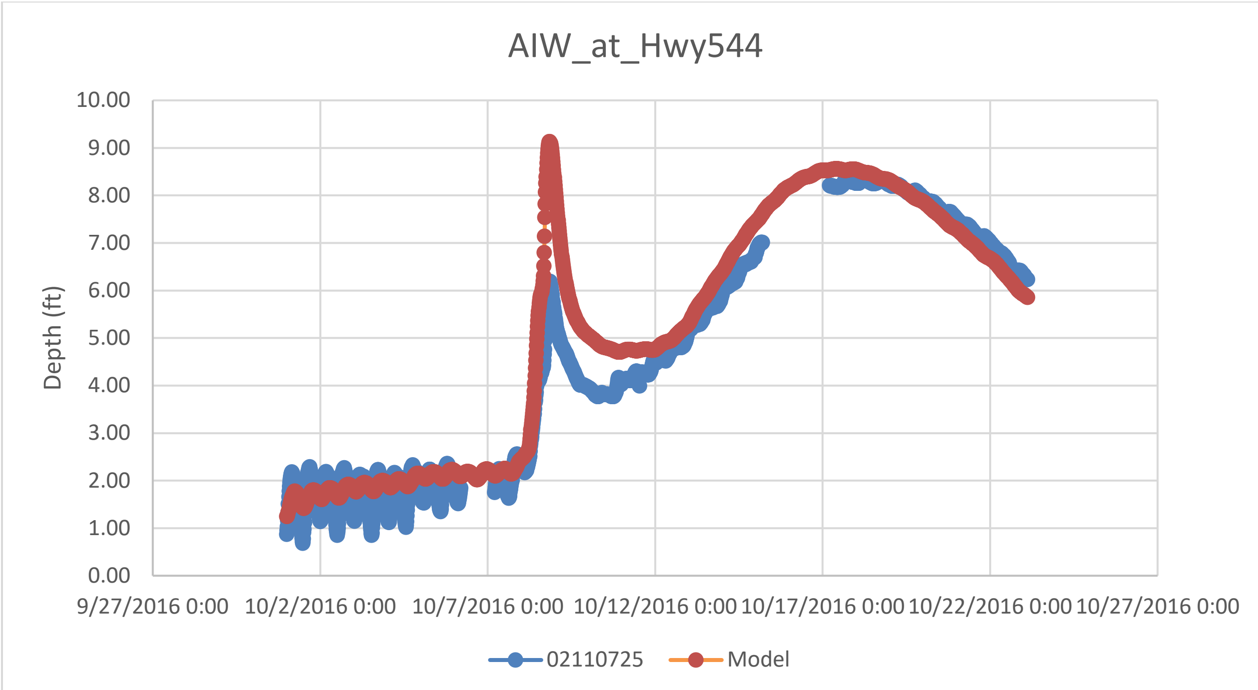


Figure 65. Validation for Hurricane Matthew Gage 02110725

Table 20 shows results from the validated Hurricane Matthew model run. Additional discussion of sensitivity analysis and parameter adjustment can be found in Section 5.3.4.

Table 20. Summarized Results of Hurricane Matthew Validation

Gage Location	Gage ID	Observed Flow (cfs)	Computed Flow (cfs)	Std. Dev	Variance (%)
Waccamaw_at_Freeland	2109500	33.46	32.855	0.43	1.81%
Wacc_nr_Longs	2110500	21.17	21.11	0.04	0.28%
BuckCreek_nr_Longs	2110400	22.18	21.926	0.18	1.15%
Wacc_abv_Conway	2110550	15.77	16.257	0.34	3.09%
CrabtreeSwamp_at_Conway	2110701	12.07	12.098	0.02	0.23%
Wacc_at_ConwayMarina	2110704	11.75	11.62	0.09	1.11%
AIW_at_Hwy544	2110725	8.35	9.133	0.55	9.38%
Wacc_at_Buksport	2110802	8.49	8.619	1.11	1.52%
Wacc_at_Pawleys	21108125	5.84	6.277	0.31	7.48%
PeeDee_at_Hwy701	02135200 **	112050	113237.6	839.78	1.06%

5.3.3 Hurricane Joaquin (October 2015 Flood) Validation

Model validation was performed to validate the water surface elevation and flow output results for Hurricane Joaquin and the subsequent flood event. Figure 66 shows total rainfall accumulation across the project area and Figure 67 shows the approximate maximum point rainfall accumulation timeseries. Similarly to Hurricane Florence the hyetograph data is pulled from location at the approximate maximum precipitation depth for the event. See the point on the map below for the approximate location.

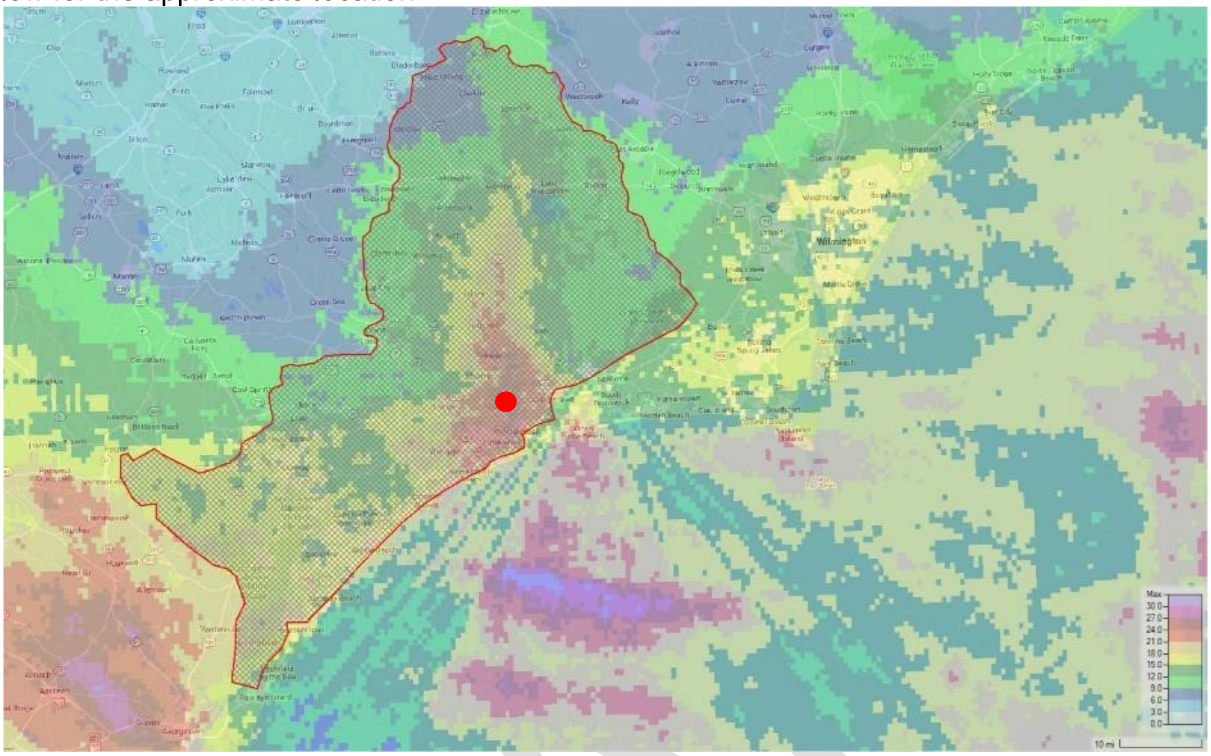


Figure 66. Total Rainfall Accumulation for Hurricane Joaquin

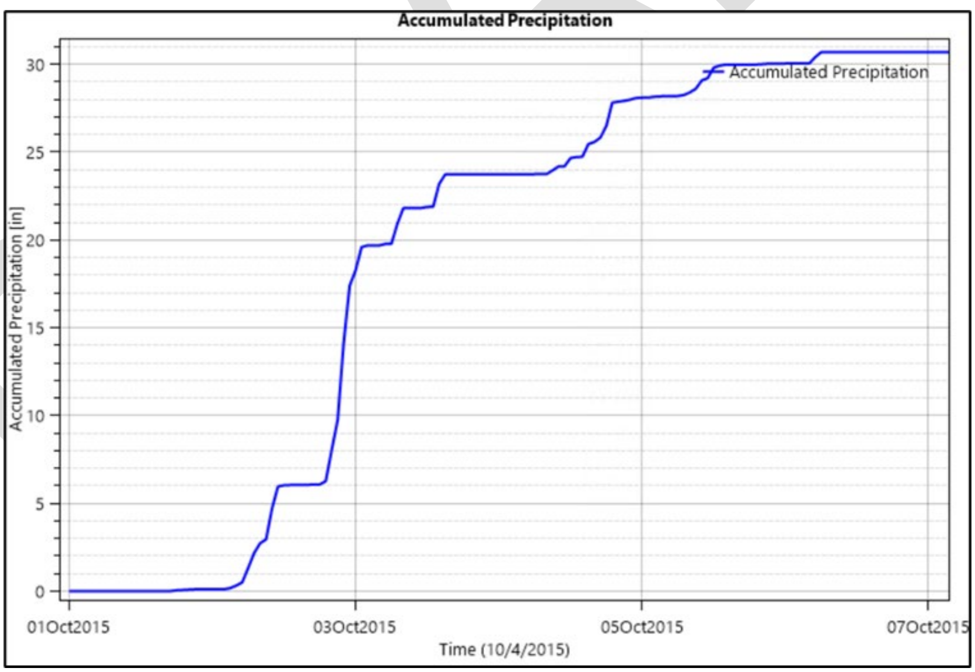


Figure 67. Approximate Maximum Accumulated Precipitation for Hurricane Joaquin

Results for the Hurricane Joaquin calibration event at select USGS gages are shown in Figure 59 through Figure 63.

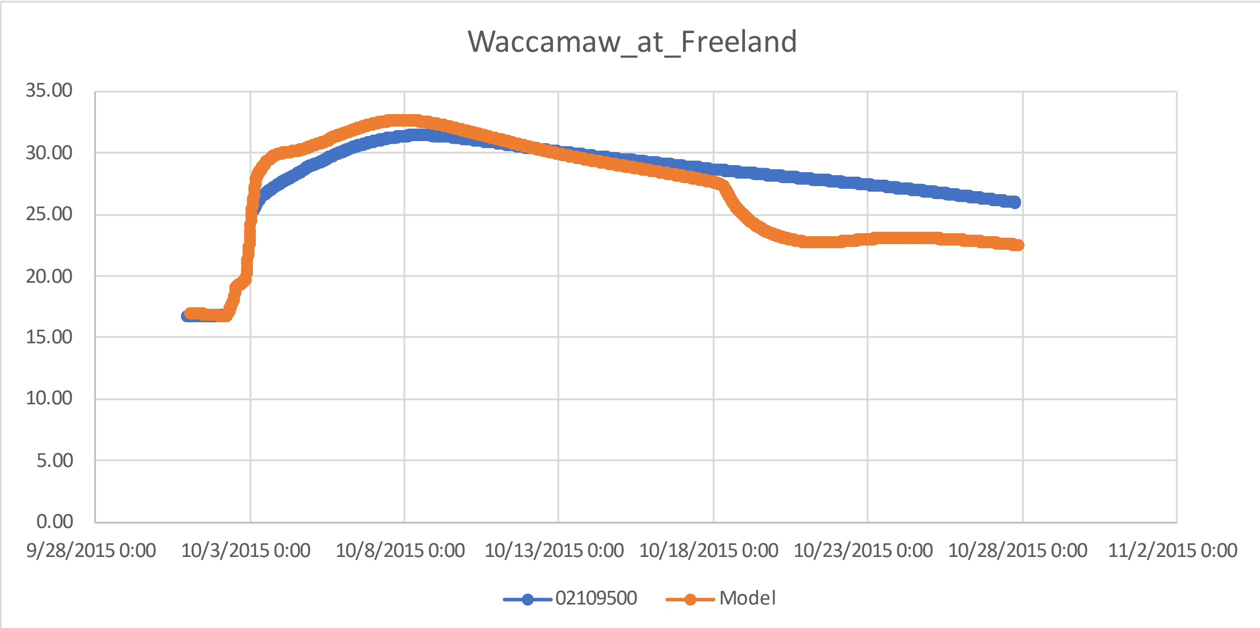


Figure 68. Validation for Hurricane Joaquin Gage 02109500

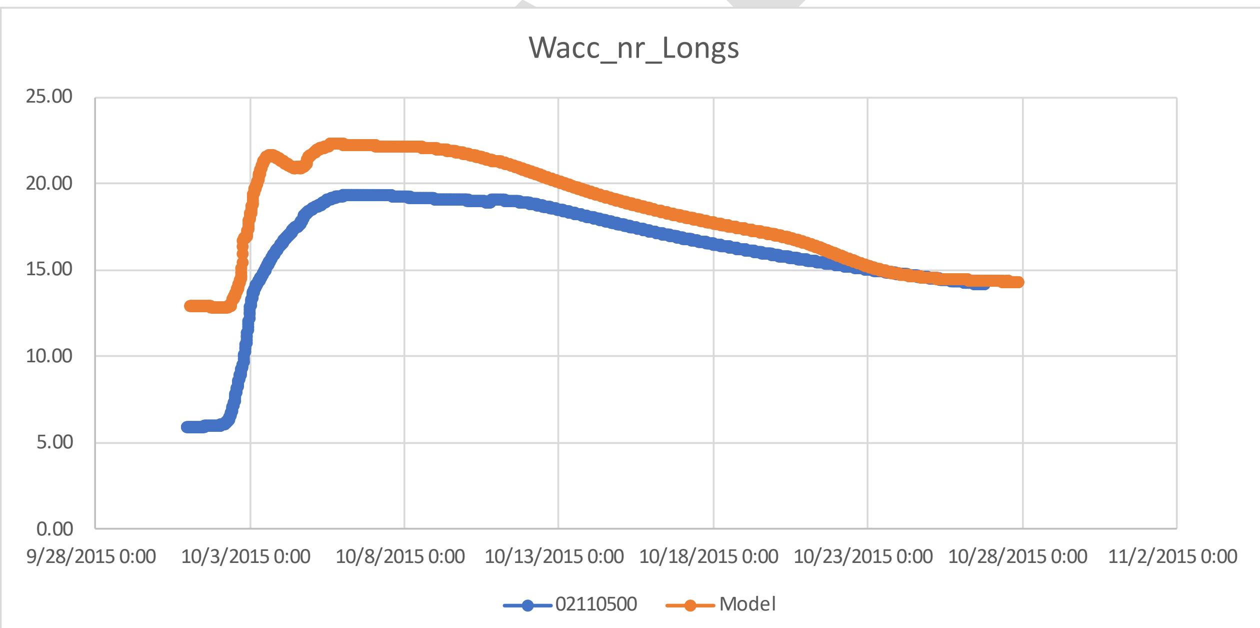


Figure 69. Validation for Hurricane Joaquin Gage 02110500

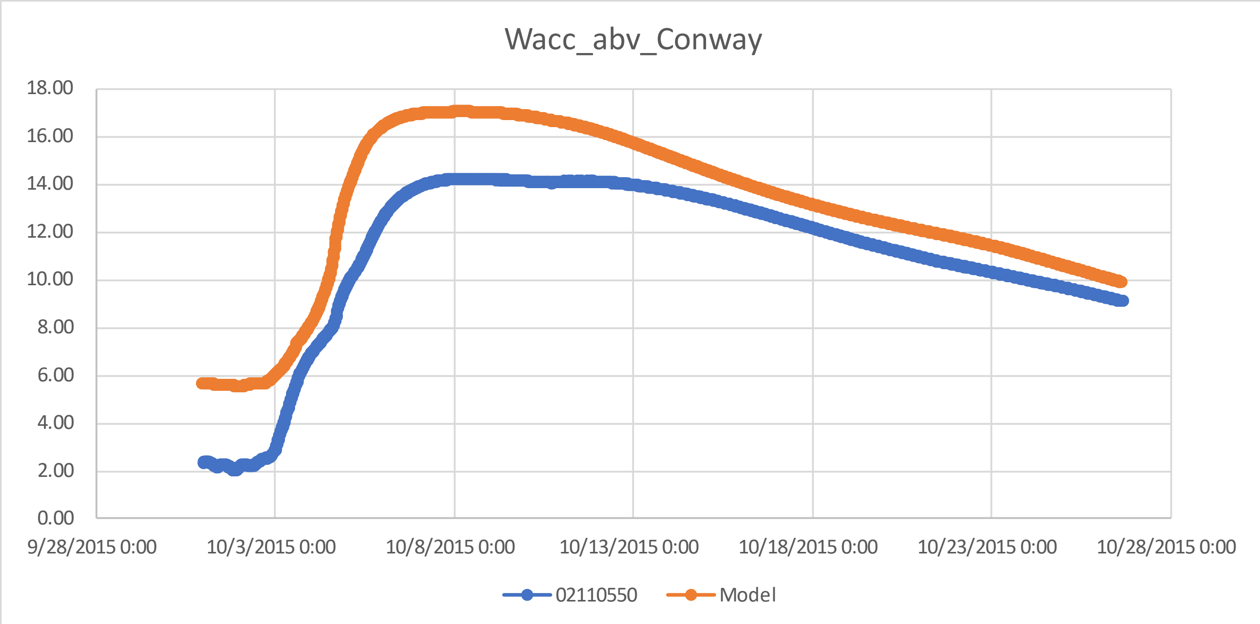


Figure 70. Validation for Hurricane Joaquin Gage 02110550

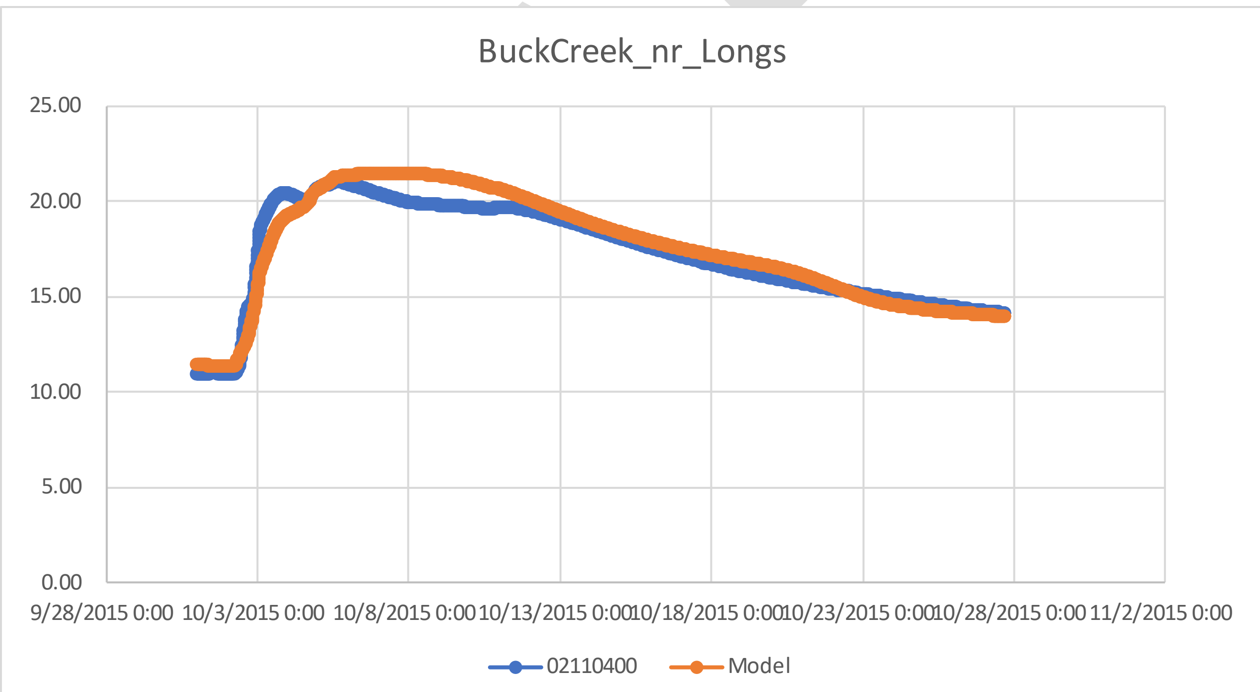


Figure 71. Validation for Hurricane Joaquin Gage 02110400

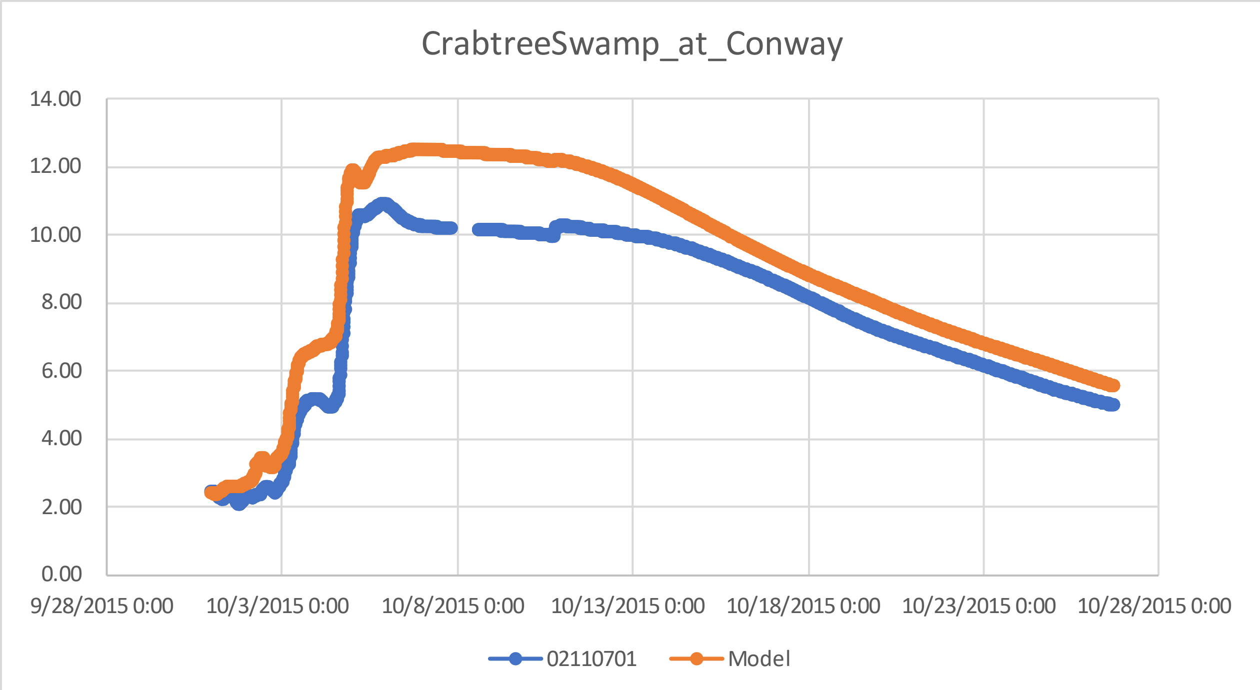


Figure 72. Validation for Hurricane Joaquin Gage 02110701

Table 20 shows results from the validation of Hurricane Joaquin model run. Additional discussion of sensitivity analysis and parameter adjustment can be found in Section 5.3.4.

Table 21. Summarized Results of Hurricane Joaquin Validation

Gage Location	Gage ID	Observed Flow (cfs)	Computed Flow (cfs)	Std. Dev	Variance (%)
Waccamaw_at_Freeland	2109500	31.41	32.624	0.86	3.87%
Wacc_nr_Longs	2110500	22.01	22.366	2.1	1.66%
BuckCreek_nr_Longs	2110400	21.05	21.45	0.28	1.90%
Wacc_abv_Conway	2110550	16.19	16.994	1.98	4.97%
CrabtreeSwamp_at_Conway	2110701	12.07	12.6	0.37	4.39%
Wacc_at_ConwayMarina	2110704	12.09	12.415	1.64	2.69%
AIW_at_Hwy544	2110725	7.9	7.707	1.26	2.69%
Wacc_at_Buksport	2110802	8.49	8.02	1.04	5.53%
Wacc_at_Pawleys	21108125	4.89	4.966	0.05	1.55%
PeeDee_at_Hwy701	02135200 **	51240	49890.48	954.25	2.63%

5.3.4 Sensitivity Analysis and Results

Certain parameters and inputs to the HEC-RAS model can drastically impact the resulting water surface elevation and flow values. To understand the sensitivity of the model results to changes in the input parameters, sensitivity tests and model calibration were performed to identify what changes to the input data would be necessary to increase the model's accuracy. The sensitivity of modeled water surface elevation and flow results were assessed for the following items:

- Initial Flow Conditions
- Bathymetry and Terrain
- Roughness Values

– Hydraulic Structures
– Climate Non-Stationarity
– Coastal Impacts

It should be noted that these sensitivity checks were modeled cumulatively with each subsequent analysis. For example, the results of the initial flow conditions analysis were included in each of the subsequent analyses (roughness, bathymetry, and bridges), the results of the roughness value analysis were included in the bathymetry and bridges analyses, and so on.

5.3.4.1 Initial Flow Conditions Sensitivity

Once the initial model geometry and inputs were developed, Hurricane Florence was simulated using a “dry” initial condition. This simulation, without an initial condition set up, did not calibrate well to actual stream gage measurements. Therefore, it was necessary to develop an initial conditions input file to introduce a base flow and “wet” the model before performing calibration runs. This was done by simulating a 2-year, 96-hour rainfall event for 60 days. The starting conditions for each calibration event were selected based on the timestep of the receding limb of the hydrograph that matched with the stream gauge conditions at the start of the calibration simulation. Figure 73 shows the USGS Stream gage at Freeland, NC (01209500) and the model results for without and with initial conditions startup file.

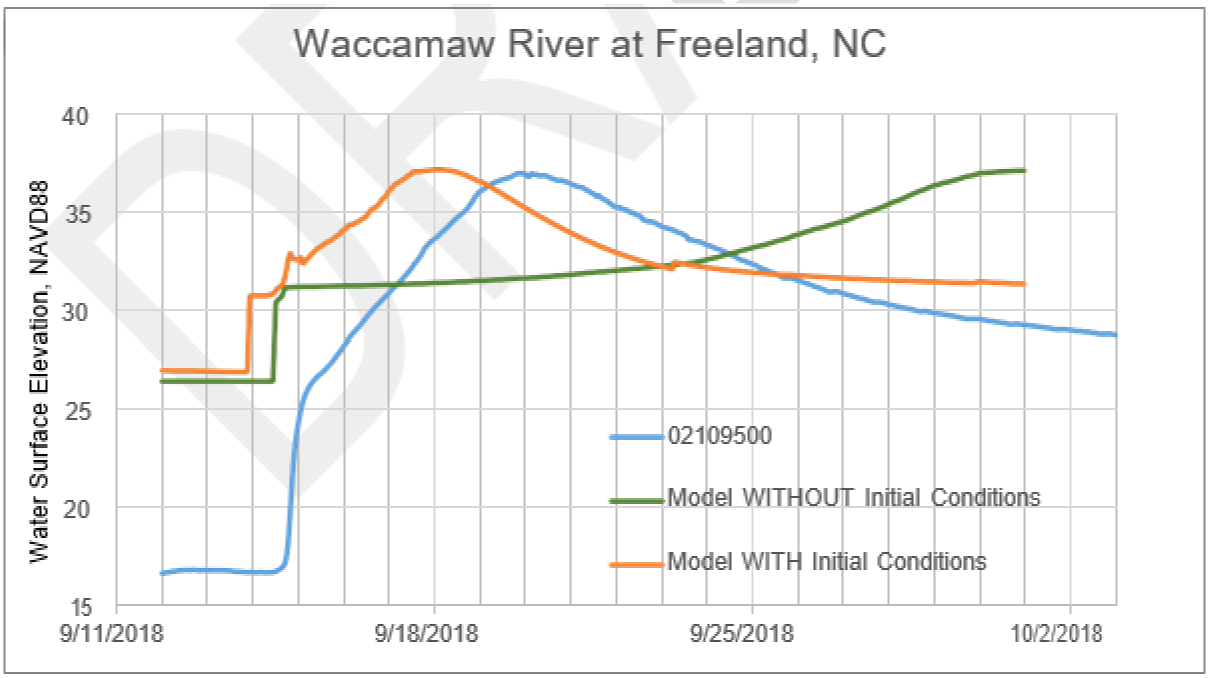


Figure 73. Waccamaw River USGS Stream Gage vs. Model results for With and without Initial Conditions setup

Based on the sensitivity analysis results for initial conditions setup, the water surface elevation and flow results are very sensitive to this input. Initial conditions setup should be considered when developing any models using the 2D rain-on-grid approach for this basin.

5.3.4.2 Bathymetry and Terrain

The addition of supplemental estimated bathymetry was also assessed because the provided bathymetry only covered a portion of the Waccamaw River. A review of the impacts of estimated bathymetry was necessary because most of the stream gages had measurements well below the lidar elevations. This elevation difference is due to the lidar being flown during relatively high water in the channels. Estimated bathymetry (beyond what was provided by USACE SAC) was incorporated to the HEC-RAS terrain file as discussed in this report's Model Data and Layers section. Figure 74 shows the USGS Stream gage at Freeland, NC (01209500) and the model results for the with and without the additional estimated channel bathymetry.

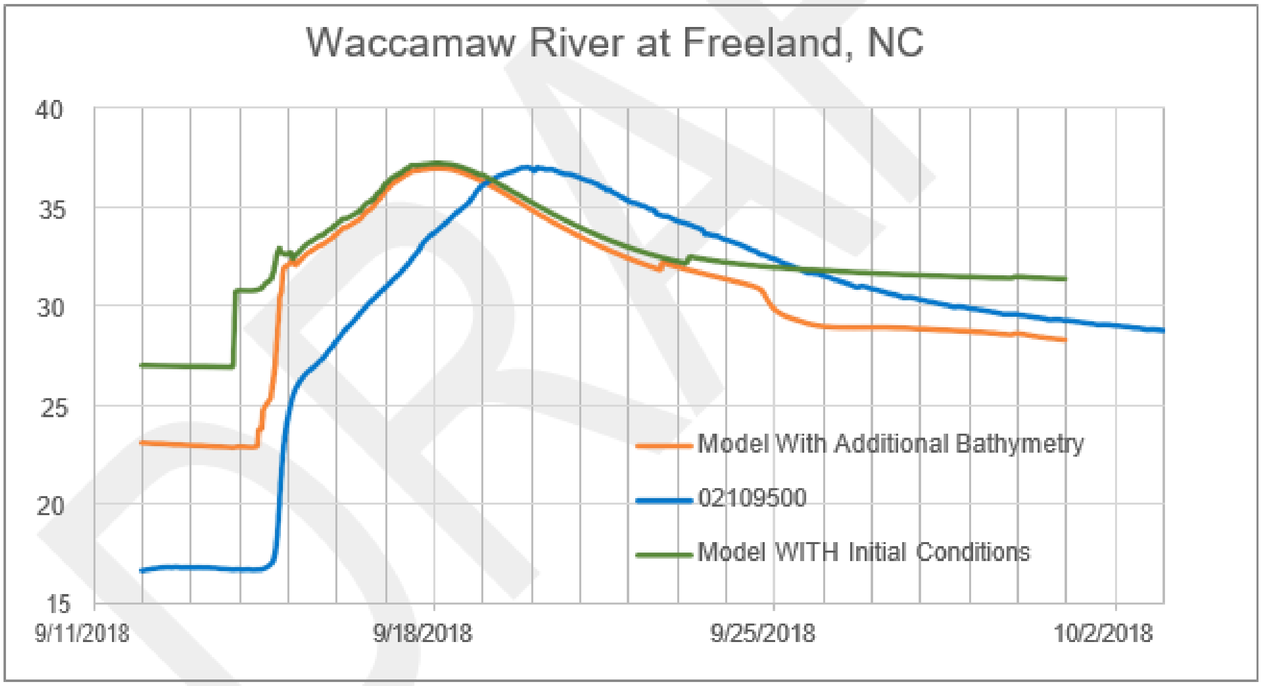


Figure 74. Waccamaw River USGS Stream Gage vs. Model Results for With and Without Additional Bathymetry

Based on these results, the additional estimated bathymetry was included in the final model terrain/geometry because the water surface elevations at lower elevations were sensitive to this parameter. Additionally, where water was ponding behind embankments, hydro-enforcement was performed using terrain "slices" to represent hydraulic structures where field survey was unavailable. These slices were added to simulate the ability to pass flow through the embankments and reduce the attenuation that was occurring due to a large amount of ponding. Figure 75 shows an example of a terrain slice.

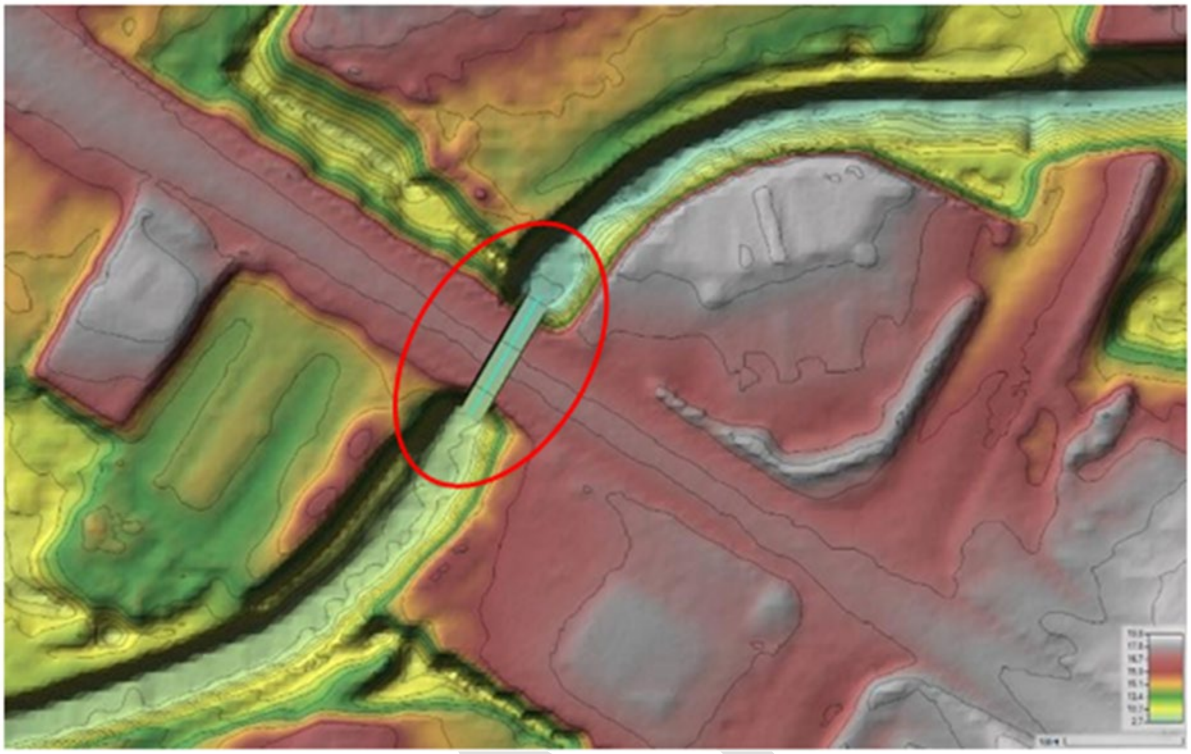


Figure 75. Example of a "terrain slice" hydro-enforcement through a road embankment

5.3.4.3 Roughness Values

The roughness values associated with the 2D mesh can significantly impact the resulting water surface elevation and flow values. As discussed in the Model Approach and Methodology section, three calibration regions were developed to represent the major roughness regions (base, floodplain, and channel). Modifications were made to the roughness values associated with those regions to assess their sensitivity.

The sensitivity analysis results indicate that the base roughness value significantly impacts the timing of the flood peak. The floodplain roughness values impact the timing of the flood peak, but they also substantially impact the resulting water surface elevations during large flood events. The channel roughness appeared to be the least impact on the timing of the flood peak and the water surface elevations. Channel values had more of an impact on the front and back ends of the flood when the water surface elevations were lower and primarily contained within the channel. The NLCD Woody Wetlands land cover type dominated the Waccamaw River watershed, so the model results were very sensitive to changes in roughness value for that land cover type. The initial roughness value associated with that land cover type was 0.2. This initial value was increased to 0.3 in the base mesh and reduced to 0.15 in the floodplain mesh as part of the calibration process. The final Manning's Roughness values are presented in Table 21.

Table 22: Manning's Roughness Coefficient Table

NLCD ID	Land Cover Description	Base Area	Floodplain Area	Channel Area
11	Open Water	0.025	0.02	0.04
21	Developed Open Space	0.024	0.024	0.04
22	Developed Low	0.03	0.03	0.04

	Intensity			
23	**Developed Medium Intensity**	0.025	0.025	0.04
24	**Developed High Intensity**	0.02	0.02	0.04
31	**Barren Land Rock-Sand-Clay**	0.02	0.02	0.04
41	**Deciduous Forest**	0.3	0.15	0.04
42	**Evergreen Forest**	0.3	0.15	0.04
43	**Mixed Forest**	0.3	0.15	0.04
52	**Shrub-Scrub**	0.08	0.03	0.04
71	**Grassland-Herbaceous**	0.05	0.024	0.04
81	**Pasture-Hay**	0.07	0.03	0.04
82	**Cultivated Crops**	0.07	0.04	0.04
90	**Woody Wetlands**	0.3	0.15	0.04
95	**Emergent Herbaceous Wetlands**	0.1	0.048	0.04

Figure 67 shows the impact of the roughness value modifications compared to the “with additional bathymetry” simulation discussed in the previous section.

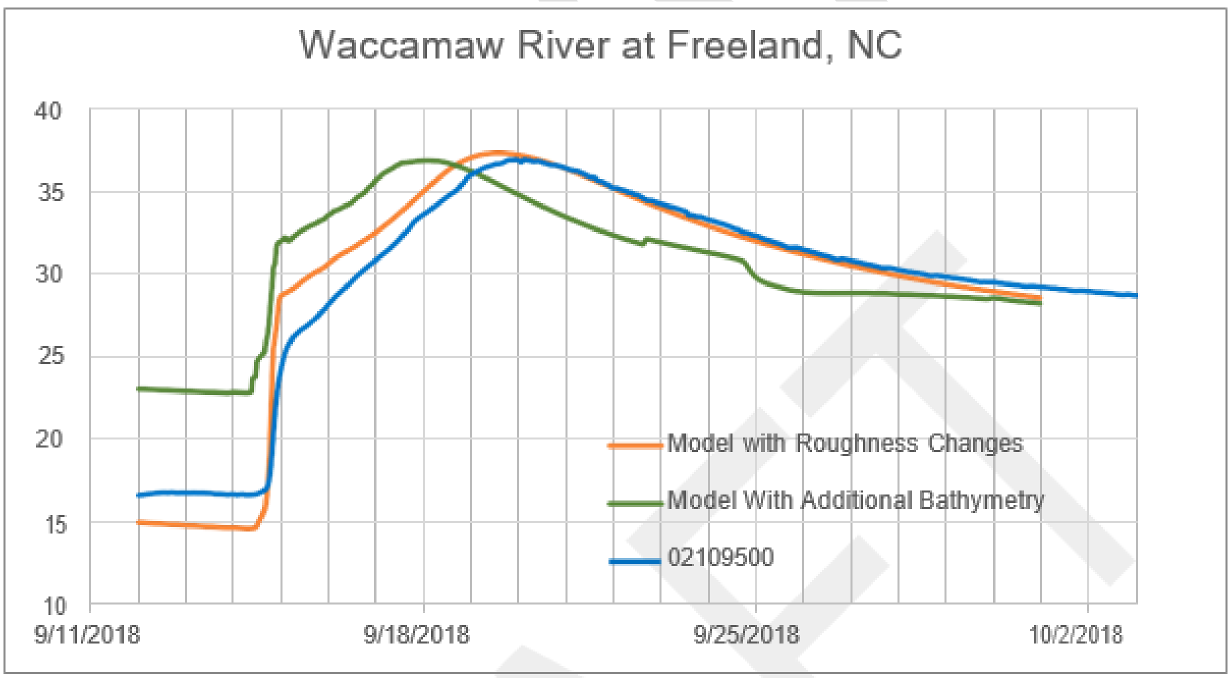


Figure 76. Waccamaw River USGS Stream Gage vs. Model Results for Modifications to Roughness Values

R-squared is a statistical measure that indicates how much of the variation of a dependent variable is explained by an independent variable in a regression model. The resulting R- squared value was above 0.90, which indicated good calibration to the actual stream gage data.

5.3.4.4 Hydraulic Structures

An analysis of how hydraulic structures (bridges) impact water surface elevations and flows for this watershed was also performed. Because most of the flow velocities throughout the Waccamaw River watershed are very low (less than 1 foot per second, fps), it was beneficial to test the benefits of incorporating the bridges in the 2D mesh because they tend to cause local model instabilities, sometimes increasing model run times and skewing the results.

After testing multiple bridges along the Waccamaw River, the results indicated that the bridges were causing minor water surface elevation impacts within the vicinity of the bridges (typically less than 0.1 feet). This negligible impact is primarily due to the low channel and floodplain velocities. In addition, the bridge approach embankments appear to have a larger impact on the restriction of flow in the floodplains, so they tend to control the losses associated with each roadway crossing of the floodplain. The embankments were included in the 2D mesh, so the bulk of the losses were accounted for at each roadway crossing of the floodplain. Nine bridges were included in the model geometry.

5.3.4.5 Climate Non-Stationarity Sensitivity Analysis

The sensitivity of the Waccamaw River's hydrologic response to climate non-stationarity was tested using the methodology developed by the North Carolina Institute of Climate Studies (NCICS) for SERDP and NOAA. More information about the project that developed the methodology can be found at https://precipitationfrequency.ncics.org/. The website "provides scientifically based estimates of future values for intensity– duration–frequency (IDF) curves for heavy precipitation events for locations in the United States. These future values incorporate changes due to potential global warming."

This website has a tool that is similar to the NOAA PFDS website. The tool adjusts the current NOAA Atlas 14 Average Recurrence Interval Precipitation Depths to account for the chosen future climate scenario. For the sensitivity test, the 2075 RCP4.5 scenario (mid-range greenhouse gas scenario with an approximately 50-year horizon) was selected. Because only point data was available, the location that represented the average precipitation depth across the Waccamaw River 2D area was selected (Lat 33.85559, Lon -78.9368) and a percent increase was calculated for the 50-year horizon for the 100-year event. At that location, the increase in rainfall for that time horizon goes from approximately 12.68 inches to 14.53 inches for the 100-year average recurrence interval. This is an increase of approximately 14.6%. This percent increase was then applied to the 100-year AEP NOAA Atlas 14 precipitation grid by using a scaling factor of 872.7 in HEC-HMS. Note that the factor developed for this single point was applied to all of the HEC-HMS basins for the Pee Dee River. It is important to understand that the values may vary across large watersheds, so a more detailed study would be needed to determine how spatial variability across the Pee Dee River basin could change the results. Additionally, comparisons were not done for other event scenarios. It is possible that the percent increase is not consistent between average recurrence intervals and may be higher or lower depending on the scenario. Further investigation would be required to determine this variability.

The results of the simulation indicate that climate non-stationarity could have a significant impact on future water surface elevations and flooding conditions within the Pee Dee and Waccamaw River basins. A 14.6% increase in total rainfall for a 96-hour event produced a rise in water surface elevation of more than 2 feet for the Waccamaw River at Conway, SC as shown in Figure 77. It should be noted that the 90% confidence intervals for the rainfall values are large for the 100-year event, 10.70 to 15.93 inches for Atlas 14 and 11.73 to 19.12 inches for the NCICS values.

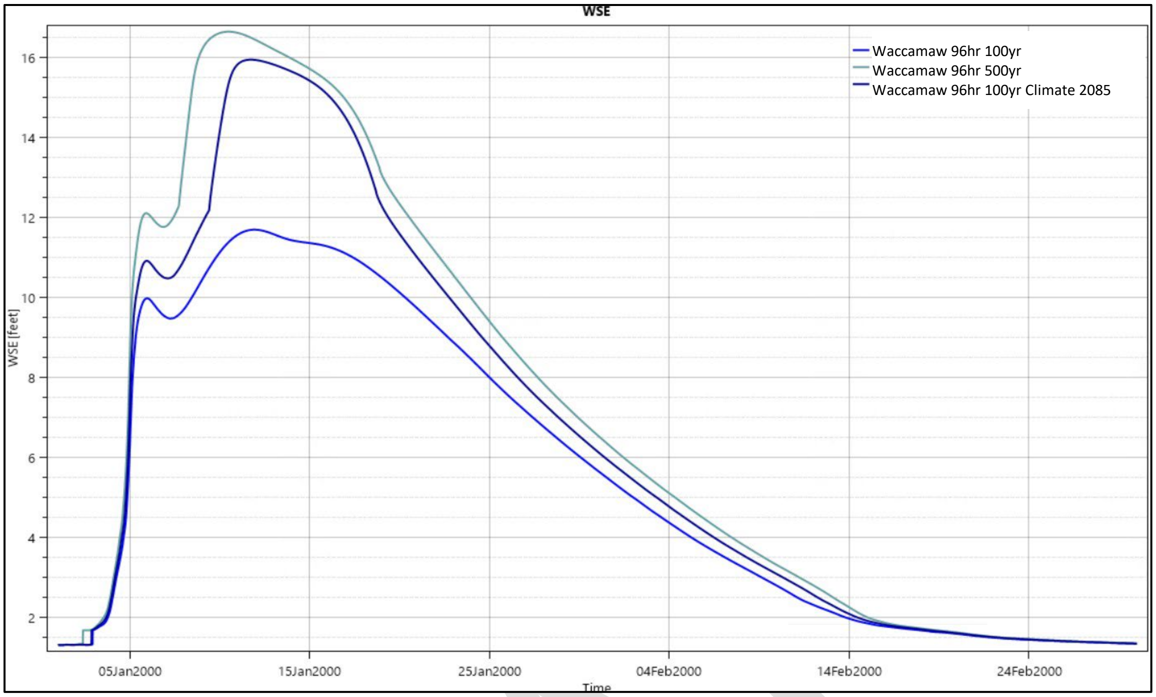


Figure 77. Model Results for Waccamaw River at Conway, SC- Comparison of Current versus Future Climate Conditions

5.3.4.6 Coastal Impacts Analysis

Sea level change (SLC) for the Waccamaw River study was evaluated following the guidelines presented in USACE Engineer Pamphlet (EP) 1100-2-1 “Procedures to Evaluate Sea Level Change: Impacts, Responses and Adaptation”. The purpose of the EP was to provide instructional and procedural guidance to analyze and adapt to the direct and indirect physical and ecological effect of projected sea level change on USACE projects and systems of projects needed to implement Engineer Regulation (ER) 1100-2-8162.

ER 1100-2-8162 “Incorporating Sea Level Change in Civil Works Programs” provides both a methodology and a procedure for determining a range of SLC estimates based on global sea level change rates, the local historic sea level change rate, the construction (base) year of the project, and the design life of the project. Three estimates are required by the guidance, a Low (Baseline) estimate representing the minimum expected SLC, an Intermediate estimate, and a High estimate representing the maximum expected SLC. The guidance will be used to evaluate the future sea levels, the impacts to the Waccamaw River study area during a 50-Year period and to assess the risk associated with the SLC estimates.

An initial step in evaluating sea level change for the Waccamaw River basin study was to identify a near-by NOAA water level gage with a sufficiently long data record and analysis of SLC are included in Appendix A2: Climate and Sea Level Change.

The NOAA sea level viewer provides layers that define areas that are affected by coastal effects at various degrees. Figure 78 shows the impact on Bucksport. Bucksport is heavily affected by the Riverine flooding from both the Waccamaw and Pee Dee River.

Figure 78. Astronomical High tide level for Bucksport, SC from Sea Level Tracker

Four cross sections were obtained at various locations along the Pee Dee and Waccamaw River with the cross-sectional value of water surface elevation comparisons with fluvial-only 1% AEP, SLC at 1% AEP and SLC and Astronomical High Tide combination at 1% AEP. Astronomical High Tide at the Springmaid Pier gage is indicated as 4.16 ft-NAVD88 according to NOAA Datums for 8661070. The results and cross-sectional comparisons are shown in figures 79 through 86. The SLC and tidal effect further upstream near Conway was observed to be the least with less than 0.05 ft in difference for the combination but the furthest downstream experienced 1.35 ft difference. This location is at the confluence with the Pee Dee River as well. There are no proposed structural measures at this location or nearby.

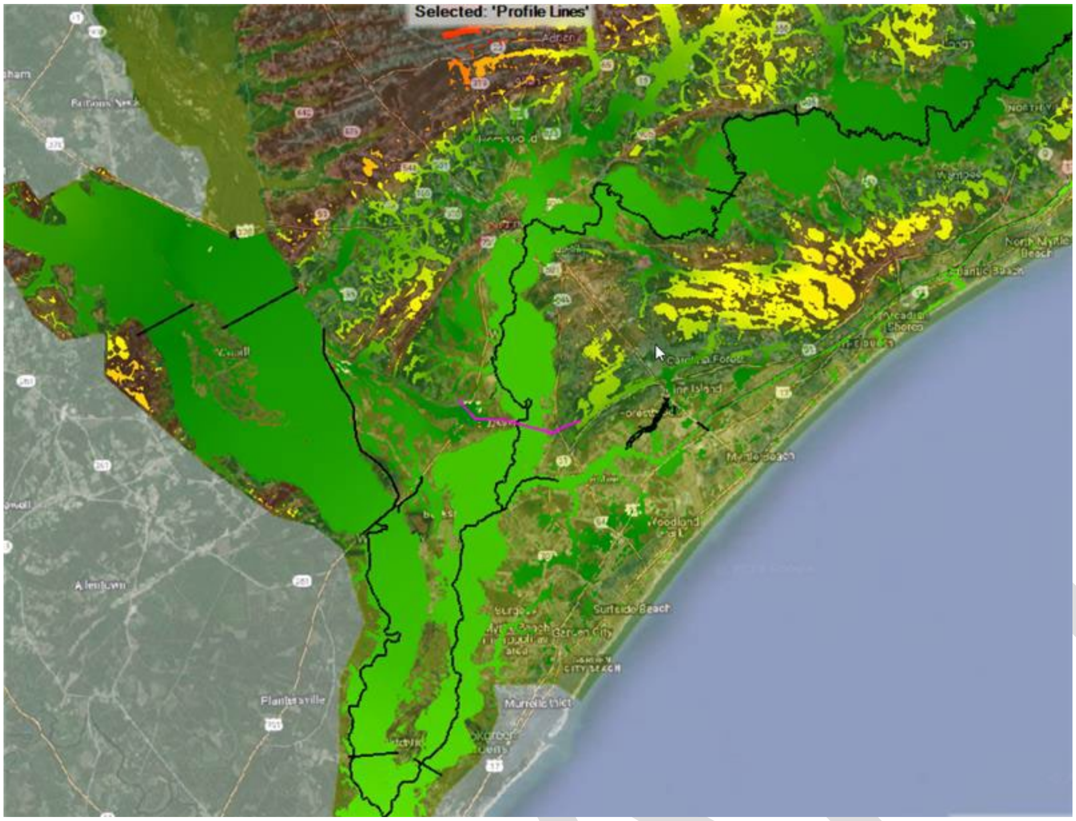


Figure 79. Cross section profile line 2 downstream of Conway and Socastee

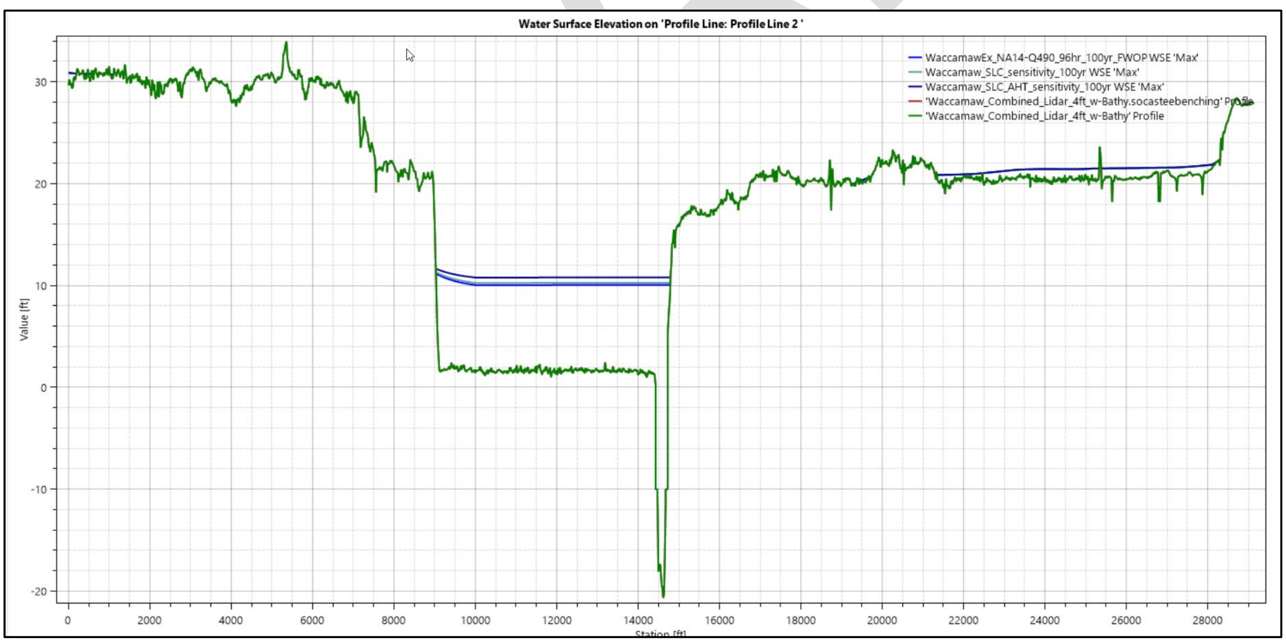


Figure 80. Cross section WSE (max) with FWOP, SLC, and SLC with Astronomical high tide at Profile 2

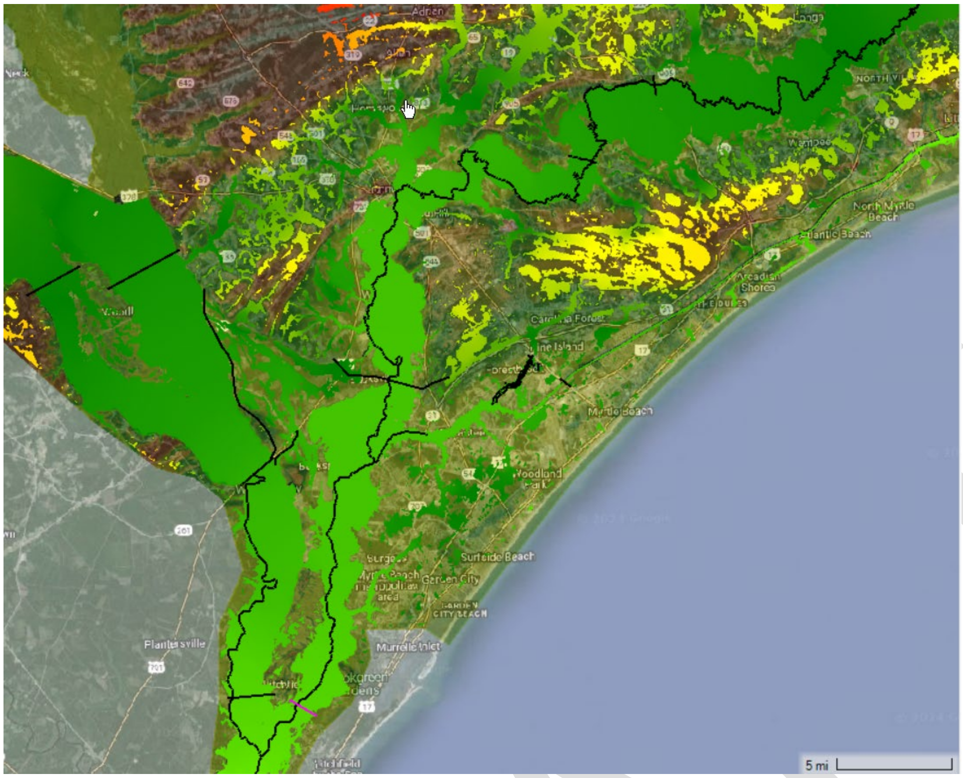


Figure 81. Cross section profile line 2 yr check (furthest downstream)

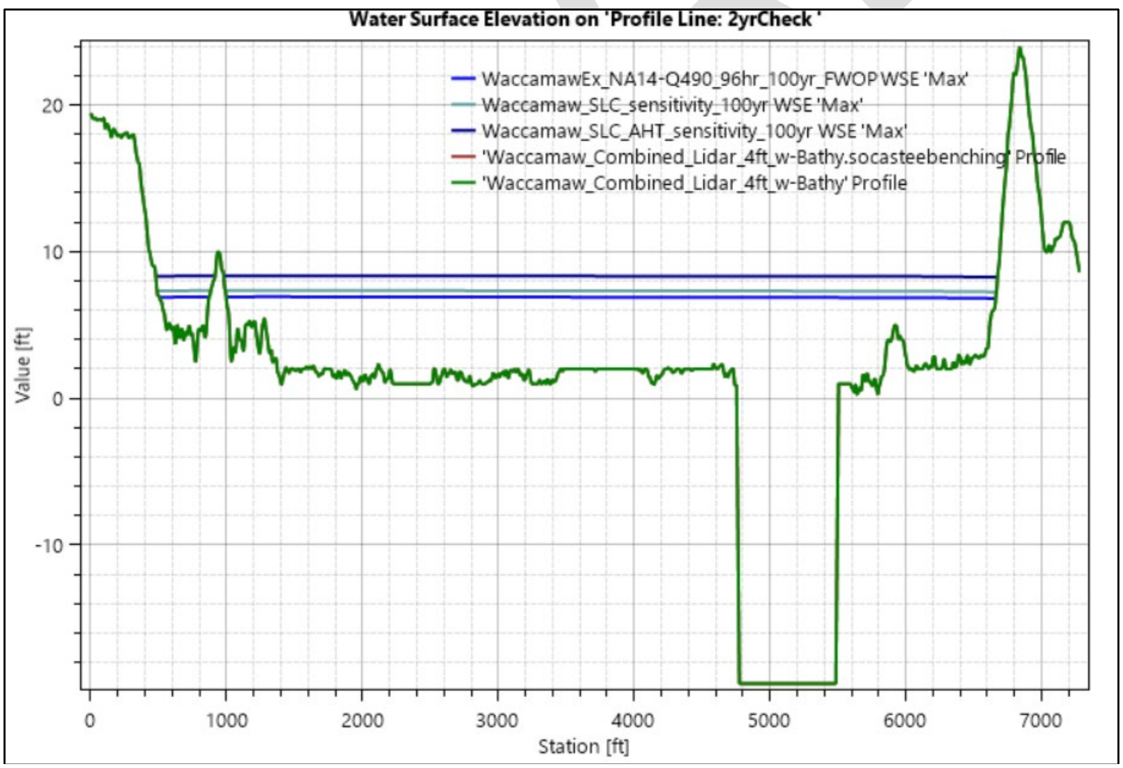


Figure 82. Cross section WSE (max) with FWOP, SLC, and SLC with Astronomical high tide at Profile 2yrCheck

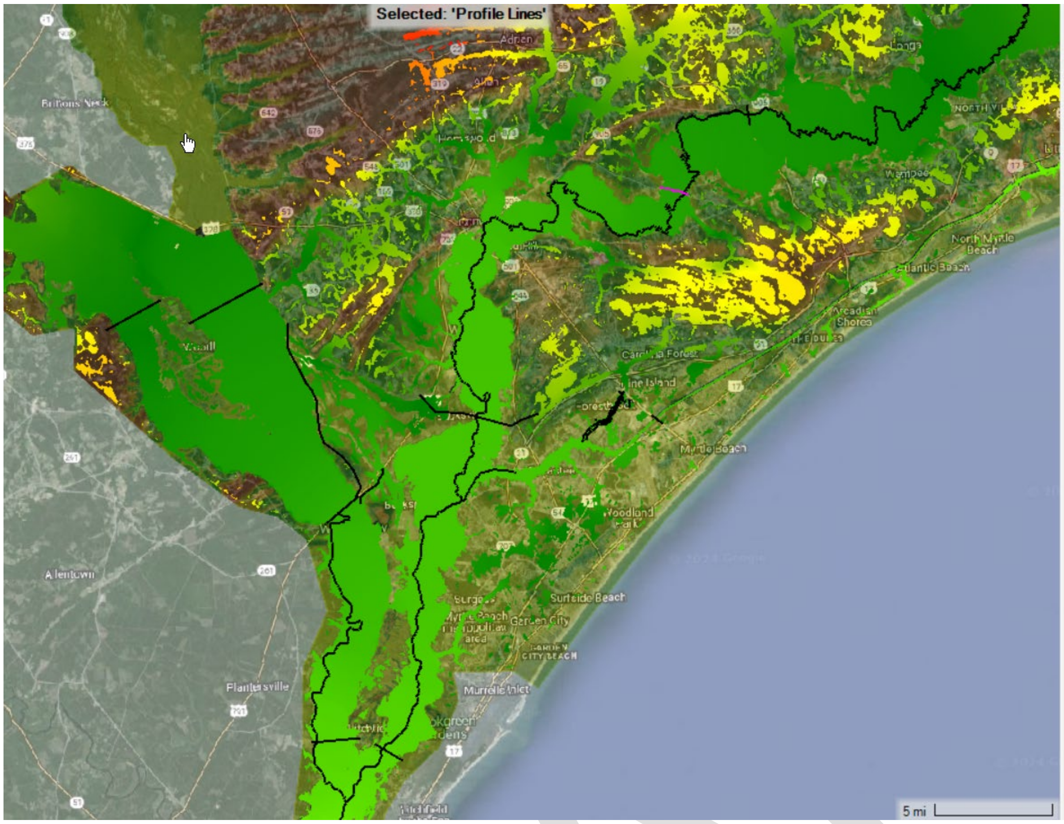


Figure 83. Cross section location 'Profile Line 10' (in pink) in Conway

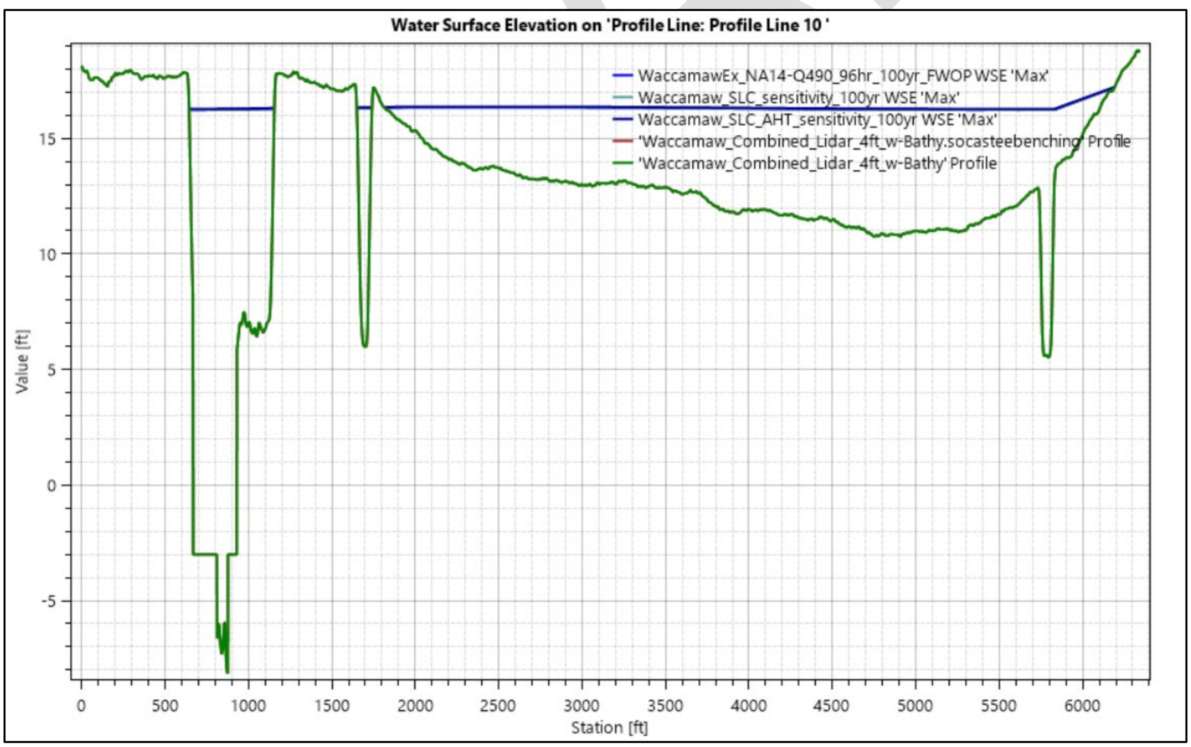


Figure 84. Cross section WSE (max) with FWOP, SLC, and SLC with Astronomical high tide at Profile Line 10

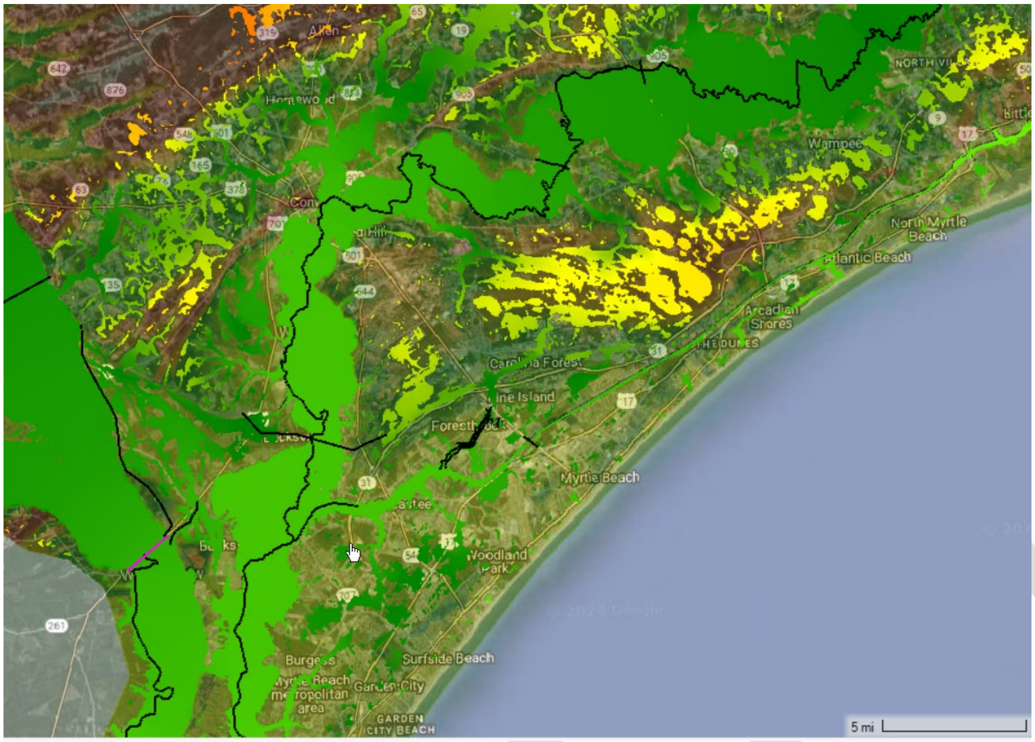


Figure 85. Cross section location (in pink) Pee Dee River

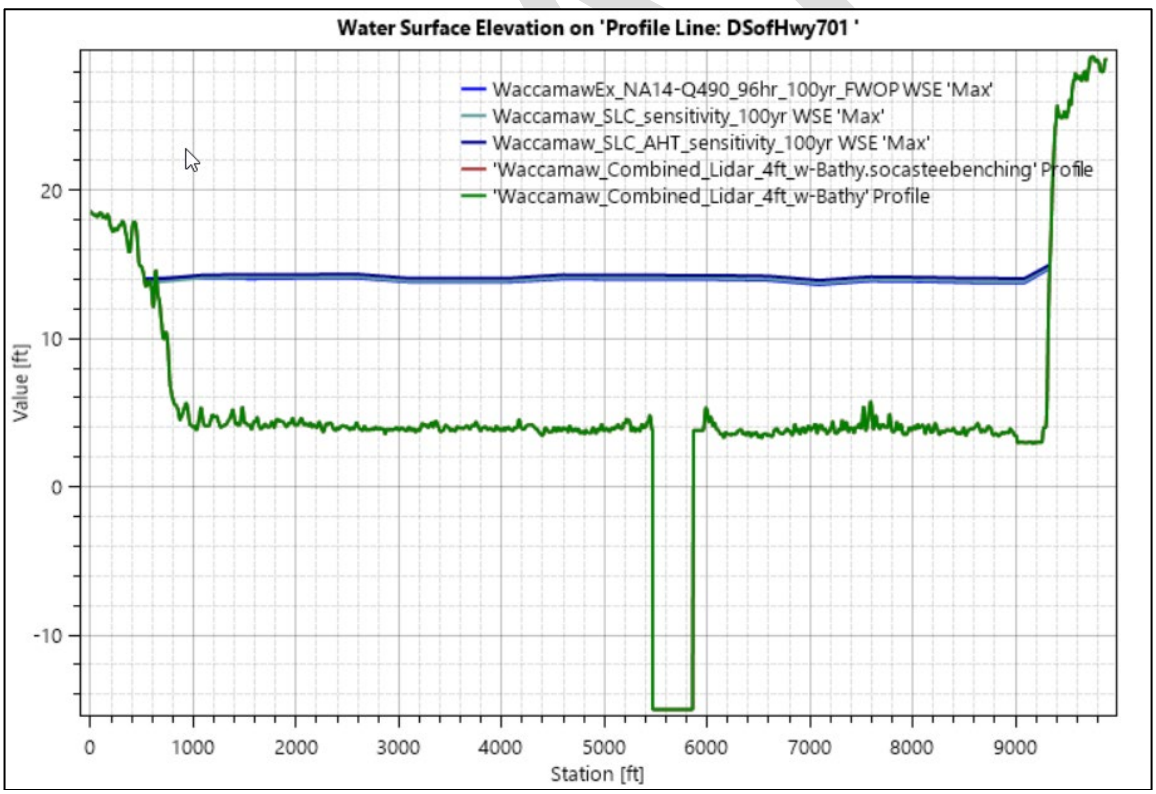


Figure 86. Cross section WSE (max) with FWOP, SLC, and SLC with Astronomical high tide at profile DSof HWy701 Pee Dee River

5.3.4.1 Sensitivity Results and Discussion

Sensitivity Results are presented for the following:
- Initial Flow Conditions (was Very Sensitive)
- Roughness Values (was Sensitive)
- Bathymetry (was Sensitive)
- Hydraulic Structures (was Slightly Sensitive)
- Climate Non-Stationarity (was Sensitive)
- Coastal Effects

Based on the sensitivity analysis results for initial conditions setup, the water surface elevation and flow results are very sensitive to this input. Initial conditions setup should be considered when developing any models using the 2D rain-on-grid approach for this basin. Based on these results, the additional estimated bathymetry was included in the final model terrain/geometry because the water surface elevations at lower elevations were sensitive to this parameter.

The sensitivity analysis results indicate that the base roughness value significantly impacts the timing of the flood peak. The floodplain roughness values impact the timing of the flood peak, but they also substantially impact the resulting water surface elevations during large flood events. The channel roughness appeared to be the least impact on the timing of the flood peak and the water surface elevations. Channel values had more of an impact on the front and back ends of the flood when the water surface elevations were lower and primarily contained within the channel. After testing multiple bridges along the Waccamaw River, the results indicated that the bridges were causing minor water surface elevation impacts within the vicinity of the bridges (typically less than 0.1 feet). This negligible impact is primarily due to the low channel and floodplain velocities.

The NLCD Woody Wetlands land cover type dominated the Waccamaw River watershed, so the model results were very sensitive to changes in roughness value for that land cover type. The initial roughness value associated with that land cover type was 0.2. This initial value was increased to 0.3 in the base mesh and reduced to 0.15 in the floodplain mesh as part of the calibration process.

The results of the simulation indicate that climate non-stationarity could have a significant impact on future water surface elevations and flooding conditions within the Pee Dee and Waccamaw River basins. A 14.6% increase in total rainfall for a 96-hour event produced a rise in water surface elevation of more than 2 feet for the Waccamaw River at Conway, SC as shown in Figure 68. The results of the Coastal Impacts sensitivity analysis were that the further downstream areas were more impacted than the regions further upstream. Sensitivity cross sectional WSE were highlighted.

Several versions of the model were simulated in order to refine the model based on the results of the sensitivity analysis. The versions and the changes that were implemented are described in Table 22.

Table 23. Model versions and descriptions

Model	Model Description
Version 1	NO BATHY, NO HOT START, NO MANNINGS REFINE AREAS, NO MESH REFINE AREAS

Version 2	NO BATHY, WITH HOT START, NO MANNINGS REFINE AREAS, NO MESH REFINE AREAS
Version 3	LIMITED BATHY, WITH HOT START, NO MANNINGS REFINE AREAS, NO MESH REFINE AREAS, FIXED HWY 701 BOUNDARY
Version 4	WITH SOME BATHY, WITH HOT START, NO MANNINGS REFINE AREAS, NO MESH REFINE AREAS
Version 5	WITH SOME BATHY, WITH HOT START, NO MANNINGS REFINE AREAS (BUT WITH MANNINGS CHANGE FOR FOREST), NO MESH REFINE AREAS
Version 6	WITH SOME BATHY, WITH HOT START, NO MANNINGS REFINE AREAS (BUT WITH FURTHER MANNINGS CHANGE IN FORESTS), NO MESH REFINE
Version 7	WITH SOME BATHY, WITH HOT START, NO MANNINGS REFINE AREAS (BUT WITH FURTHER MANNINGS CHANGE IN FORESTS), NO MESH REFINE
Version 8	WITH SOME BATHY, WITH HOT START, NO MANNINGS REFINE AREAS (BUT WITH FURTHER MANNINGS CHANGE IN FORESTS), NO MESH REFINE
Version 9	WITH SOME BATHY, WITH HOT START, NO MANNINGS REFINE AREAS (BUT WITH FURTHER MANNINGS CHANGE IN FORESTS), NO MESH REFINE
Version 10	WITH SOME BATHY, WITH REFINED HOT START FOR LOWER STARTING WSEL, WITH MANNINGS REFINE AREAS, NO MESH REFINE
Version 11	WITH SOME BATHY, WITH REFINED HOT START FOR LOWER STARTING WSEL, WITH MANNINGS REFINE AREAS, NO MESH REFINE
Version 12	WITH SOME BATHY, WITH REFINED HOT START FOR LOWER STARTING WSEL, WITH MANNINGS REFINE AREAS, NO MESH REFINE
Version 13	WITH SOME BATHY, WITH REFINED HOT START FOR LOWER STARTING WSEL, WITH MANNINGS REFINE AREAS, NO MESH REFINE

Calibration results in regard to how well computed time of peak was able to replicate observations at USGS streamflow gage sites is listed in Table 23, Table 24, and Table 25 below. This difference may be attributed to the phenomenon of floodplain storage that was discussed earlier in the section related to differences in peak discharge between computed and observed.

Table 24. Time of Peak Comparison – Waccamaw River Mainstem HEC-RAS Model Computed vs. Observed for Hurricane Florence Calibration Event

Gage Location	Gage ID	Observed Time to Peak (cfs)	Computed Time to Peak (cfs)	Difference (hr)
Waccamaw_at_Freeland	2109500	9/19/2018 23:15	9/19/2018 0:15	1
Wacc_nr_Longs	2110500	9/21/2018 19:30	9/21/2018 6:15	0.6
BuckCreek_nr_Longs	2110400	9/21/2018 10:00	9/20/2018 23:15	0.4
Wacc_abv_Conway	2110550	9/23/2018 21:45	9/22/2018 14:15	1.3
CrabtreeSwamp_at_Conway	2110701	9/26/2018 0:30	9/23/2018 15:30	2.4
Wacc_at_ConwayMarina	2110704	9/26/2018 0:45	9/23/2018 16:15	2.4
AIW_at_Hwy544	2110725	9/27/2018 13:30	9/26/2018 23:00	0.6
Wacc_at_Buksport	2110802	9/27/2018 7:45	9/26/2018 21:15	0.4
Wacc_at_Pawleys	21108125	9/27/2018 13:45	9/27/2018 5:15	0.4
PeeDee_at_Hwy701	02135200 **	9/23/2018 11:00	9/26/2018 19:15	3.3

Table 25. Time of Peak Comparison – Waccamaw River Mainstem HEC-RAS Model Computed vs. Observed for Hurricane Matthew Calibration Event

Gage Location	Gage ID	Observed Time to Peak (cfs)	Computed Time to Peak (cfs)	Difference (hr)
Waccamaw_at_Freeland	2109500	10/12/2016 16:30	10/11/2016 22:30	0.8
Wacc_nr_Longs	2110500	10/14/2016 10:30	10/12/2016 23:45	1.4
BuckCreek_nr_Longs	2110400	10/9/2016 6:30	10/12/2016 19:30	3.5
Wacc_abv_Conway	2110550	10/16/2016 18:30	10/14/2016 15:00	2.1
Wacc_at_ConwayMarina	2110704	10/18/2016 5:15	10/16/2016 2:00	2.1
AIW_at_Hwy544	2110725	10/18/2016 0:30	10/8/2016 20:30	9.2
Wacc_at_Buksport	2110802	10/22/2016 1:00	10/17/2016 19:45	4.2
Wacc_at_Pawleys	21108125	10/17/2016 12:30	10/17/2016 16:30	0.2
PeeDee_at_Hwy701	02135200 **	10/16/2016 7:00	10/16/2016 13:30	0.3

Table 26. Time of Peak Comparison – Waccamaw River Mainstem HEC-RAS Model Computed vs. Observed for Hurricane Joaquin Calibration Event

Gage Location	Gage ID	Observed Time to Peak (cfs)	Computed Time to Peak (cfs)	Difference (hr)
Waccamaw_at_Freeland	2109500	10/8/2015 4:15	10/7/2015 20:00	0.3
Wacc_nr_Longs	2110500	10/6/2015 7:00	10/5/2015 14:00	0.7
BuckCreek_nr_Longs	2110400	10/5/2015 10:15	10/7/2015 12:45	2.1
Wacc_abv_Conway	2110550	10/8/2015 2:00	10/8/2015 4:00	0.1
CrabtreeSwamp_at_Conway	2110701	10/5/2015 16:15	10/6/2015 23:45	1.3
Wacc_at_ConwayMarina	2110704	10/10/2015 17:00	10/7/2015 7:30	3.4
AIW_at_Hwy544	2110725	10/11/2015 22:45	10/5/2015 11:45	6.5
Wacc_at_Buksport	2110802	10/12/2015 2:15	10/11/2015 6:00	0.8
Wacc_at_Pawleys	21108125	10/5/2015 17:45	10/4/2015 21:45	0.8
PeeDee_at_Hwy701	02135200 **	10/10/2015 15:45	10/9/2015 19:45	0.8

5.3.5 Future Projected Sea Level Change Considerations

Per Engineering and Construction Bulletin 2018-14, determination was made as to whether sea level rise would affect river stage by increasing (or decreasing) water surface elevation downstream of the model domain. Based on developed floodplain topography within the HEC-RAS hydraulic model, minimum elevation (NAVD88 datum) for project areas of the Bucksport, Socastee, Conway, and Longs/Red Bluff were under consideration. Figure 87 through Figure 90 show the NOAA Sea Level Rise Viewer with the combination of Astronomical High Tide and the projected Sea Level Rise to 2085.

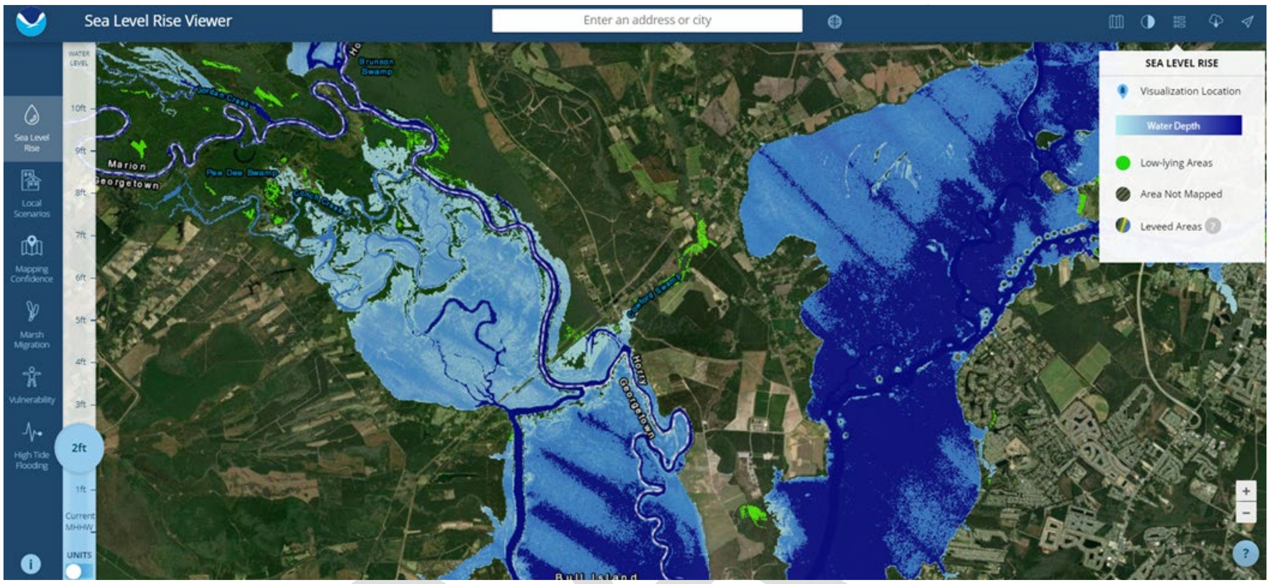


Figure 87. NOAA Sea Level Rise Viewer – SLR projected to the year 2085 for focus area, Bucksport.

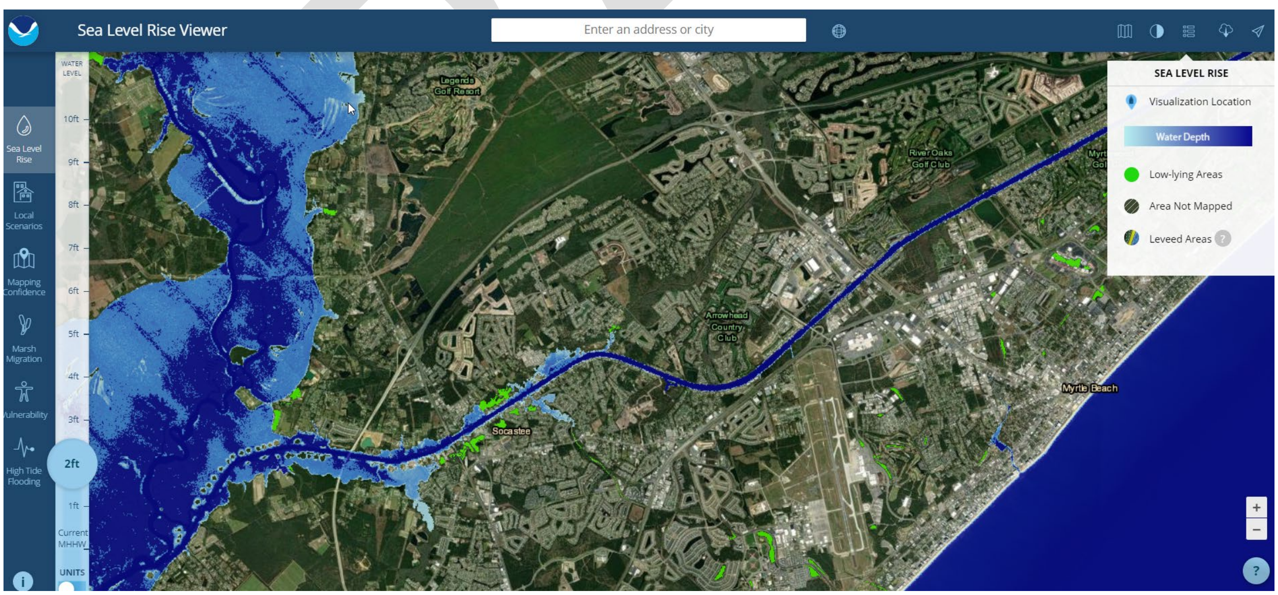


Figure 88. NOAA Sea Level Rise Viewer – SLR Projected to 2085 at Socastee Creek

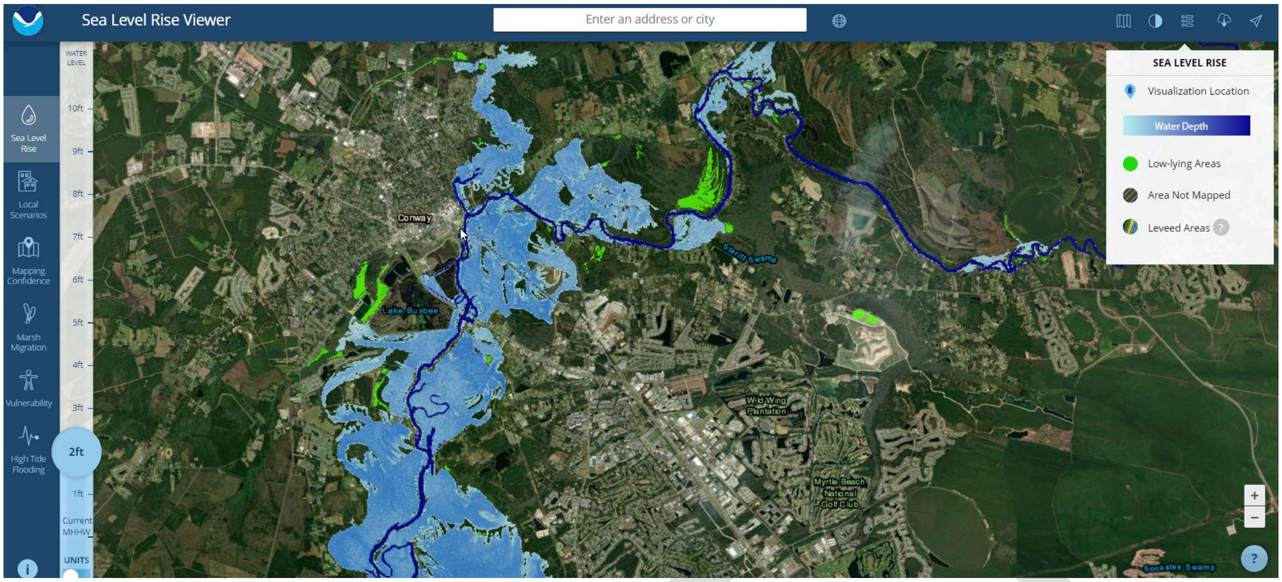


Figure 89. NOAA Sea Level Rise Viewer – SLR Projected to 2085 at Conway

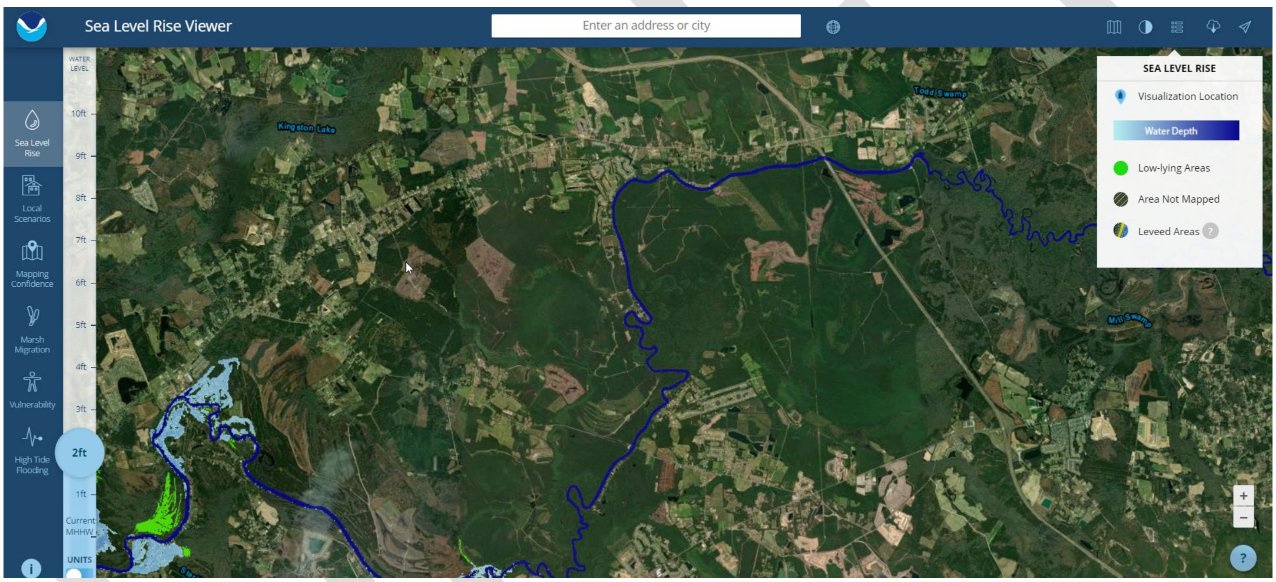


Figure 90. NOAA Sea Level Rise Viewer – SLR Projected to 2085 at Longs/Red Bluff

The study utilized the US Army Corps of Engineers (USACE) online tool, Sea Level Tracker, to assess sea level change (SLC) in the Waccamaw River basin. This tool incorporates extreme water levels based on statistical probabilities derived from historical data. It compares mean sea level (MSL) trends from NOAA tide gauges with USACE SLC scenarios (Low, Intermediate, High) derived from global and local effects as per USACE guidelines.

The Sea Level Tracker calculates SLC scenarios using historical MSL data represented by 19-year or 5-year midpoint moving averages. It was used to evaluate the NOAA Springmaid Pier gauge data, determining a regional SLC rate of 0.0133 ft/yr, adjusted for vertical land motion, sourced from Technical Report NOS CO-OPS 065 (Zervas et al., 2013). This rate was adopted as the Low USACE estimated SLC rate.

For the period 2035 to 2085, the study projected a sea level increase of 0.665 ft based on the regional rate. Figure 5 3 from the Tracker tool illustrates trends from 1992 to 2024, showing the 5-year and 19-year moving averages. The 19-year average aligns below the Low SLC curve, while the 5-year average trends above the Intermediate curve, both indicating upward slopes. Overall, the study leveraged the Sea Level Tracker to analyze current and projected rates of SLC, considering both historical data and USACE scenarios to assess future trends in sea level rise for the Waccamaw River basin study. The SLC analysis is presented in Appendix A2.

The study conducted a sensitivity analysis to assess the impact of sea level change (SLC) on hazard levels for the Waccamaw River project. This approach aimed to evaluate the correlation between SLC and increases in water levels without needing to model multiple SLC scenarios for each storm event, which would be computationally intensive and time-consuming. Instead, hindcasts were performed with and without SLC to estimate the effect on total water surface elevation.

In collaboration with the US Army Corps of Engineers (USACE) Climate Preparedness and Resilience Community of Practice, simulations were conducted using HEC-RAS. SLC was modeled based on the USACE Intermediate scenario projecting a 1.32 ft increase in NAVD88 by the year 2085. Each simulation maintained consistent upstream boundary conditions at a 1% Annual Exceedance Probability (AEP), while downstream conditions varied between scenarios: no SLC, Intermediate year 2085 SLC, and Intermediate year 2085 SLC with highest astronomical tides.

Cross-section plots were generated to visualize maximum water surface elevations for each scenario at various locations (as depicted in Figure 74-82). This methodology allowed for a comprehensive analysis of how projected SLC could influence flood hazard levels along the Waccamaw River, aiding in resilience planning and infrastructure design considerations.

Table 27. Nonlinear SWL Residuals from Storm Surge

Location	Annual Exceedance Frequency [Average Nonlinear SWL Residual in feet per foot SLC]						
	50%	20%	10%	5%	2%	1%	0.20%
Longs	0.18	0.13	0.07	0.06	0.06	0.07	0.07
Conway	-0.03	-0.02	-0.01	-0.01	-0.01	0	0
Socastee	0.09	0.11	0.11	0.11	0.11	0.12	0.16
Bucksport	-0.04	-0.03	-0.02	-0.02	-0.01	-0.01	0
HEC-RAS Boundary	-0.01	0	0	-0.01	-0.02	-0.04	-0.05

The 2015 USACE Climate Change Adaptation Plan references ETL 1100-2-1 for guidance on how to plan and implement adaptation to changing sea level. Because focus areas in this study are far enough inland such that minimal effects of SLC are realized, future sea levels will thus have minimal impact on the adaptation plan.

6.0 Existing Conditions Model and Results

6.1 Existing Model Description

Synthetic event inputs for the HEC-RAS model were extracted from the updated FEMA HEC-HMS model. The extracted information included rainfall depth information (as discussed in the Model Approach and Methodology section) and the computed flow hydrographs. The inflow hydrographs that represent the synthetic events for the Little Pee Dee River, the Pee Dee River, and the Lynches River are shown in Figure 90 through Figure 92.

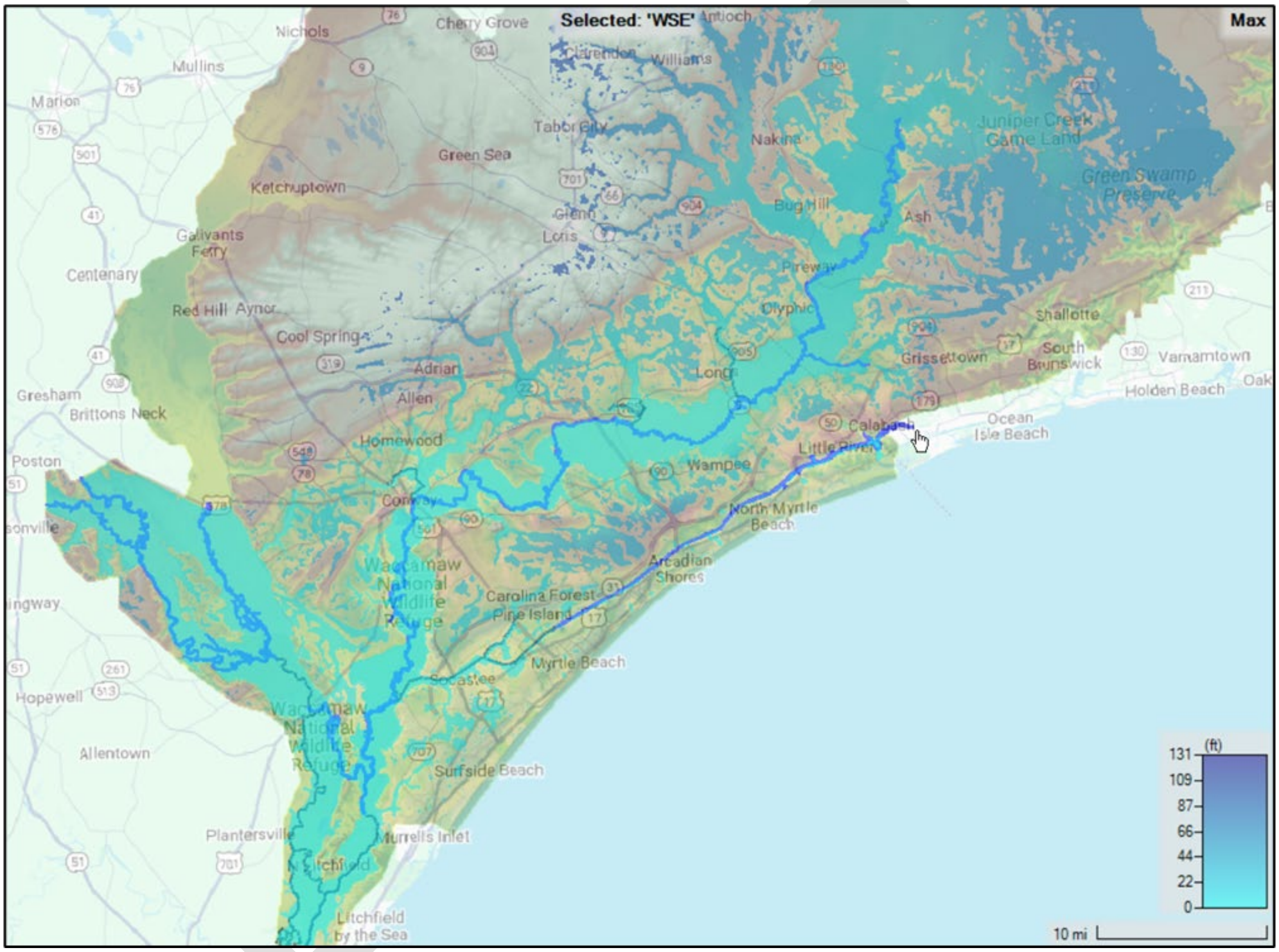


Figure 91. 1% AEP floodplain for entire watershed

The synthetic events were developed using the HEC-HMS model with updated rainfall parameters. Due to lack of documentation, we cannot comment regarding what calibration was performed by the FEMA contractor who developed the HMS model. Development of an updated/calibrated Pee Dee River HMS model would be a relatively significant effort and not within the scope of the project. A hydrograph shape would still need to be estimated and then applied to each value, as the hydrograph shape was the driving factor. Peak flow rate can be addressed, but the shape of the hydrograph was the important factor to calibrate particularly for addressing the secondary peak from the Pee Dee River causing the backwater effects in the Waccamaw River. Figures 92- 94 show the additional peak from the Pee Dee River as the hydrograph shape and peak was the driving factor in development of the synthetic events.

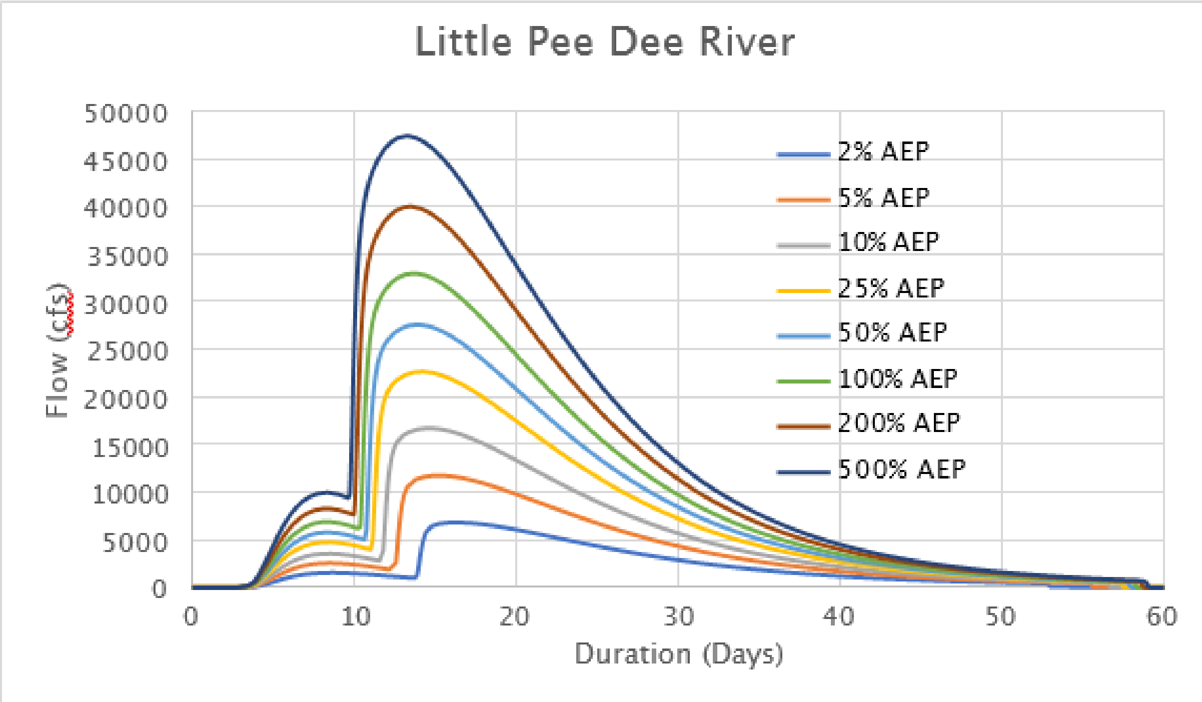


Figure 92. Inflow hydrographs for the Little Pee Dee River for the Synthetic Rainfall events

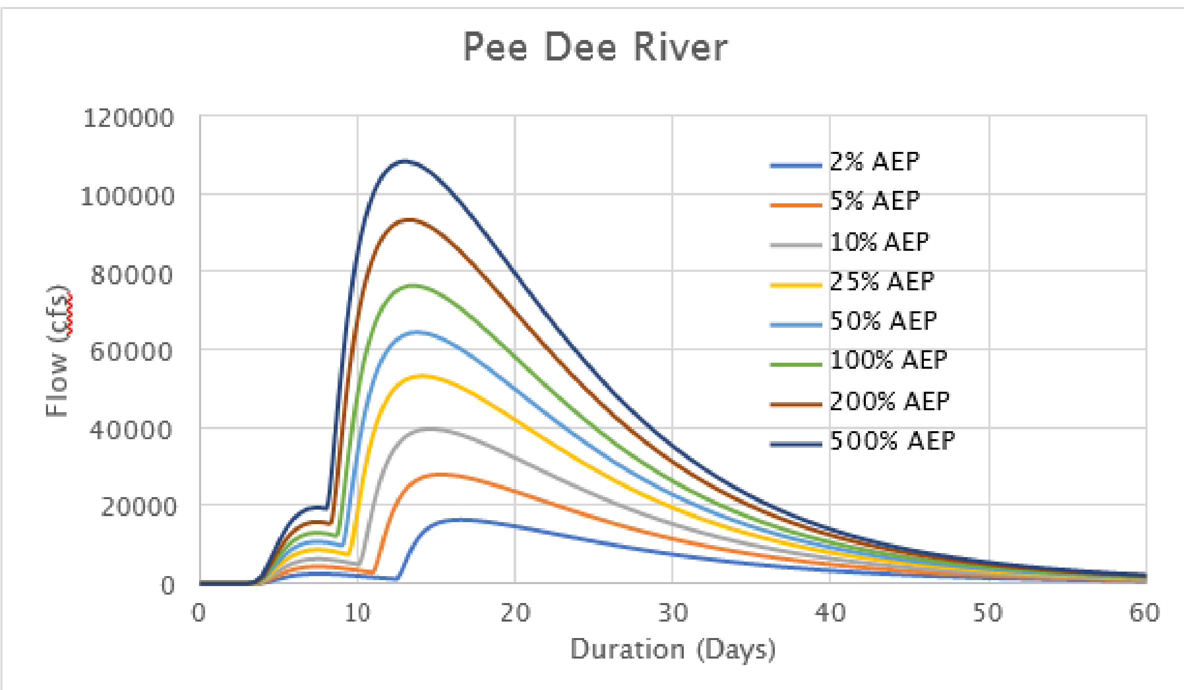


Figure 93. Inflow hydrographs for the Pee Dee River for the Synthetic Rainfall events

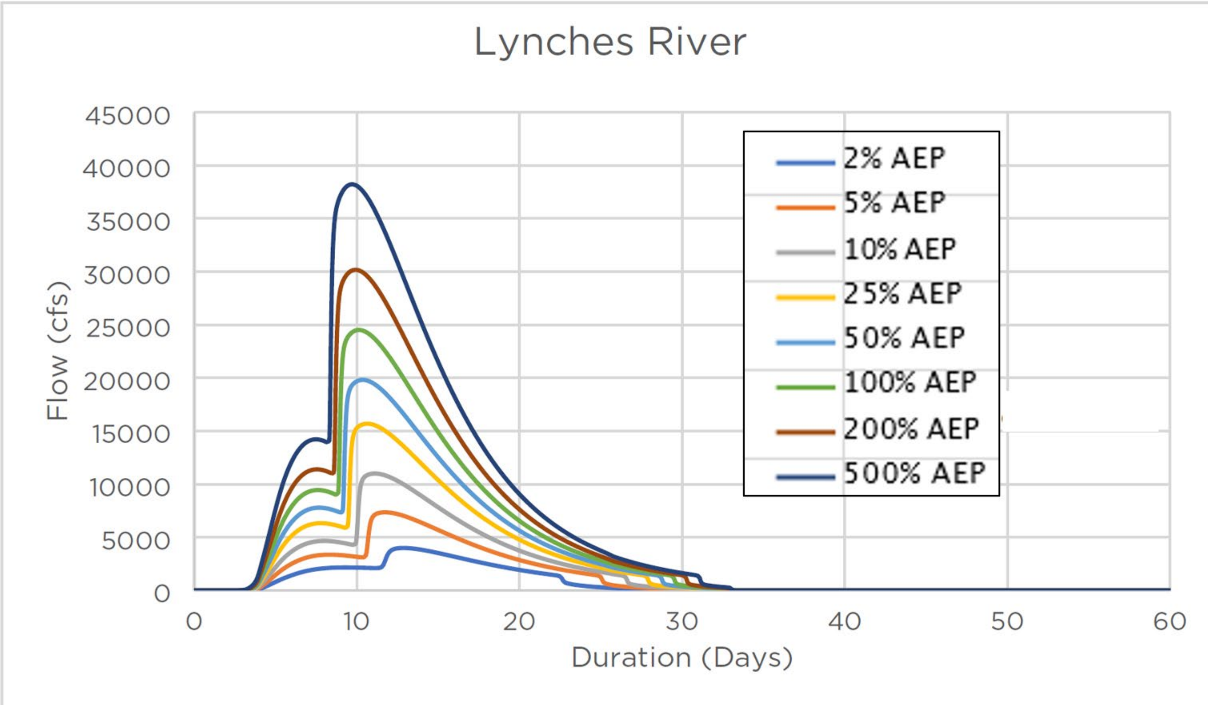


Figure 94. Inflow hydrographs for the Lynches River for the Synthetic Rainfall events

Table 28. List of simulated events

Simulation Description	Purpose
Hot Start – 2-year, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Approximately match the starting conditions of the Florence, Matthew, and Joaquin calibration dataset.
Hurricane Florence	Simulate Hurricane Florence for purposes of model calibration.
Hurricane Matthew	Simulate Hurricane Matthew for purposes of model calibration.
Hurricane Joaquin	Simulate Hurricane Joaquin for purposes of model calibration.
Hot Start – 2-year, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Provide an approximate normal water level condition for the start of the synthetic event simulations.
2-year, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 2-year storm event.
5-year, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 5-year storm event.
10-year, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 10-year storm event.
25-year, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 25-year storm event.
50-year, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 50-year storm event.
100-year, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 100-year storm event.
200-year, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 200-year storm event.
500-year, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 500-year storm event.

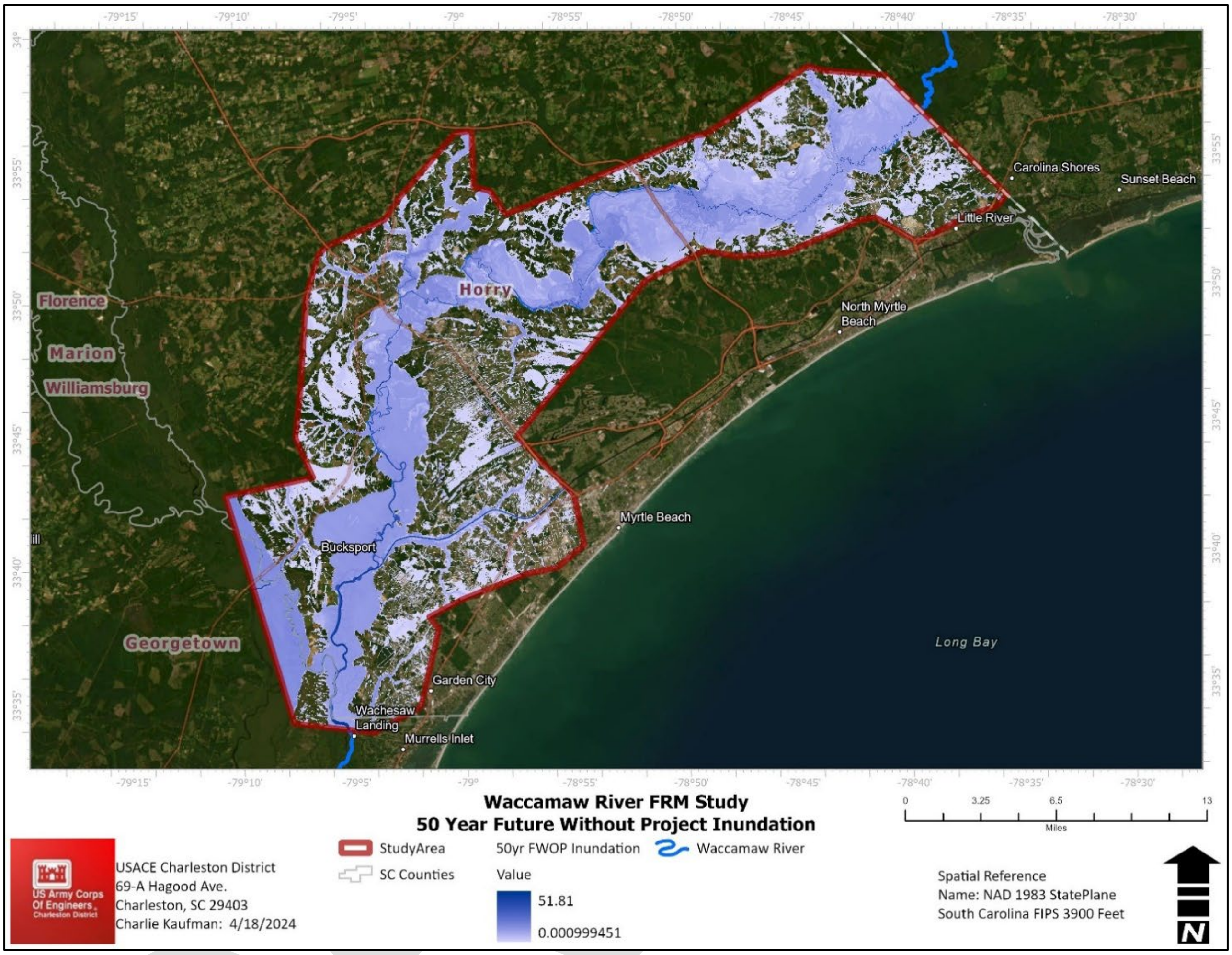


Figure 95. 2% AEP (50-year) 96 hour NOAA Atlas 14 Q4 90% Rainfall event

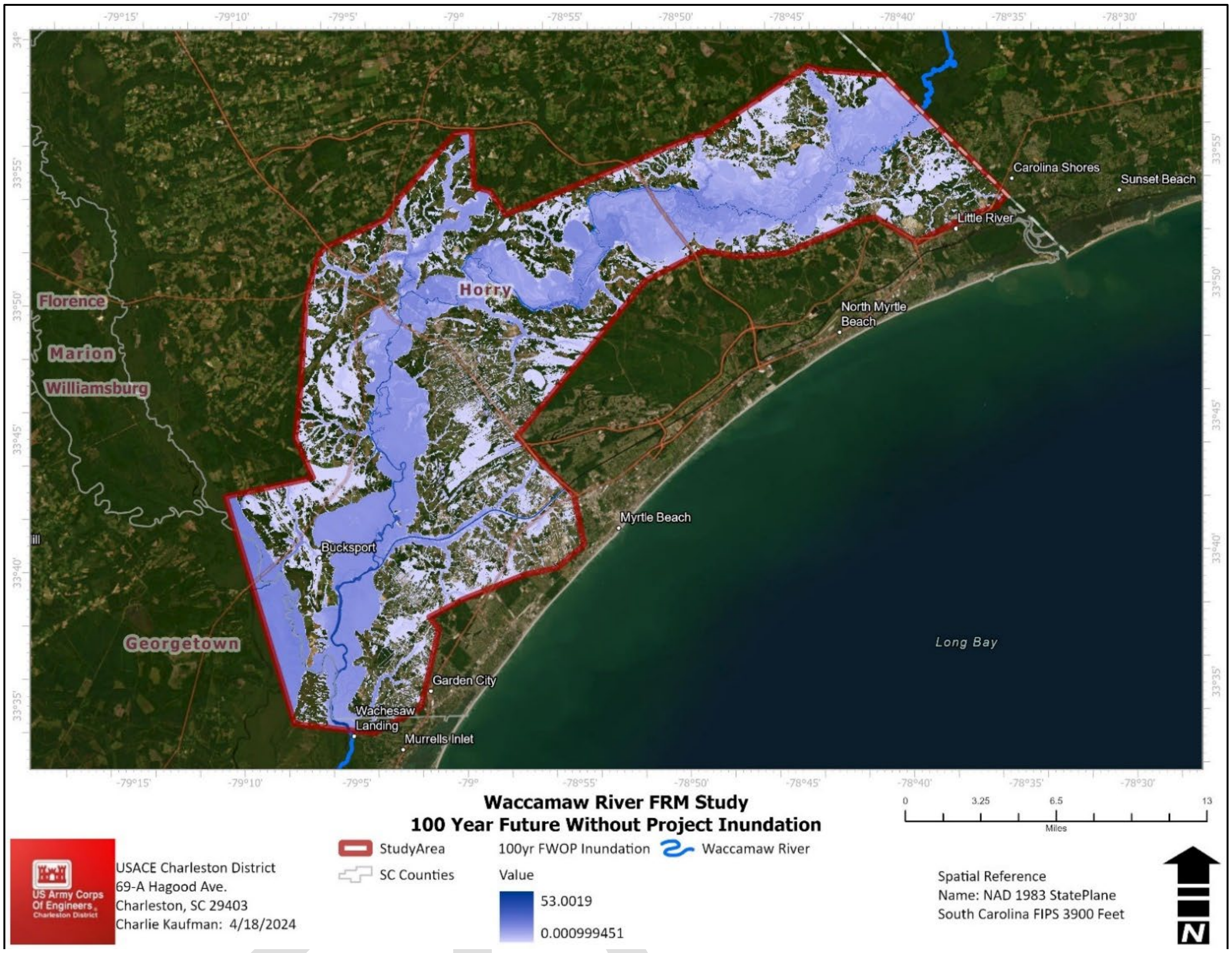


Figure 96. 1% AEP (100 Year) 96 hour NOAA Atlas 14 Q4 90% Rainfall event

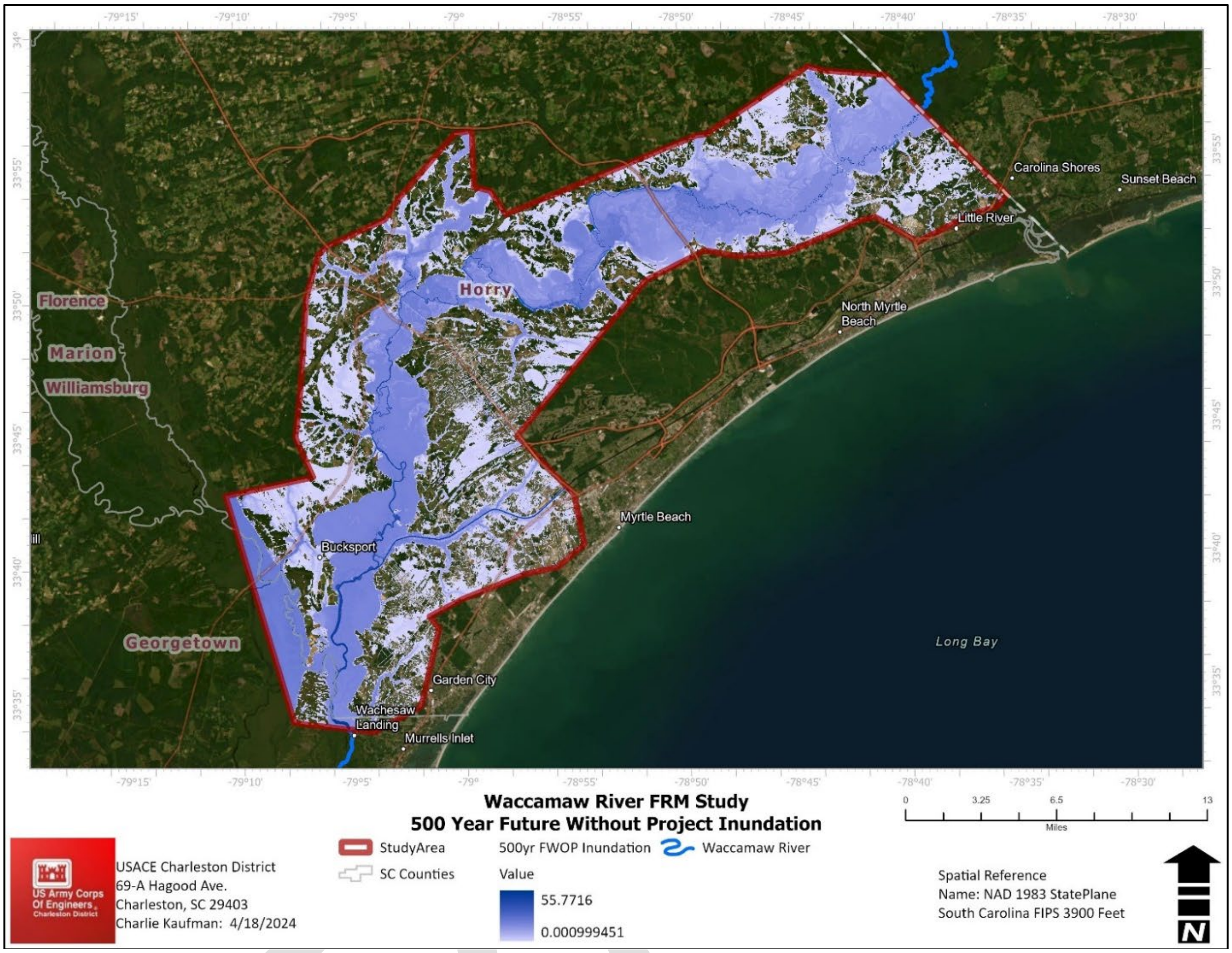


Figure 97. 0.2% AEP (500 year) 96 hour NOAA Atlas 14 Q4 90% Rainfall

6.2 Existing Conditions Simulation Results

For the HEC-RAS model, rain on grid precipitation was put into the model for each event. Appropriate insertion of flow changes was made by applying combined flow records at all headwaters cross sections. Storage areas at the headwaters of tributaries were fed a flowrate for initial model stabilization purposes. Uniform lateral hydrographs were used in subbasin that were not significantly affected by tributary inflows.

Simulation of the 0.5-, 0.2-, 0.1-, 0.04-, 0.02-, 0.01-, 0.005-, and 0.002-AEP events produced profiles representative of the flooding potential for current floodplain conditions. Select existing conditions design event inundations and corresponding water surface profiles for specific study reaches are shown in the following figures within this section. Figure 98 through Figure 107 show the location of the profile highlighted in pink in the map and the streamwise profile comparison for the select synthetic storm events. Overall, the storm events are showing a linear response. The projected Climate to 2085 is also included in these data comparisons.

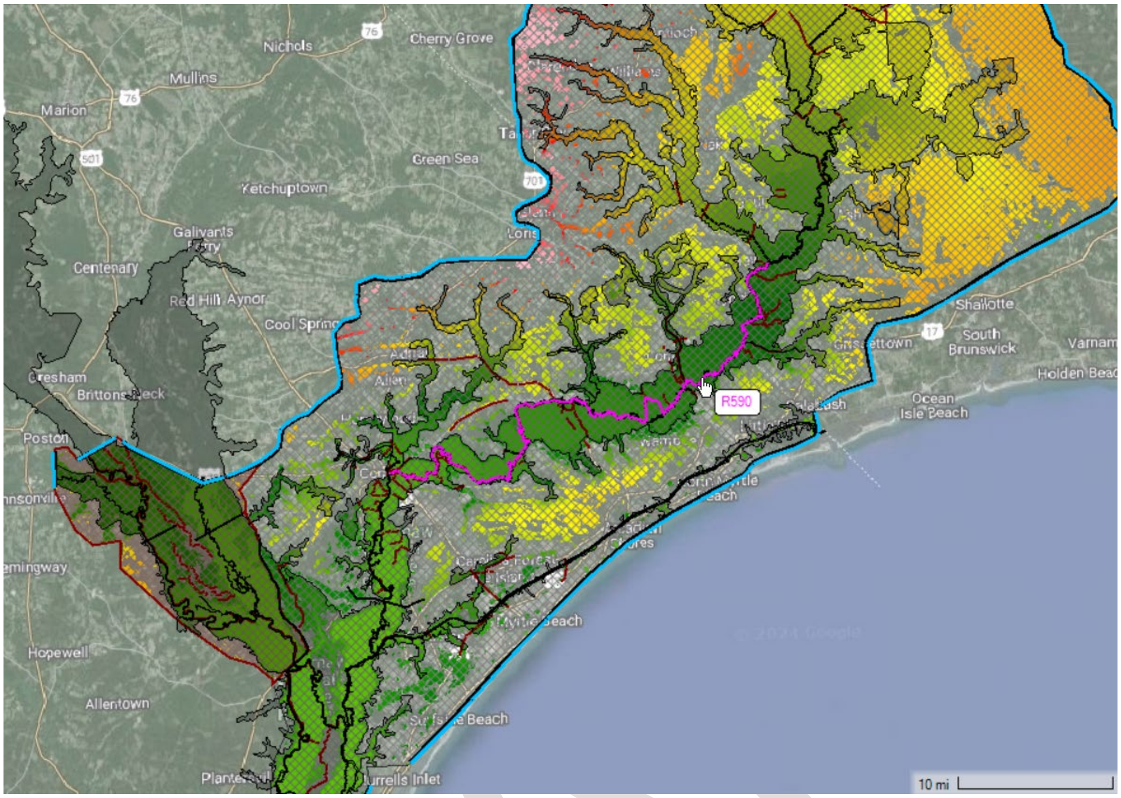


Figure 98. Location (in pink) of the Middle Waccamaw River WSE (max) data comparison (Conway)

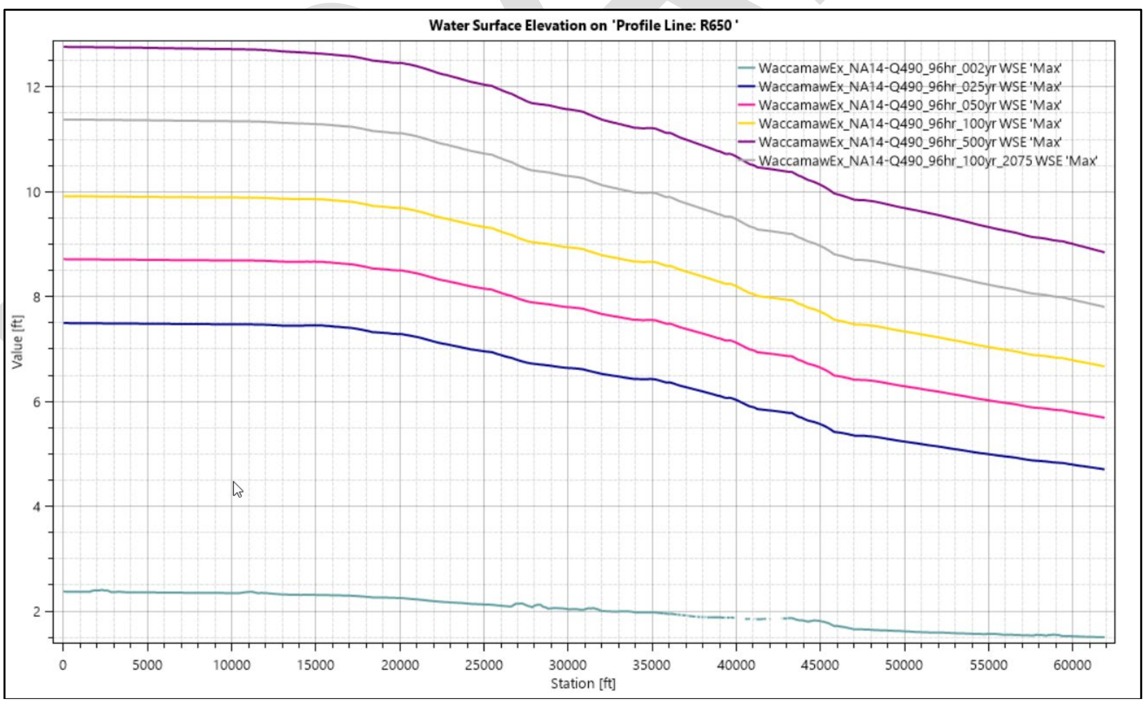


Figure 99. WSE (max) data comparison for middle location (Conway)

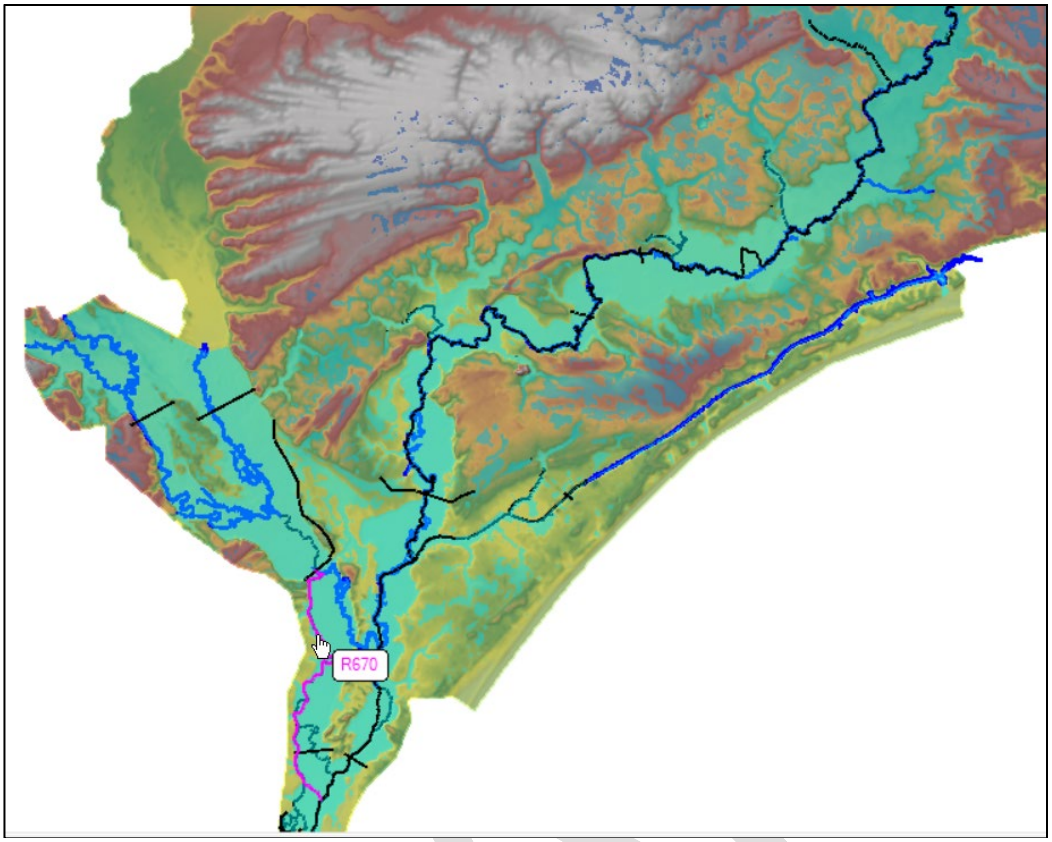


Figure 100. Location (in pink) of the Pee Dee River WSE (max) data comparison (Bucksport)

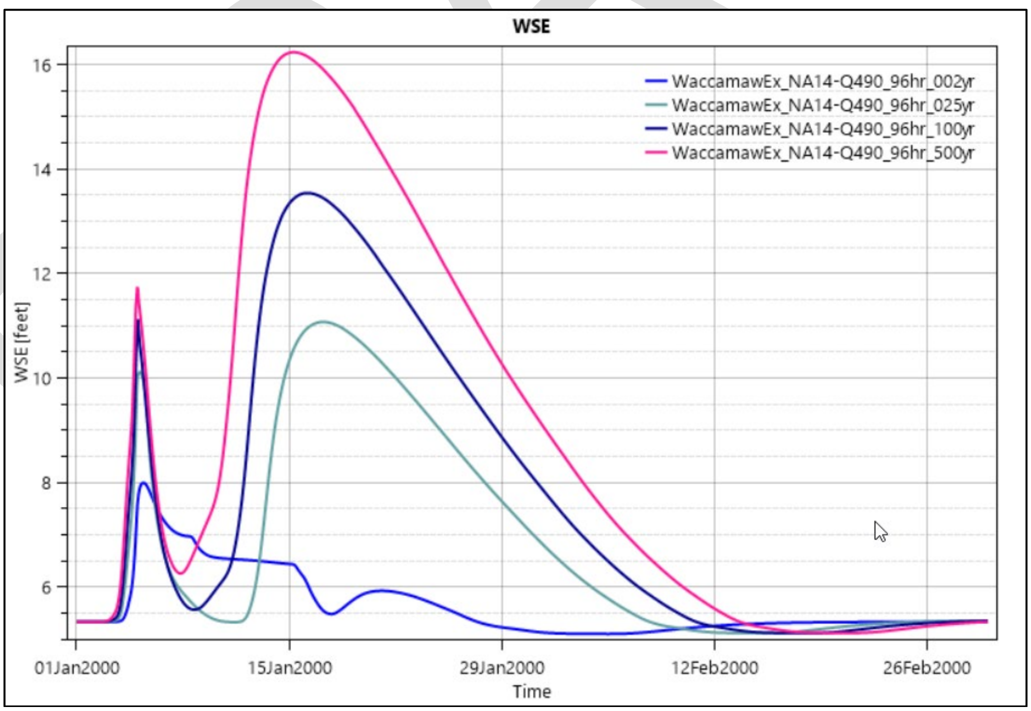


Figure 101. WSE comparison at a data point along the Pee Dee River for different synthetic storm events (Bucksport)

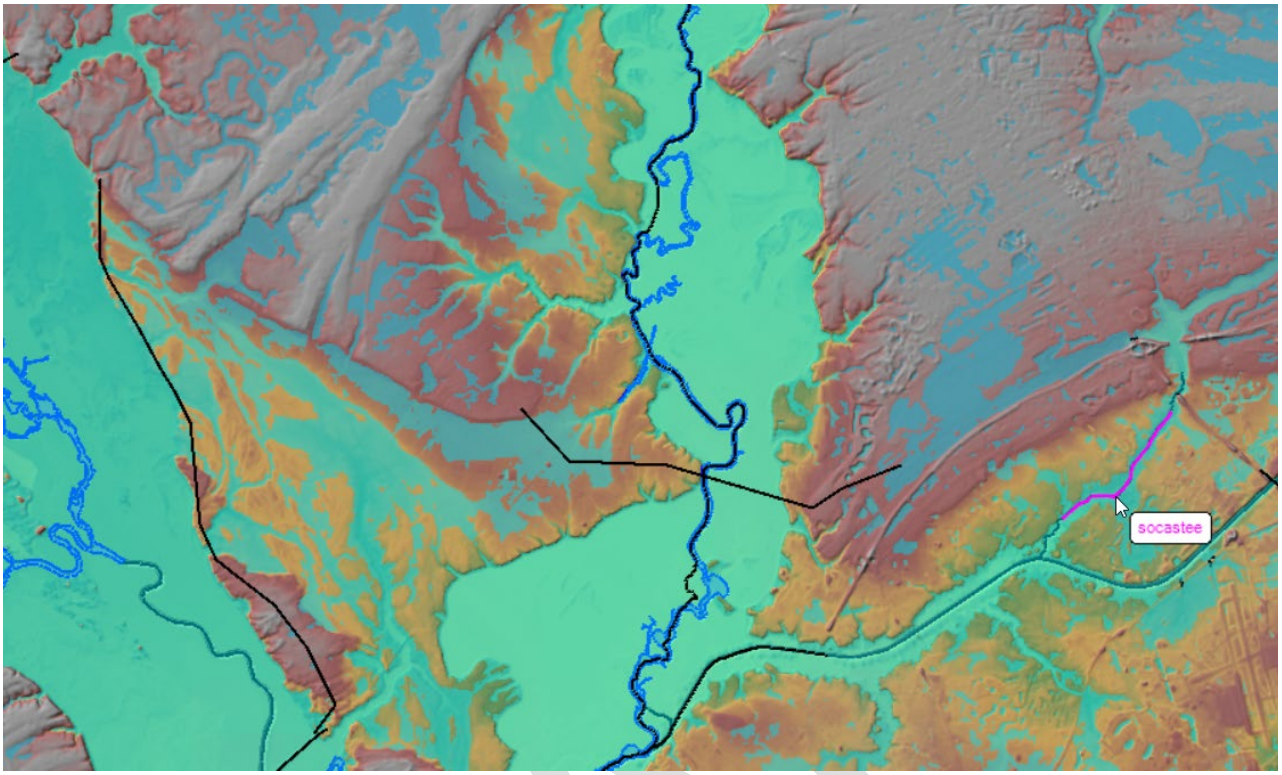


Figure 102. Location (in pink) of the WSE (max) data comparison (Socastee Creek)

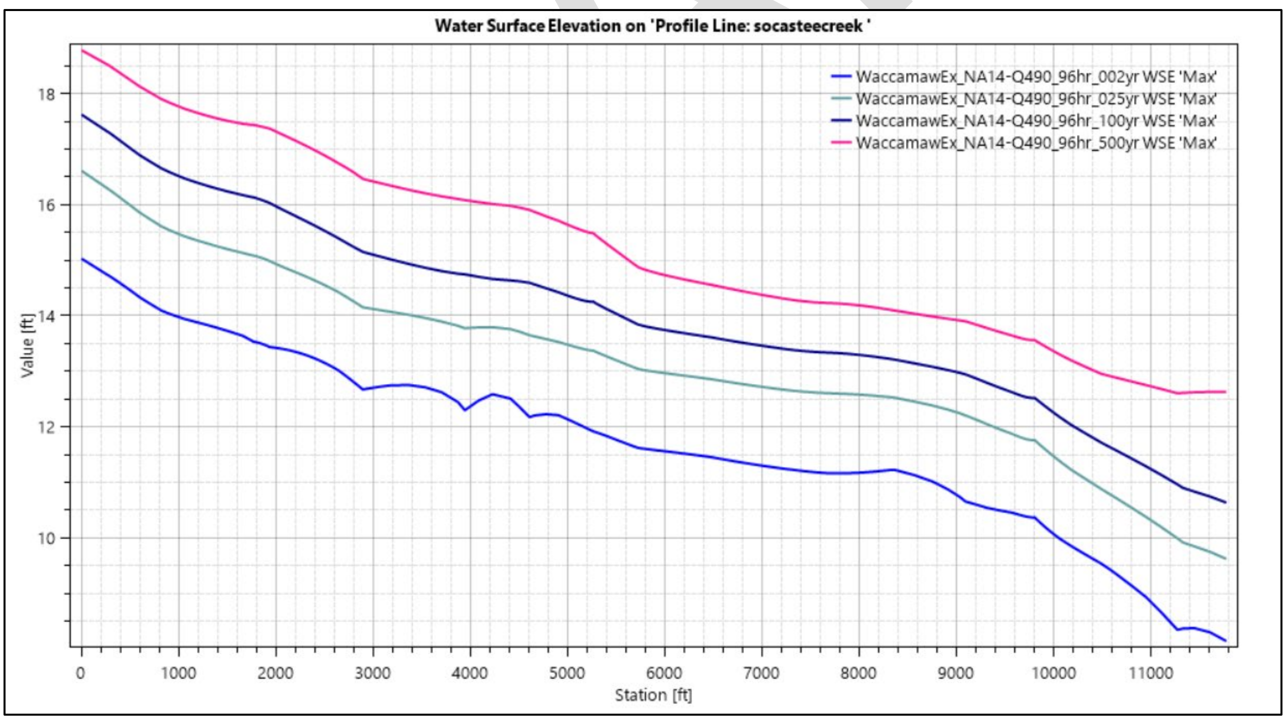


Figure 103. WSE data comparison along Socastee Creek

Figure 105 through Figure 108 show the 50%, 4%, 1% and 0.2% AEP events mapped together for the study areas of interest.

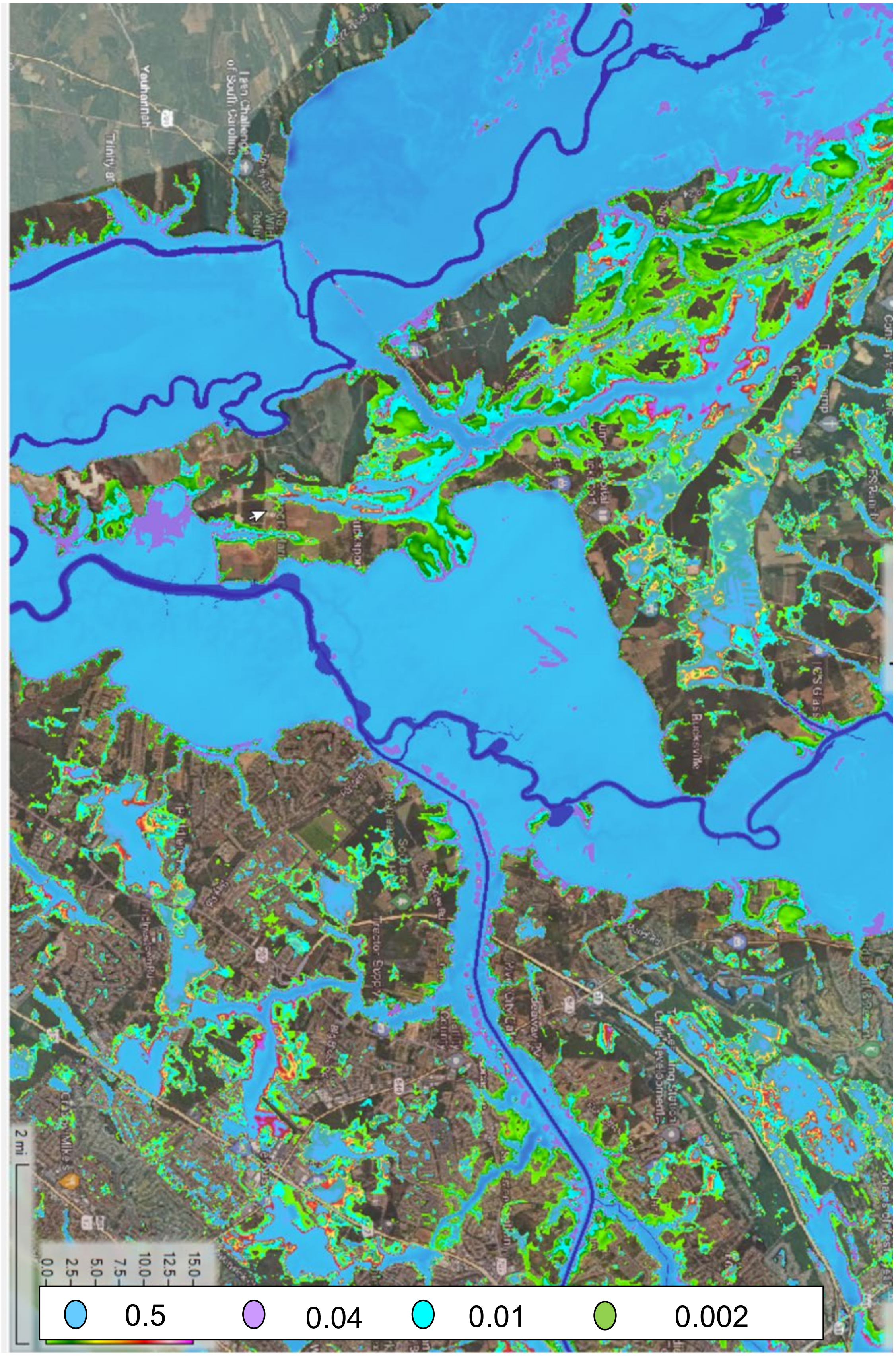


Figure 104. Bucksport Existing Conditions

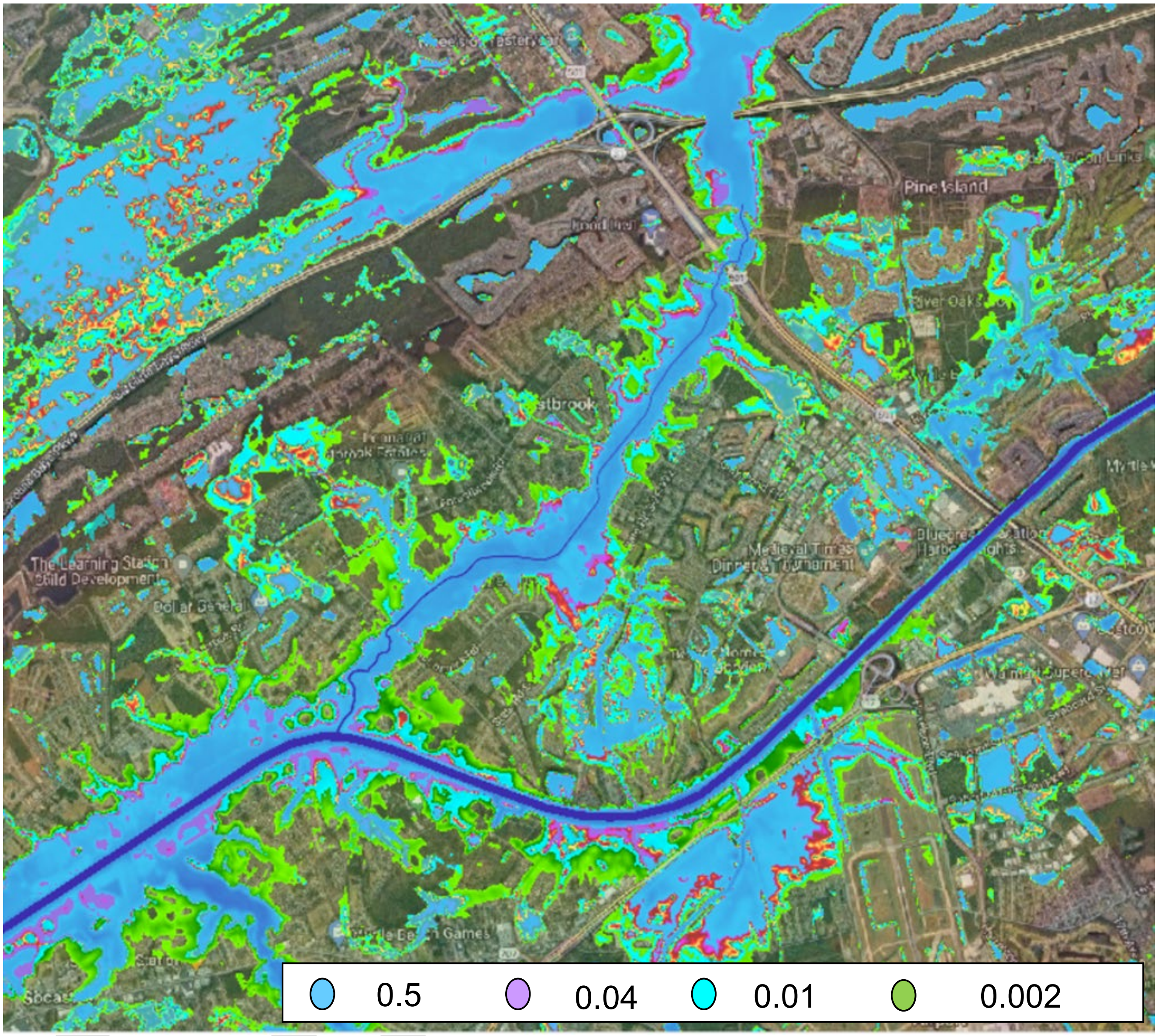


Figure 105. Socastee Existing Conditions

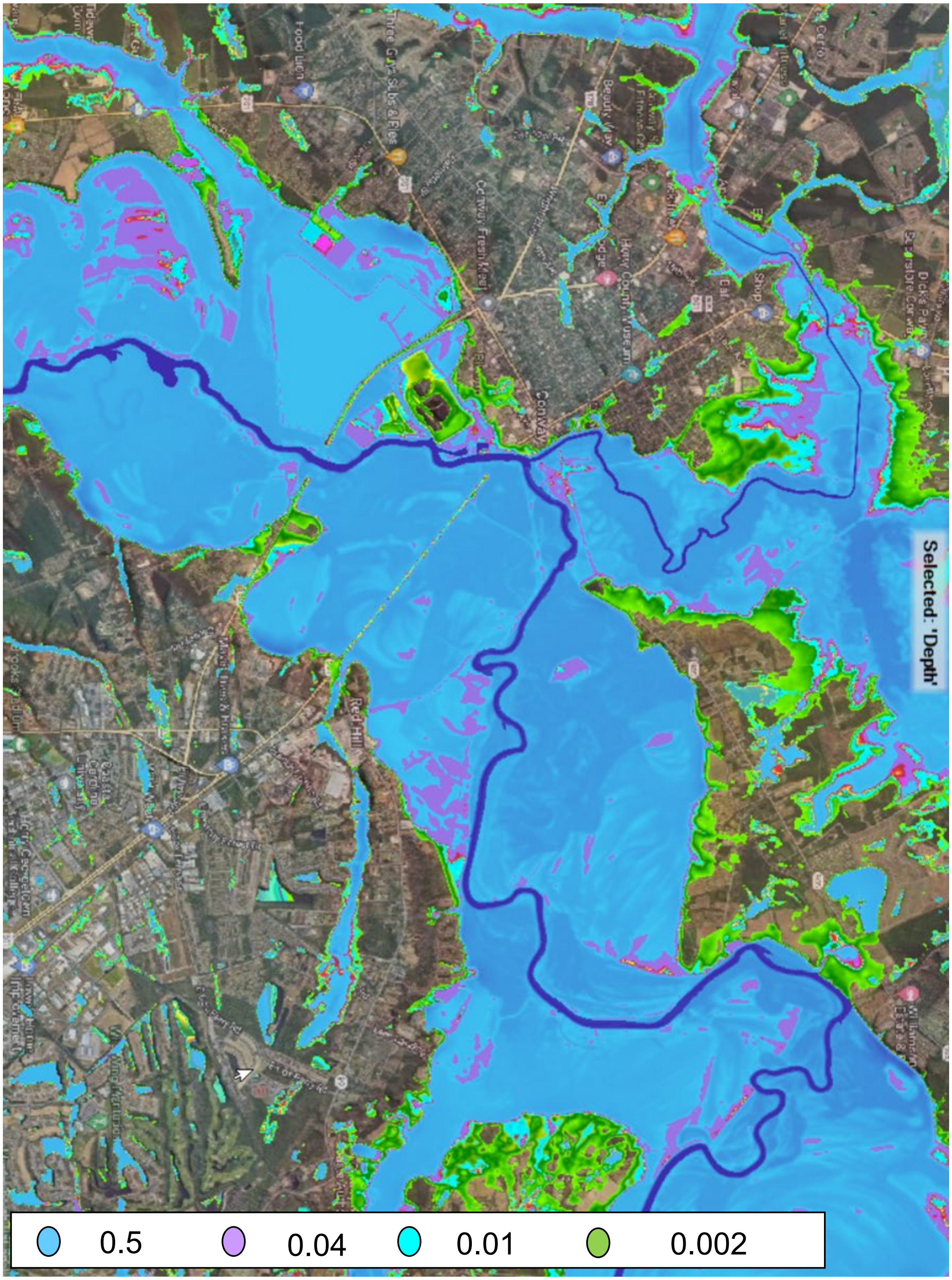


Figure 106. Conway Existing Conditions

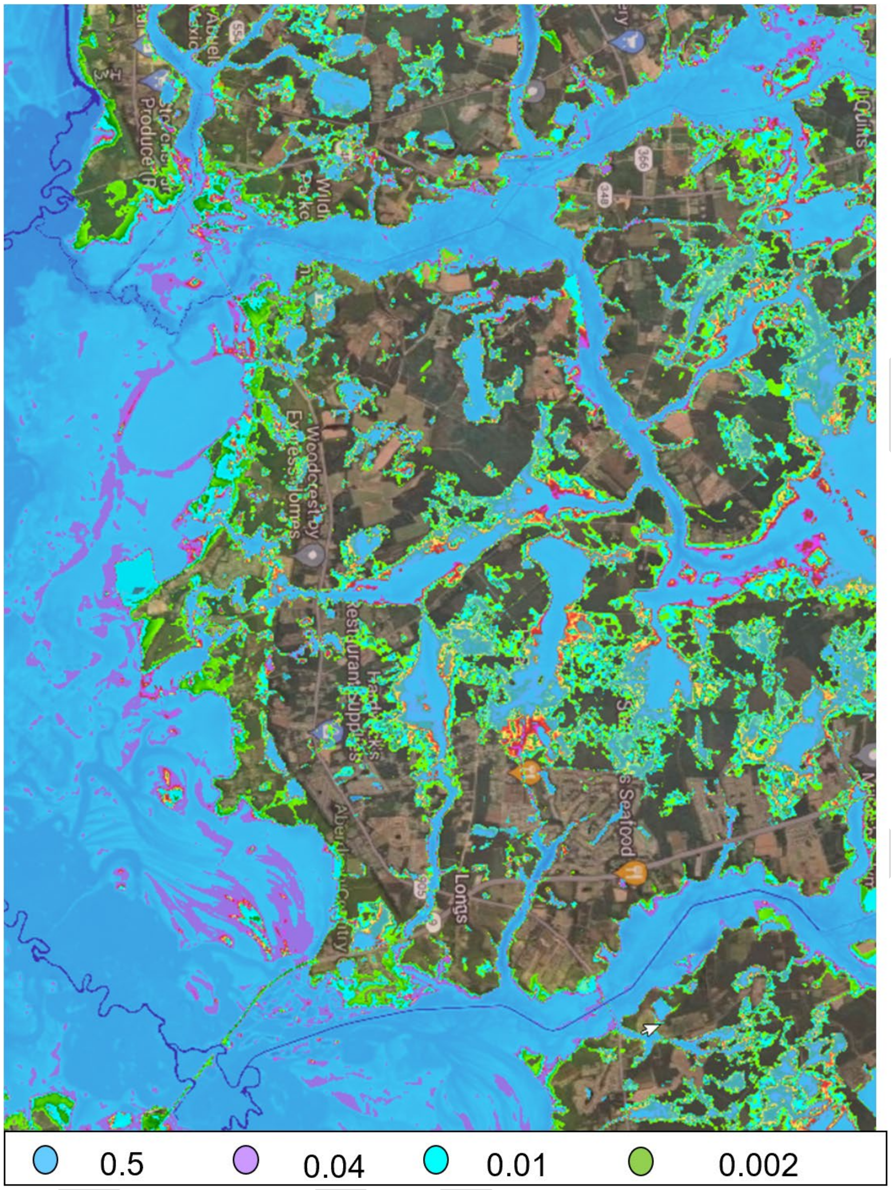


Figure 107. Longs/ Red Bluff Existing Conditions

6.3 Compound Flooding Considerations

Downstream boundary condition data used assumed some dependency in water surface elevations between riverine flows. Fundamentally, the possibility exists for both estuarine and riverine flooding to occur at the same time for the most downstream portions of the Waccamaw River basin study. Extreme winds and elevated tides that originate from coastal storms can propagate across the Pee Dee River and impede the Waccamaw River's ability to efficiently drain. Significant precipitation-based riverine discharge compounds the flooding impacts when also considering storm surge and backwater effects beyond the downstream portion of the Waccamaw River. Compound flooding within a strictly riverine environment, the combination of flow from main stem and tributary watercourses at a confluence, has been commonly documented due to availability of detailed streamflow gage records and commonality between the riverine sources. Through analysis of these data, practical engineering methods have been developed to account for such a flood scenario. The Waccamaw and Pee Dee River Watersheds interaction shares some similarities with a riverine-only scenario, but those engineering methodologies should be used with caution and acknowledgement of uncertainties.

Several sensitivity analyses were conducted as part of this basin-wide study to establish the approximate geographic extents during which a combined riverine/estuary flood event would maximize water surface elevations. It would then be inferred that design flows upstream of this extent would be governed by the riverine-source and downstream of this extent would be governed by the coastal-source. Assumptions of dependency between the riverine and estuary flood sources were also investigated to approximate residual risk. It was determined that tidal and sea level changes effect on the focus area locations were limited. Refer to the Climate and SLC Appendix A-2 for the assessment of the compound flooding considerations that concluded that there is some coastal and tidal impacts to the riverine flow, however not in the regions where damages are occurring and the proposed measures in place. Due to study limitations, these analyses were conducted under existing conditions and may not fully capture the effects of compound flooding under future conditions.

7.0 Future Without Project Conditions

7.1 Background

Future hydrologic conditions in the Waccamaw River basin will have an impact on the problems and opportunities identified. As land use conditions change, they influence the hydrologic conditions which can lead to increased flood damages to existing economic development in the floodplain. Growth in population and other economic development will create additional pressure to develop within less vulnerable, flood free areas. Increases in runoff volume and decreases in flood wave timing are directly attributed to urbanization in which impervious area prevent natural floodplain storage, intensify flood peaks, and alter flow paths.

For future conditions in the Waccamaw River basin, locally provided future land use data for Horry County areas were analyzed for estimating changes in impervious surface area for the applicable subbasins. This analysis showed a nominal change in land cover related to development in the area. Future without project conditions for the basin were developed by modeling a road raising in Bucksport and benching in Socastee Creek that are going to be completed before the start of the project.

7.2 FWOP Structural Measure Considerations

The following two projects were included in the modeling for FWOP conditions because they are projects carried forward by Horry County and that have an impact on the flow conditions. The two projects are the Big Bull Road Raising in Bucksport and Benching along Socastee Creek in Socastee. The following are the project descriptions and FWOP results.

7.2.1 Big Bull Landing Road Raising

The proposed work consists of filling and raising a 2,500 LF portion of Big Bull Landing Road (a County maintained dirt road) to an approximate elevation of 15 feet (NAVD88) as well as the installation of three (3) additional 36-inch reinforced concrete pipes (RCP) to improve drainage. In addition, the proposed Bucksport Road Bypass Channel will include a relief ditch and 48-inch reinforced concrete pipe (RCP) that will outfall to the Waccamaw River. The proposed relief system will be located about 2,200 feet south of the Big Bull Landing Road. These proposed improvements will result in the deposition of fill material, clearing and minor excavation activities within wetland resources. The proposed construction activities associated with the raising of Big Bull Landing Road will consist of the deposition of 0.085 acres of fill within wetlands associated with an unnamed tributary to Cowford Swamp. The construction activities associated with the proposed Bucksport Road Bypass Channel will result in 0.042 acres of clearing and 0.018 acres of wetland clearing/excavation.

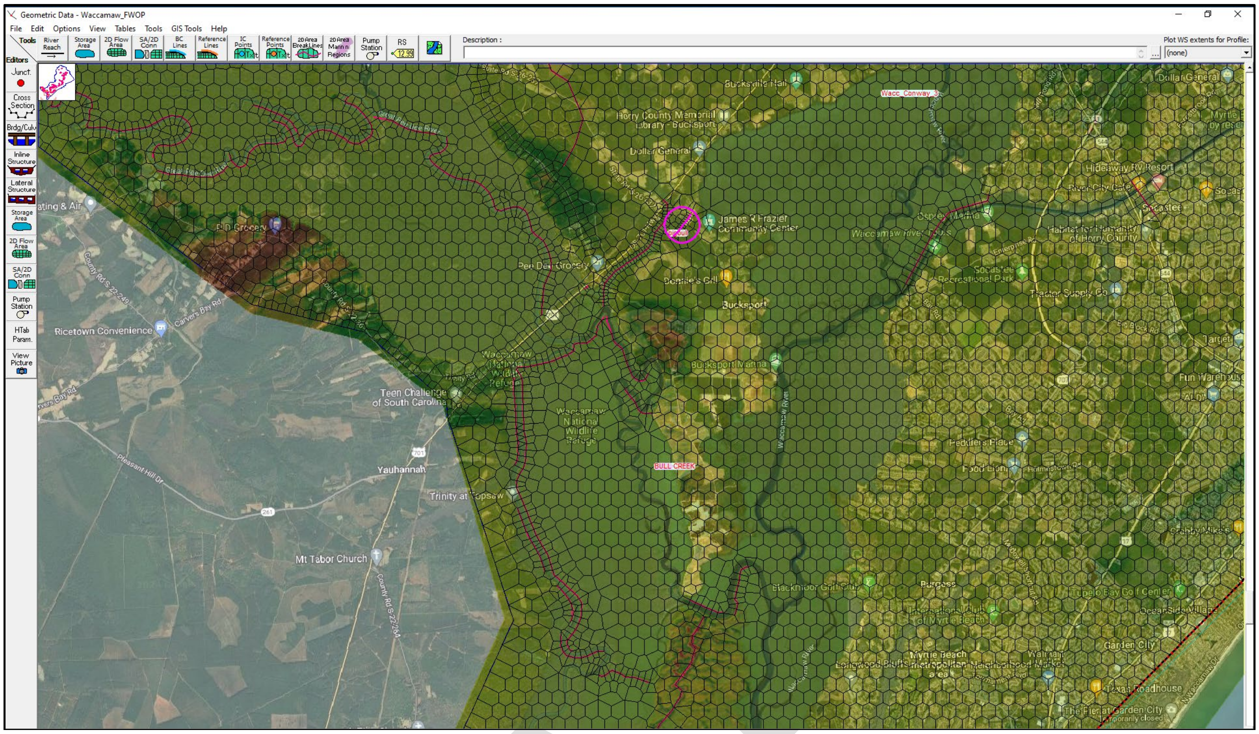


Figure 108. HEC-RAS model geometric data for the implementation of Big Bull Landing Road Raising

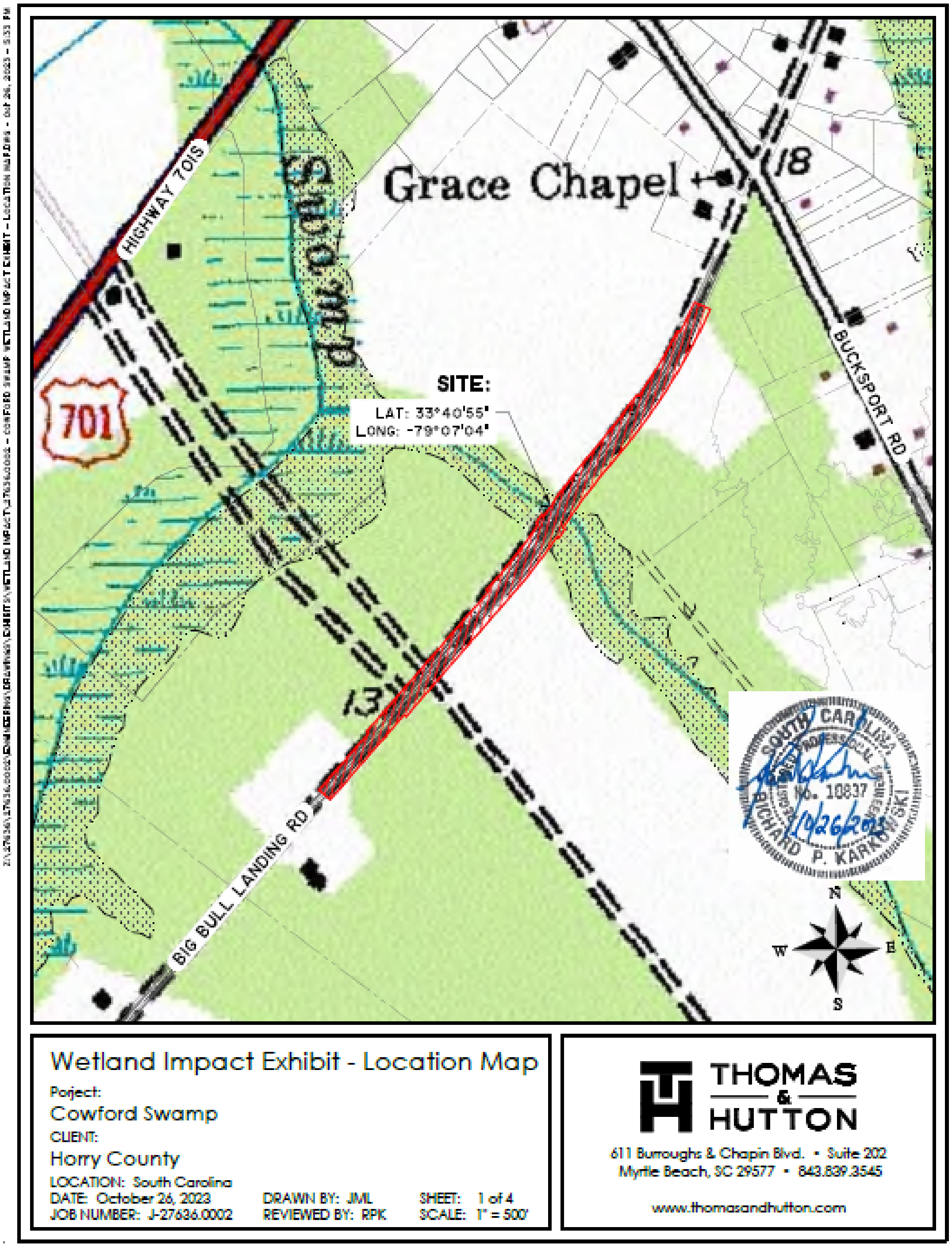


Figure 109. Location of Big Bull Landing Road Raising

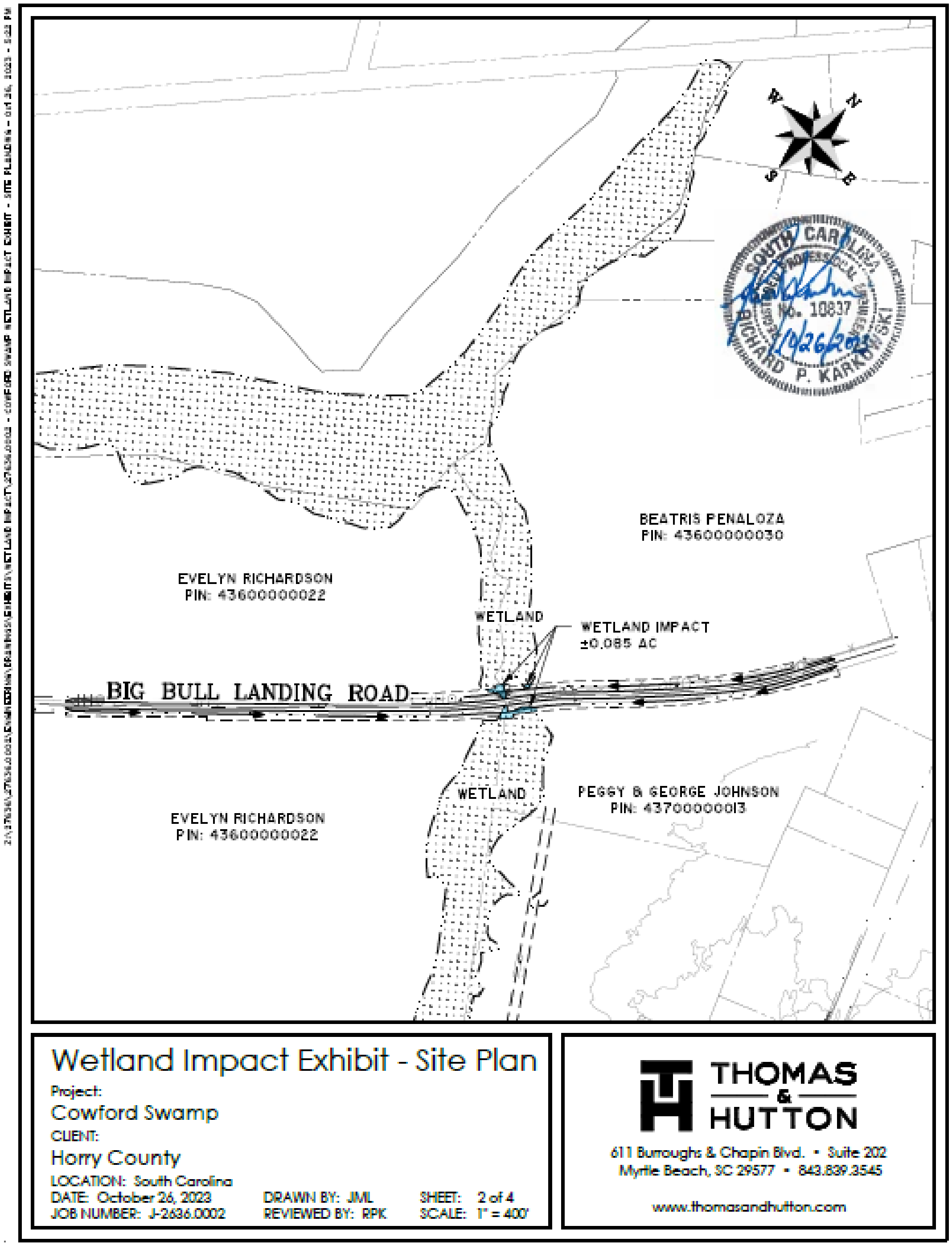


Figure 110. Wetland Impact Exhibit Big Bull Landing Road Raising

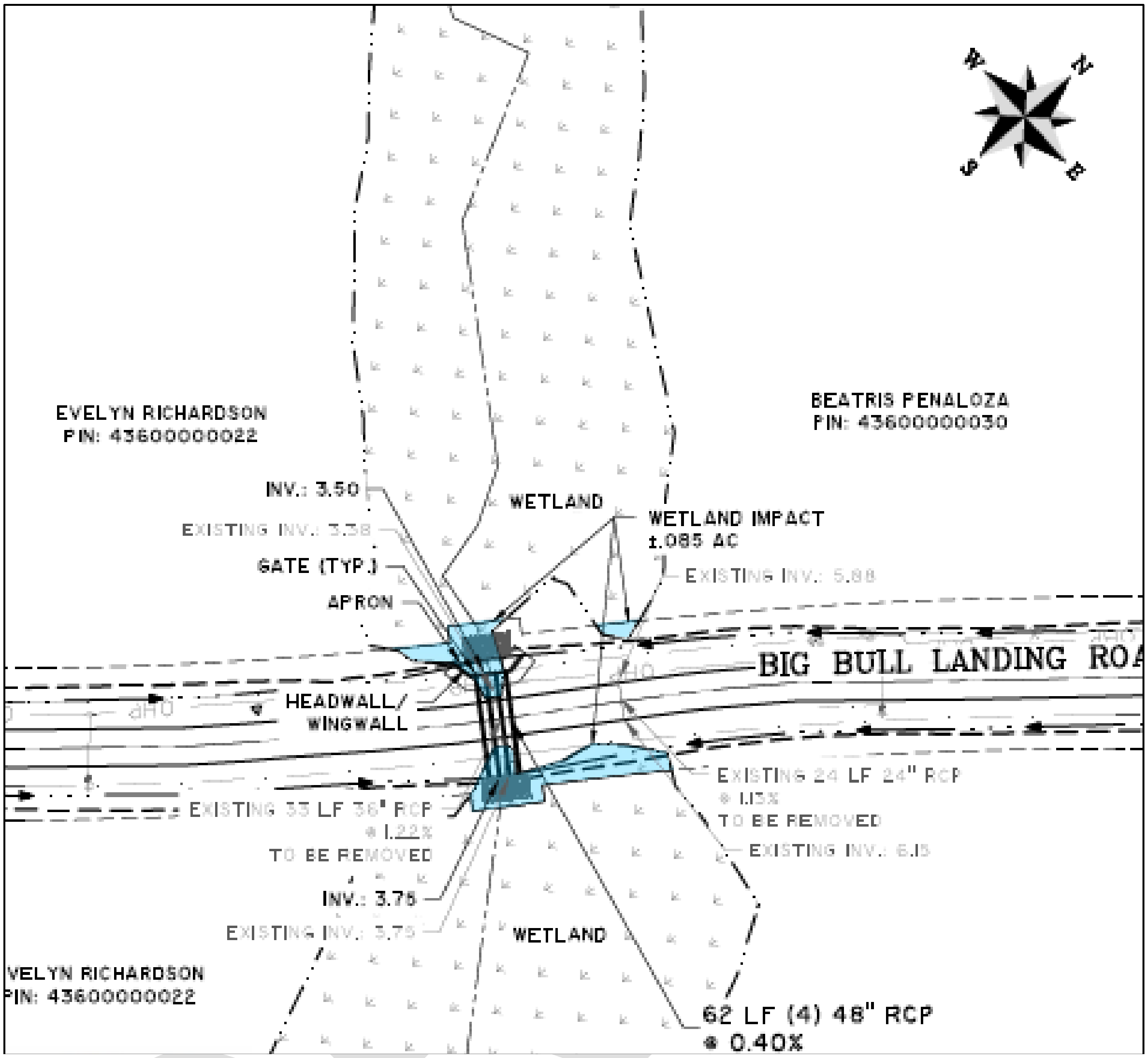


Figure 111. Site Plans of culverts and wetlands for Big Bull Landing Road Raising

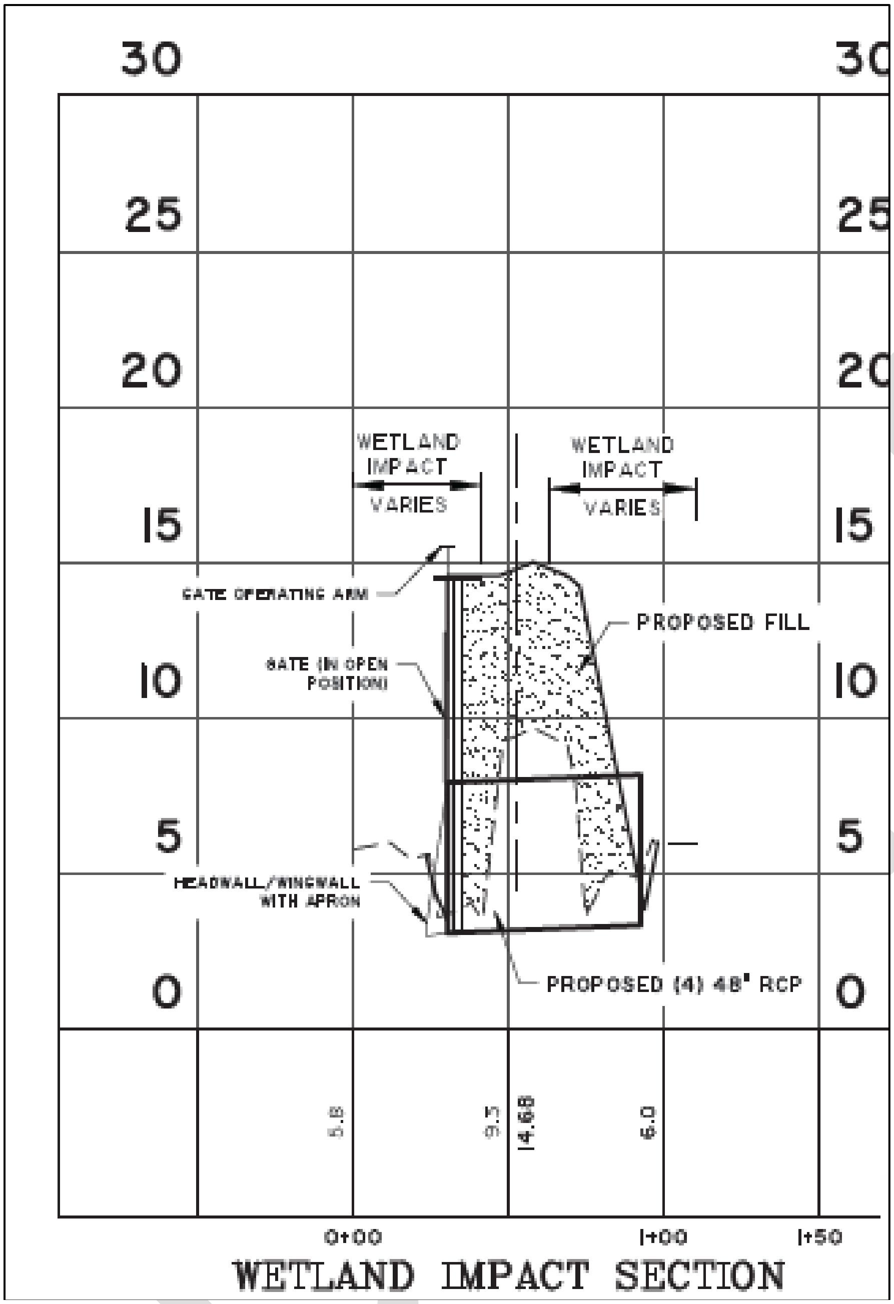


Figure 112. Proposed Wetland Impact Section for Big Bull Landing Road Raising

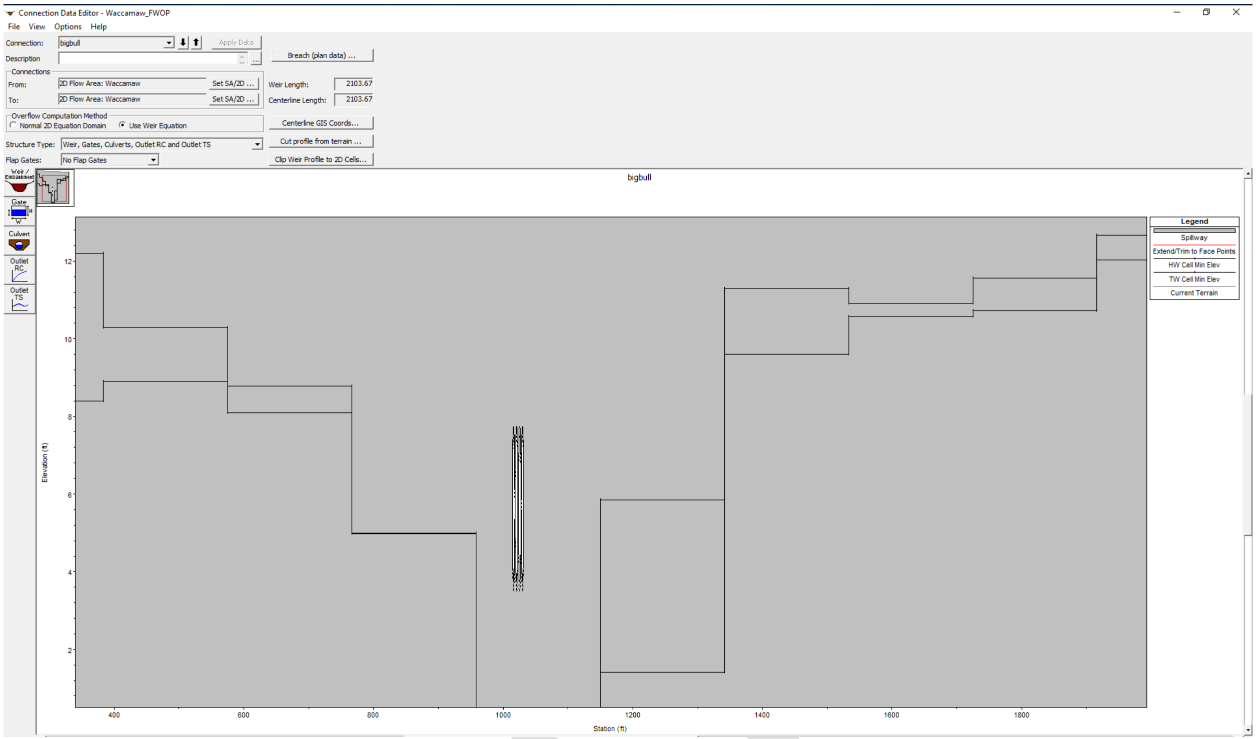


Figure 113. Connection Data Editor in HEC RAS Big Bull Landing Road Raising

The road raising was implemented into the HEC-RAS model using the Terrain Modification. The basis for any accurate river hydraulics model is a good representation of ground surface elevations for the river and floodplain areas. A good terrain model accurately describes the elevations of the river channel and floodplain by incorporating important features that control the movement of water, such as the channel bottom and channel banks, and high ground such as roadways and levees. If the initial terrain model insufficiently represents the ground surface, HEC-RAS provides tools for improving the terrain data directly in RAS Mapper. There are currently two methods for improving channel data in HEC-RAS: (1) using cross sections to create an interpolation surface to add to an existing terrain model; (2) using the vector Terrain Modification tools in RAS Mapper to improve the terrain by adding channel information, adding high ground (such as a road), adding features that impede flow (such as piers), or otherwise modifying the terrain elevations. RAS Mapper supports many different raster formats; however, the Terrain Modification tools work specifically with the RAS Terrain layer to create a compilation of vector additions to the underlying GeoTiff representation of the grounds surface. Since the existing model is a 2-D Unsteady model, the terrain modification is the best option. Terrain layers are very large datasets. Therefore, terrain modifications have been implemented as vector additions to the Terrain layer. These modifications are stored in the terrain layers .hdf file. Further, in a continued attempt to reduce data and to keep the base terrain data unmodified, there is an option to create copy of the Terrain data. The Clone Terrain option was used to not affect the existing model runs.

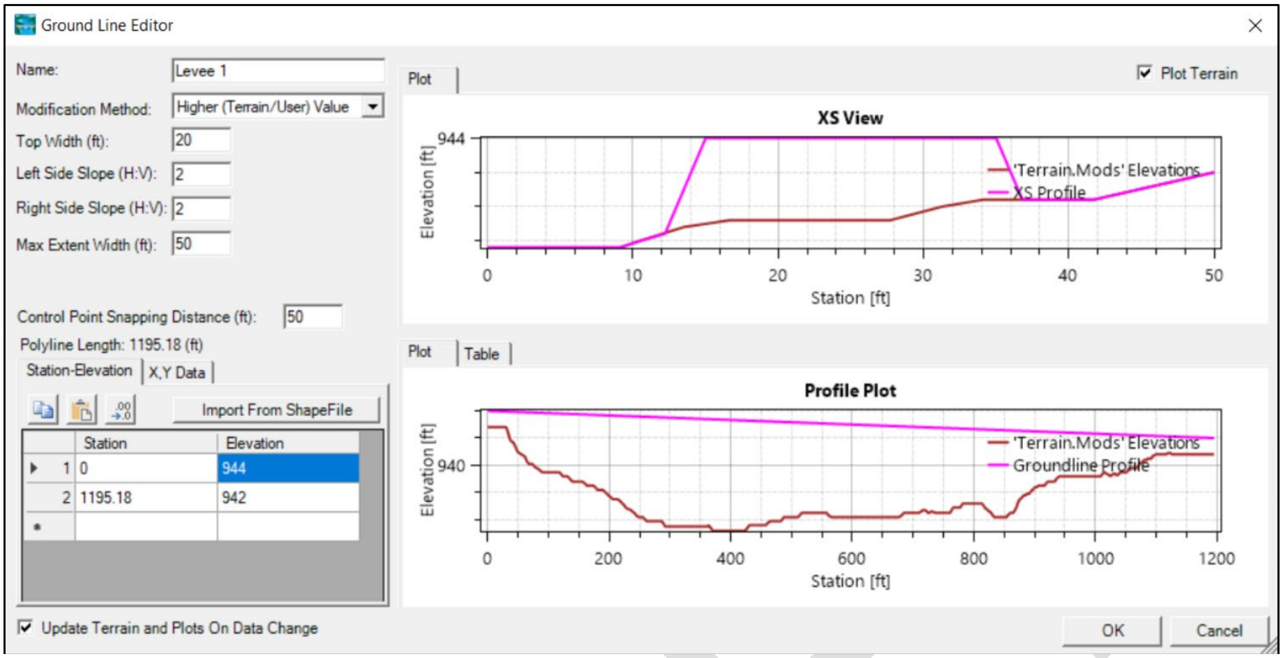


Figure 114. Example of the Terrain ground line editor used to raise Big Bull landing

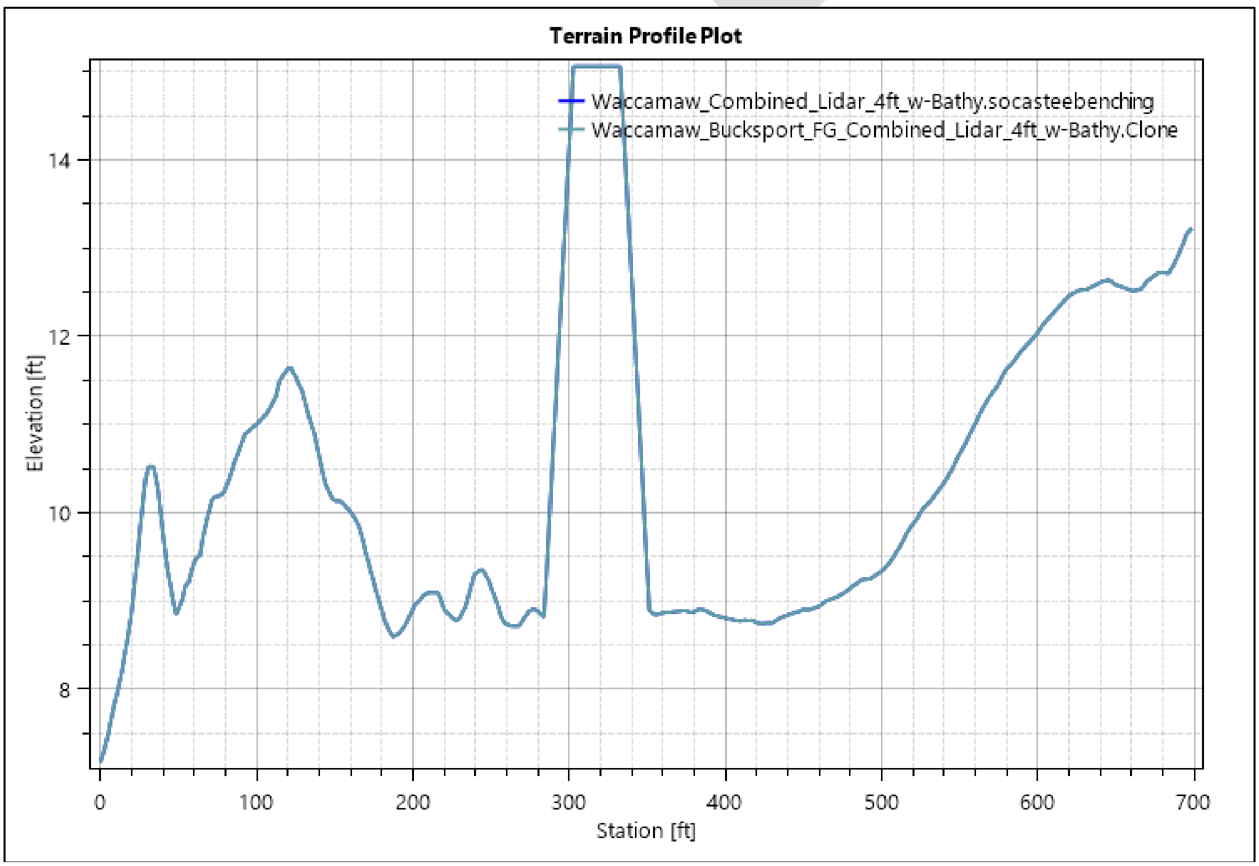


Figure 115. Imposed terrain for Big Bull Landing Road Raising

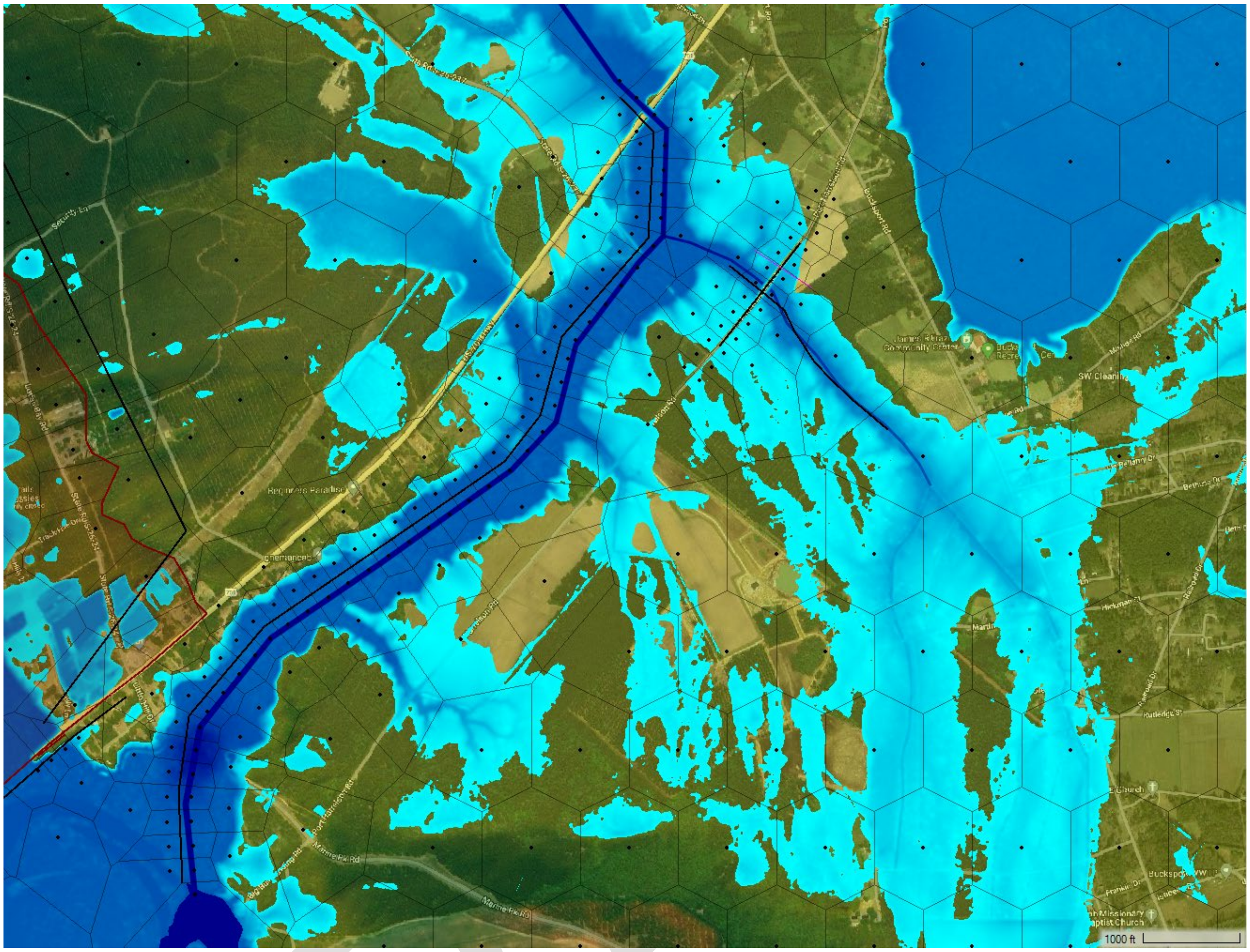


Figure 116. 1% AEP model Run for FWOP with the implementation of Big Bull Landing

7.2.2 Socastee Benching

Benching will be implemented by Horry County to reduce or eliminate repetitive flooding of vulnerable buildings and properties by benching 90 ft average width along 6000LF of the watershed above weir #2. Flood elevations at each cross section for the 100-year storm are provided in the Table 1 below. In addition, the summary table, HEC-RAS model results are conducted for all events. Flood elevation reductions resulting from the benching project are shown in Table 1 below and in the HEC-RAS report in the Attachments. The results show a reduction in flood elevations upstream of Weir 2 (upper weir) and downstream of Weir 2 flood elevations are equal. The purpose of the proposed activities is to increase flood capacity within the Socastee Creek Watershed. The work affecting waters of the United States is part of an overall project known as Socastee Creek Benching Project. The proposed project is located adjacent to Socastee Creek, west of Burcale Road, South of U.S. Highway 501 in the Conway/Socastee Township, Horry County, South Carolina (Latitude: 33.7246 °, Longitude: -78.9482 °). Similar to the terrain modification for the Road Raising in Bucksport, the channel was modified in the terrain modification in RAS Mapper using HEC-RAS. The channel was modified for 6000 LF and 90ft average width on the left bank.

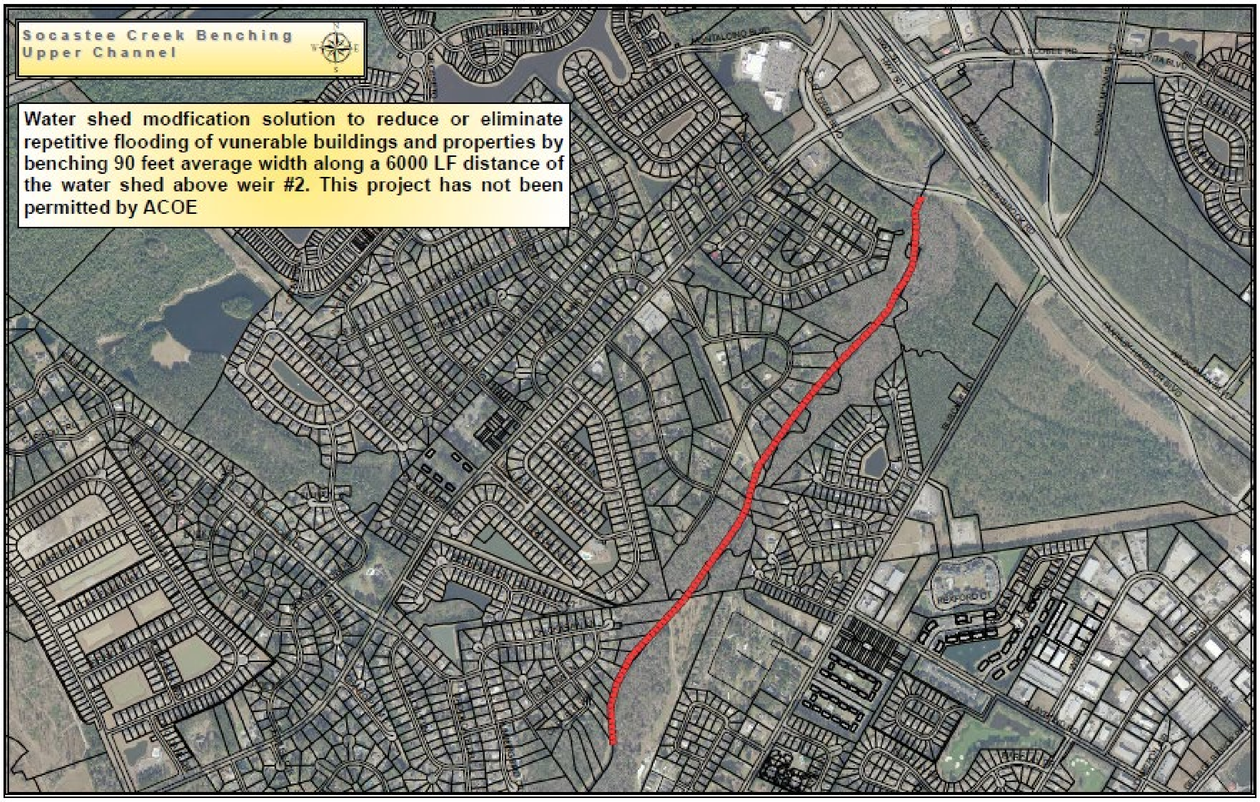


Figure 117. Location of Socastee Benching

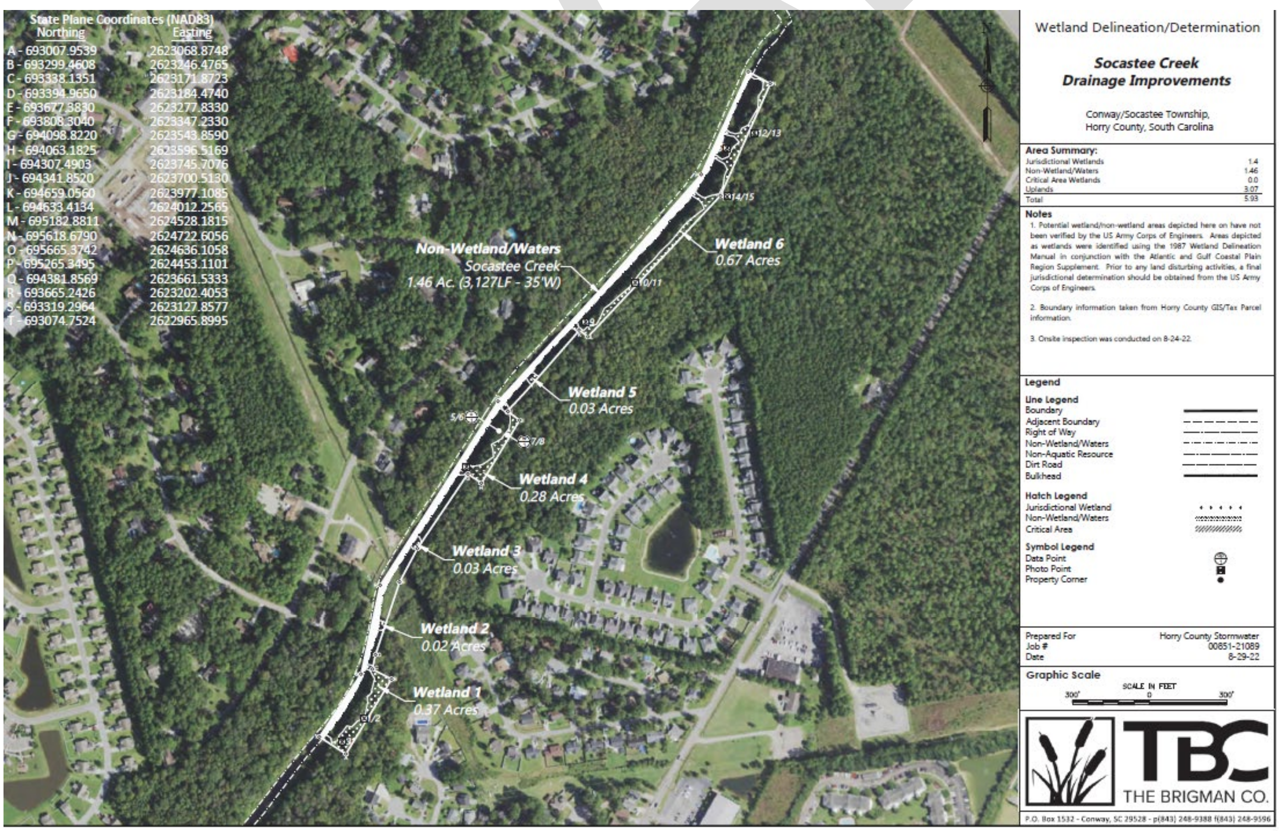


Figure 118. Socastee Benching project extents

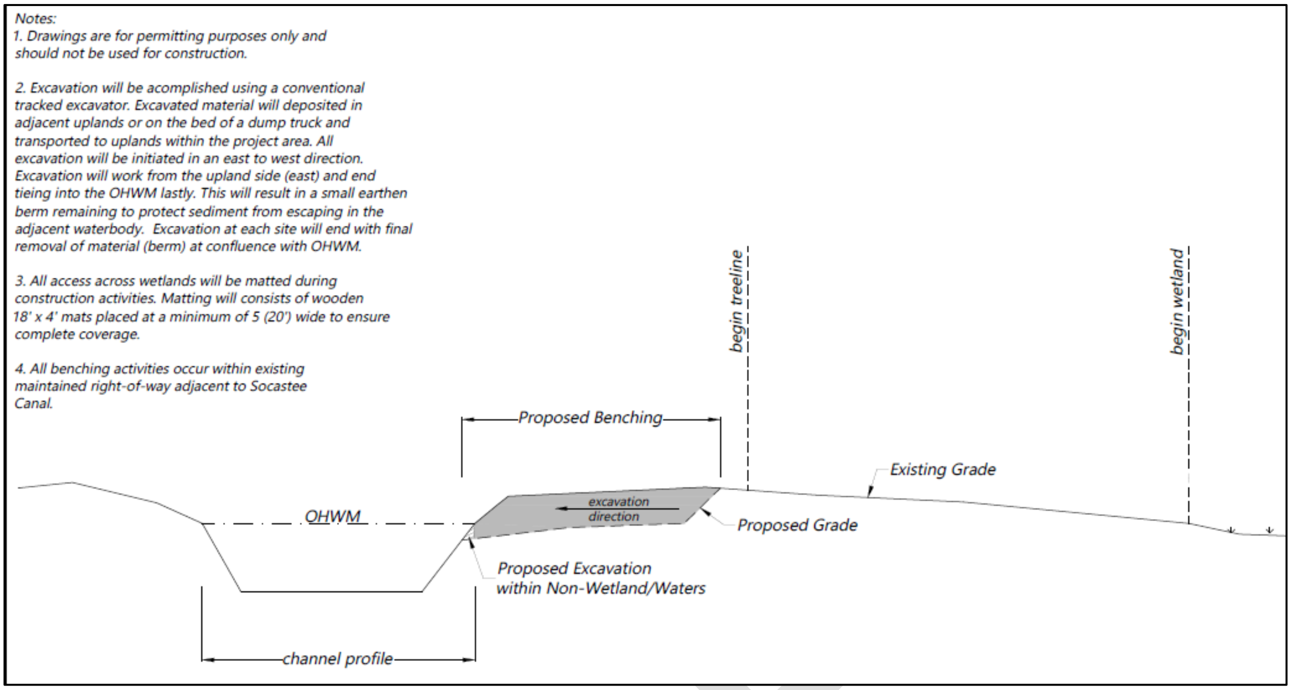


Figure 119. Proposed benching and channel profiles

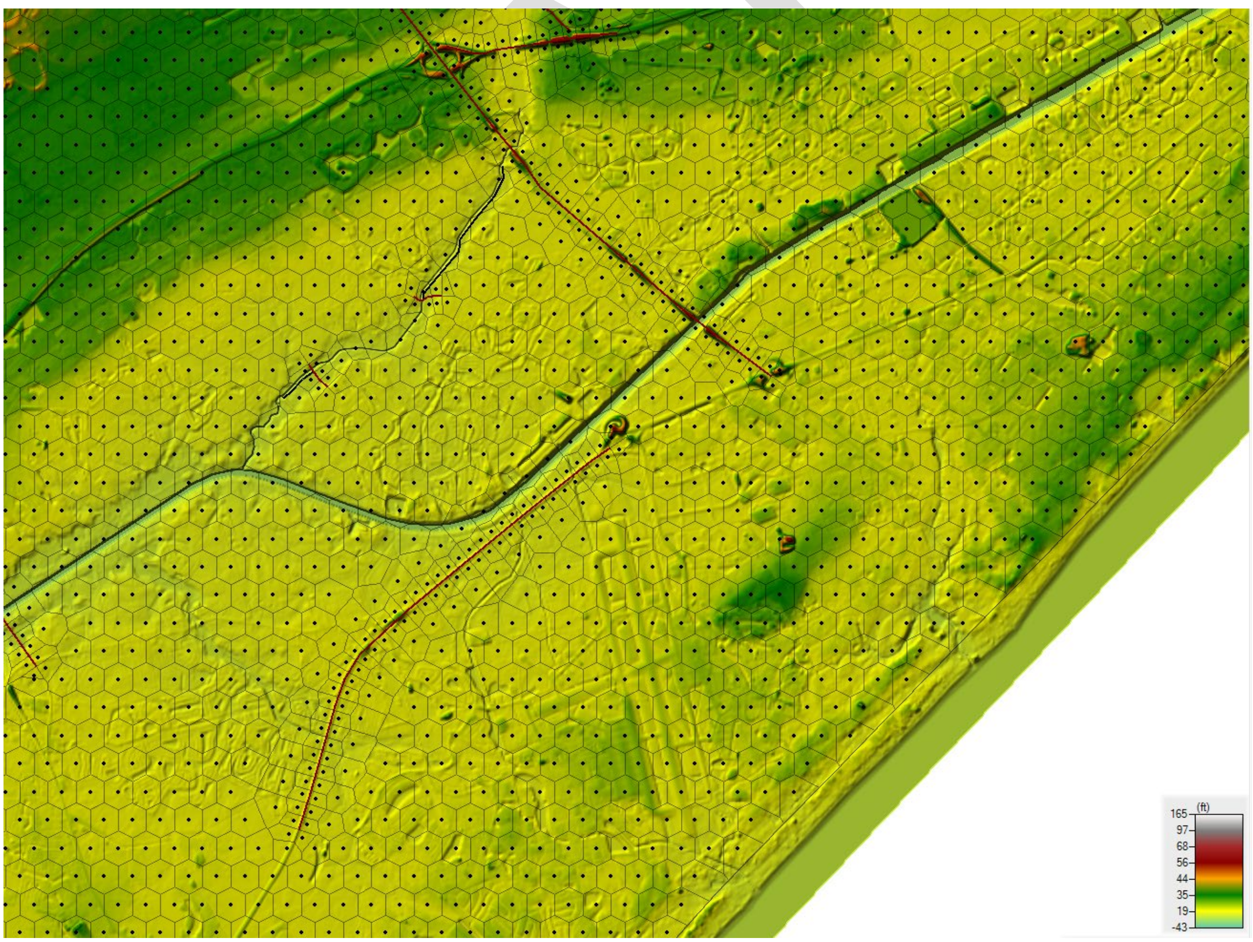


Figure 120. HEC-RAS Gridding and terrain modification

Figure 121. Map with proposed benching and 1% AEP event

Figure 122.1% AEP of the Socastee Benching project

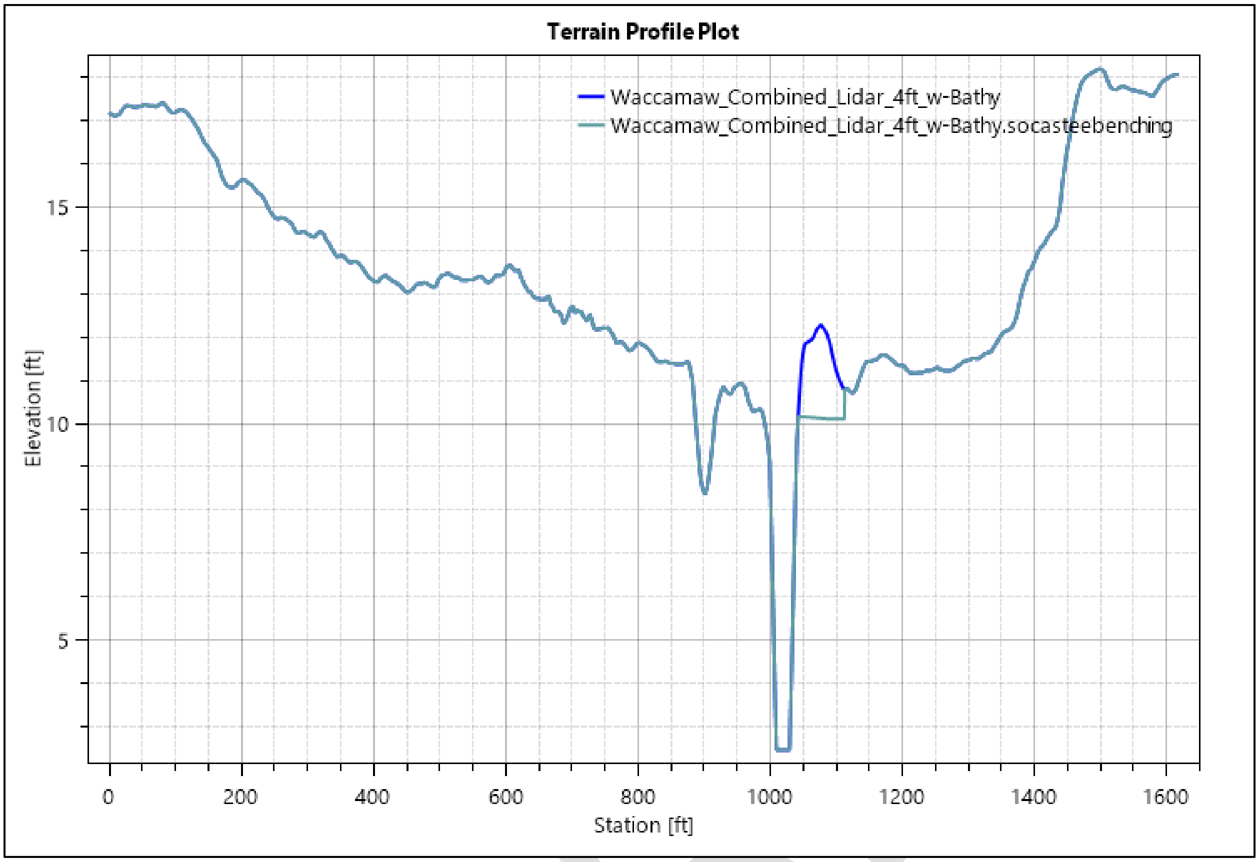


Figure 123. Modified Terrain showing existing grade in the channel and the original bathymetry of the channel.

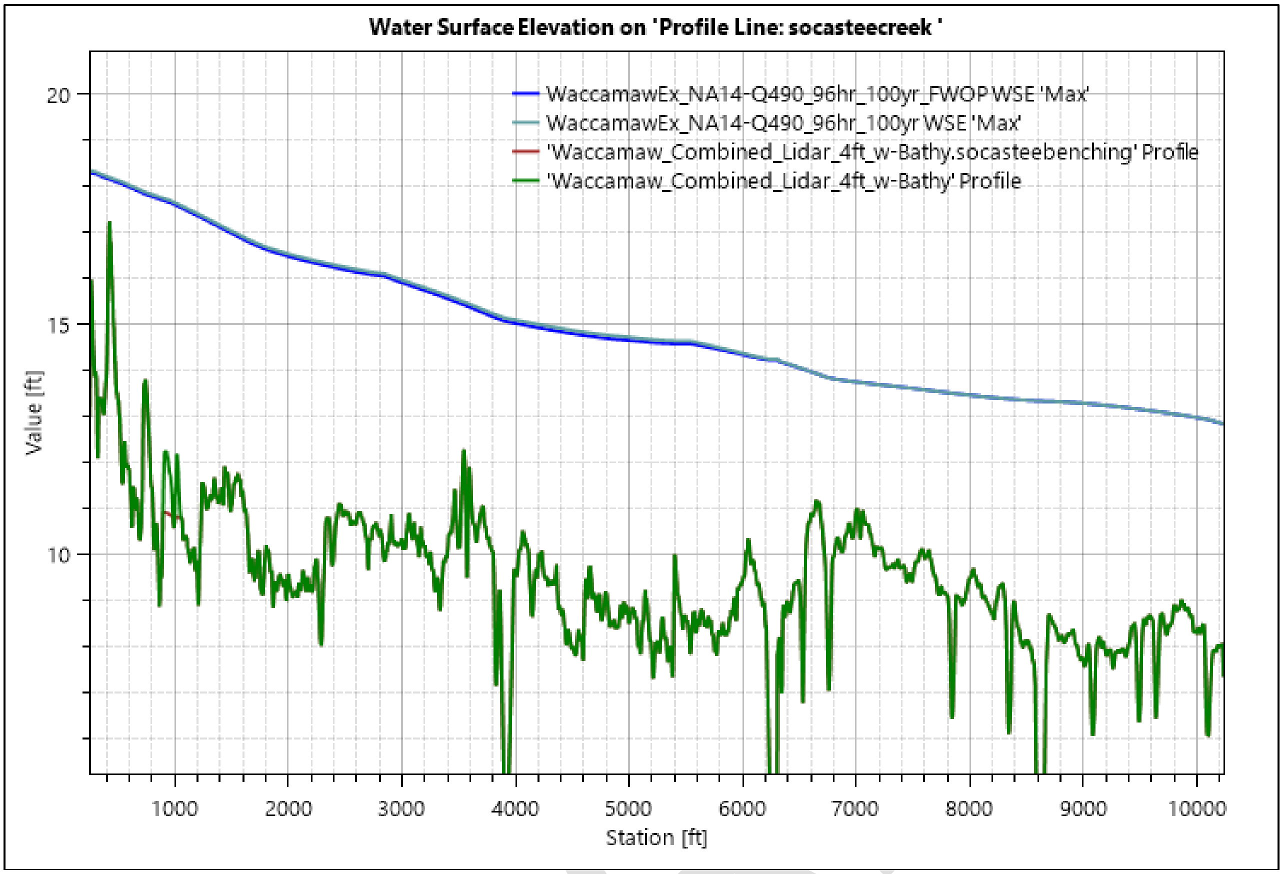


Figure 124. WSE data comparison for existing and FWOP including Socastee Benching

7.2.3 FWOP Hydraulics summary

Simulation of the 0.5-, 0.2-, 0.1-, 0.04-, 0.02-, 0.01-, 0.005-, and 0.002-AEP events with updated FWOP hydrology within the Waccamaw River basin produced profiles representative of the flooding potential for floodplain conditions that include anticipated future development. For the Bucksport and Socastee Focus area, FWOP hydraulic simulations are shown. Overall, for both Socastee Benching and Big Bull Road raising in Bucksport, a significant impact was not made on the corresponding areas from the existing conditions, however it was modeled in order to coordinate within that area for the structural measures. These measures were included because they were in proximity and implementation of them could affect the overall WSE when implementing the structural measures. The bathymetry in both Bucksport and Socastee were manually derived from existing bridge data and site plans for the two projects.

8.0 Flood Risk Management Measures

This section details the formulation and assessment of structural measures to address flood risk management in the Waccamaw River basin. A method of analysis and means of screening was based on assessment iterations due to the need to narrow down the large number of proposed measures throughout the large study area. Early assessment iterations focused on leveraging available existing reporting, data, and modeling to determine measure viability. Later iterations involved a more detailed assessment approach that included quantitative modeling to determine measure viability. This systematic approach of assessing preliminary structural measures insured that all final alternatives were effective at producing hydraulic benefits with reduced risk and minimal impacts.

8.1 Measure Development

Structural flood risk management measures were developed based on a detailed flood risk analysis of the study area and engineering judgment of structure-type performance. Measures were proposed throughout most of the Waccamaw River mainstem length as well as numerous tributaries within the basin. The scope of investigation was expanded to explore FRM opportunities in these tributaries based on existing floodplain impact areas (data provided by Horry County). The extents of exploration are in accordance with guidance (ER 1165-2-21; USACE, 1980). Notably, ER 1165-2-21 provides guidance on minimum requirements for what kinds of flood risk management measures are applicable to this feasibility study. Measures identified for this study included overbank detention sites and dam structures, levees, bridge/culvert modifications, channel modifications, road elevations and berms, barrier and debris removal, green infrastructure, and floodplain restoration.

A detention site was selected based on information provided in existing basin assessment studies from Horry County and on open space availability. Bridge and culverts were initially selected for modification based on their hydraulic performance as indicated in preliminary modeling (data provided by FEMA and SCDNR). Bridges and/or culverts that acted as constrictions significant enough to induce backwater flooding were noted and those whose negative effects coincided with inundated structures were selected for consideration. Inline detention sites were selected based on existing analysis performed following Hurricane Florence in 2018. Floodwall sites were selected based on existing flood risk in the basin and the availability of favorable topography to support such measures. Channel modification measures were selected based on existing flood risk, open space availability, changes to the stream geometry in its location and attributed upstream flood risk. Barrier and debris removal measures were selected based on historical documentation, community outreach, and field investigations. Green infrastructure and floodplain restoration measures were selected based on their potential to support existing or newly proposed traditional FRM measures.

8.1.1 Engineer Regulation 1165-2-21

Engineer regulation 1165-2-21 provides guidance for flooding considerations in small, urbanized watersheds. The regulation specifies a minimum frequency discharge and drainage area for which there would be federal interest. FRM improvements may only be captured in urban watersheds downstream from its outlet point that meet a minimum of 800 cfs for the 0.1-AEP event. A secondary requirement of drainage areas being over 1.5 square miles is stipulated when frequency discharge is unknown. Preliminary screening with ER 1165-2-21 was accomplished by utilizing the USGS StreamStats streamflow statistics and spatial analysis tool and historical documentation. (https://streamstats.usgs.gov/ss)

There were multiple tributaries to the Waccamaw River that have documented flooding concerns at the state and local community level. During this study's screening process SCDNR and other state agencies were undertaking assessments of localized flooding in the communities of Socastee, Longs, Red Bluff, Conway and Bucksport. These assessments focused on Crabtree Swamp, Buck Creek, Simpson Creek, Big Bull Landing, Cowtail Swamp and developed tributary crossing improvements to improve flood risk management.

During community outreach for the Waccamaw River basin study, additional streams were considered in addition to those included in the state assessments. Early measures visualized for implementation, prior to quantitative analyses and economic consideration, were in line with state interests (ex. focus on tributary crossings) in addition to preserving evacuation routes and overall efficiency of road networks. Road berms and/or road raises were examples of potential measures that would scale well to these smaller watershed areas.

All the forementioned tributaries were affected by the minimum frequency discharge and drainage area requirement from ER 1165-2-21 to varying degrees. In some tributary watersheds, this meant being completely screened from measure consideration; and in other cases, partial loss of FRM benefits near its headwaters. Kingston Lake and Carolina Bays in Conway were screened from further consideration in their entirety from the guidance of ER 1165-2-21. Prior to screening, Horry County and City of Conway were utilized to see if enough structural damages were occurring at the tributary confluences with the Waccamaw River mainstem to justify formulating measures based on the more significant mainstem flood inundation. However, Tilly Swamp and Stanley Creek were ultimately screened because there did not appear to be sufficient existing damages near the confluences.

At this preliminary screening level, upon ER 1165-2-21 application, there appeared to be sufficient structural damages occurring in Socastee Creek, and AIWW in Socastee, SC and Buck Creek in Longs, SC. Prior to committing to measure development and FWP conditions modeling for these two areas, an interim assessment of FWOP damages was carried out. This assessment occurred upon completion of the FWOP HEC-RAS and initial Hydrologic Engineering Center's Flood Damage Analysis (HEC-FDA) models and allowed the USACE project delivery team (PDT) to better understand the reduced available damages for measure formulation.

8.2 Preliminary Screened Measures

These measures were screened out prior to detailed economic evaluation based on disproportionate cost to benefits and considerations of environmental and/or social concerns using professional judgment and existing hydraulic analysis. Generally, the measures detailed in this section were initially assessed prior to completion of the future without project condition H&H detailed models. Furthermore, results from these screenings were instrumental in narrowing the overall hydraulic modeling footprint that would be required for detailed modeling of the recommend plan. Detailed use of the FEMA flood map and assistance from Horry County were vital in helping identify vulnerable structures within established effective and/or preliminary FEMA flood zones. SCDNR and FEMA generated flood inundation for various frequency events as determined through FEMA studies and intersected those water surface elevations with a state-wide structural inventory produced by the State of South Carolina. The repeat inundated structure inventory was taken in 2021 and included numerous structure attributes such as building footprint, foundation type, and estimated first floor elevation. In general, first floor elevations were derived from NSI data.

Lake Busbee was considered as a detention storage area in Conway, however it was screened and not included in the proposed measures. Lake Busbee has an interesting history tied to its origins and evolution. Originally, it wasn't a natural lake but rather a byproduct of industrial activity. In the mid-20th century, the area was used for sand mining operations. As the sand was extracted, a depression formed, eventually filling with rainwater to create what is now known as Lake Busbee. For several decades, Lake Busbee served various purposes. It was used for recreational activities such as fishing and boating, and its scenic beauty made it a popular spot for locals and visitors alike. However, the lake also played a role in industrial activities. Adjacent to it was a former coal-fired power plant operated by Santee Cooper.

In 2013, the coal-fired power plant was decommissioned, leading to changes in the area's landscape and land use. One significant change was the decision to drain Lake Busbee as part of the decommissioning process. This decision was met with mixed reactions from the community, as the lake had been a beloved recreational spot for many. After the lake was drained, there were discussions and debates about what should be done with the area. Some advocated for restoring the lake to its former glory, while others saw an opportunity for redevelopment and revitalization. Eventually, the decision was made to transform the site into an eco-friendly recreational area and wildlife habitat. The transformation of Lake Busbee included creating wetlands, planting native vegetation, and establishing walking trails around the perimeter. These efforts aimed to not only restore the ecological balance of the area but also to provide a space for outdoor recreation and education. Lake Busbee continues to evolve as a natural space where people can enjoy activities like birdwatching, hiking, and picnicking. Its history as a man-made lake born from industrial activity has been transformed into a story of environmental stewardship and community engagement. Because of the lake industrial activity and the ecological and contamination from the industrial park, the use of Lake Busbee for storage would not be ecologically feasible or reasonable so this option was screened out.

Channelization along Waccamaw - and structural measures overall - along the Waccamaw were overall not implemented because there the Waccamaw is listed on the Nationwide Rivers Inventory under the Wild and Scenic Rivers Act, so alterations of the river main stem would not be allowed. Therefore, channelization and floodwalls along the main stem affecting the wild and scenic nature of the river would be prohibited. In addition to the restrictions, implementation of a floodwall along the main stem of the Waccamaw, would also be cost prohibitive due to the length of the wall to reach high ground to high ground. The length of the wall to reach high ground would be longer than the actual flood protected areas. Waccamaw is a low-lying floodplain with a relatively small slope along the channel, therefore high ground is considerably further from the main stem.

8.3 Evaluated Measures

The measures in the following section went through the same screening process as those outlined in the previous sections and were found to justify more detailed hydraulic and economic analysis. The sections below describe this additional analysis.

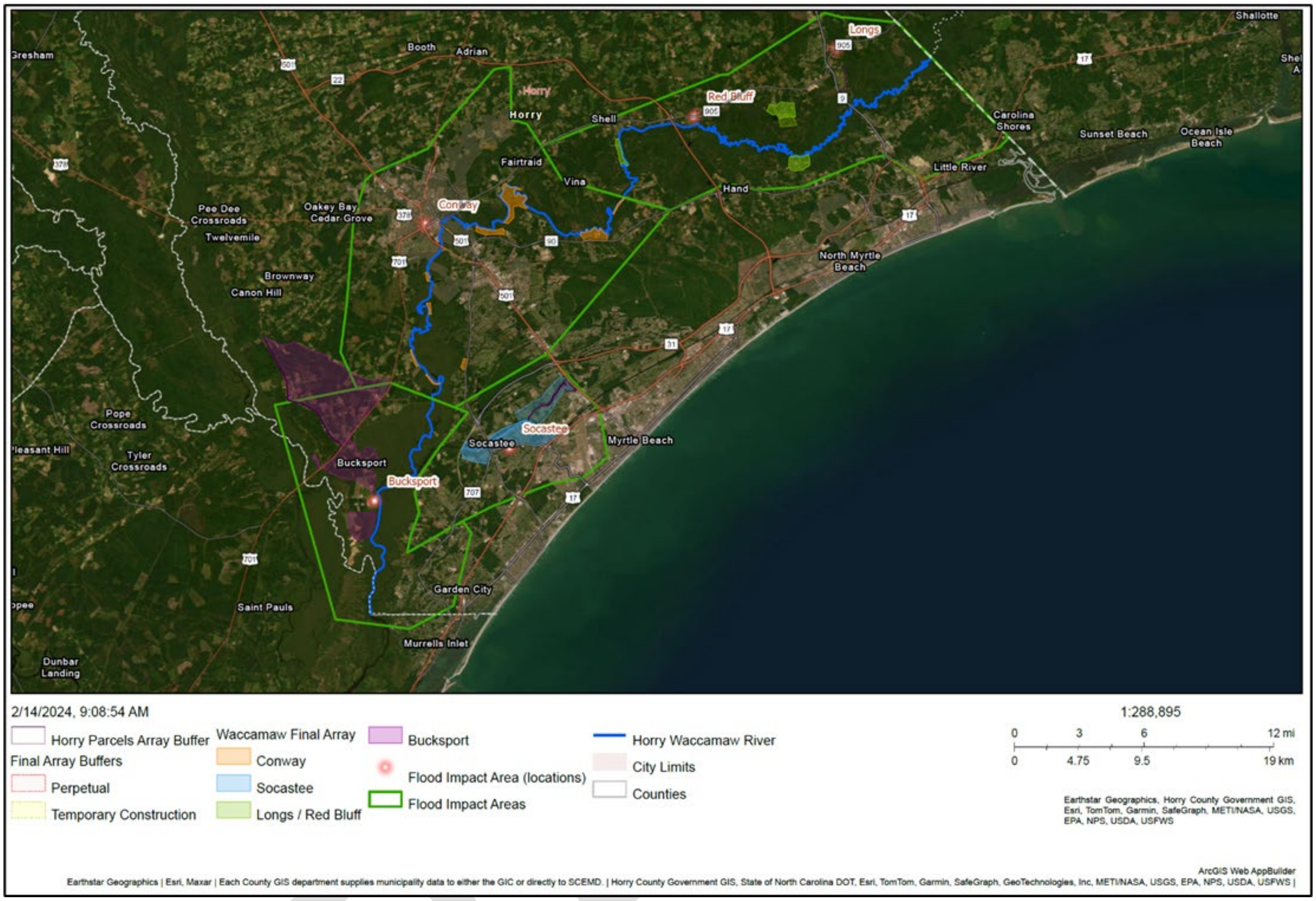


Figure 125. Waccamaw Final Array of Evaluated Measures

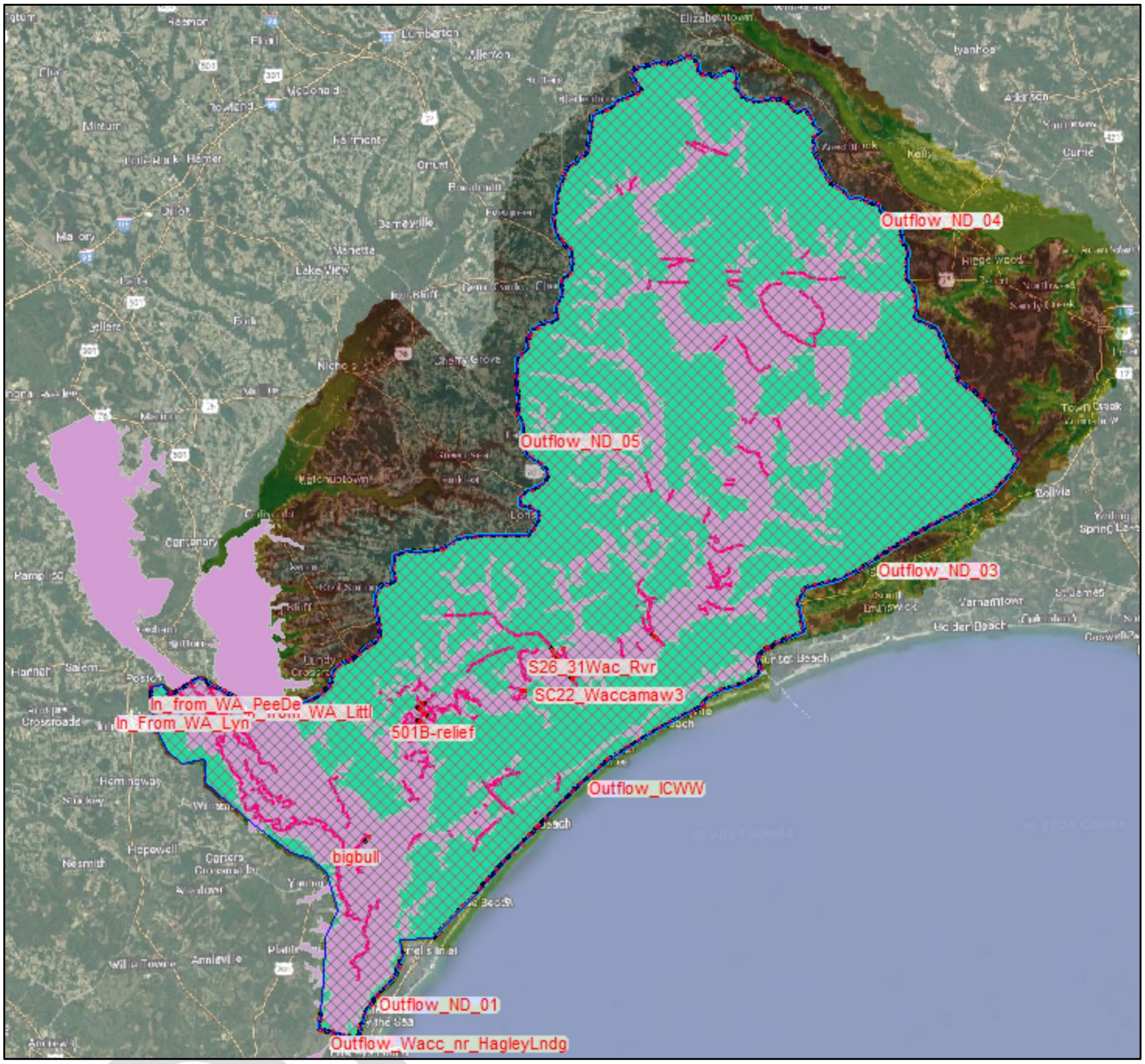


Figure 126. HEC-RAS model showing array of evaluated measures

8.3.1 Longs/Red Bluff Structural Array of Alternatives

The following structural measures were evaluated for the Longs/Red Bluff Focus area:

- LR1 – Levee/Floodwall along Buck Creek at Rolling Ridge and Cox Lane (79 million)
- LR3 – Simpson Creek Benching, Relief Bridges
- LR6 – Levee/Floodwall along Buck Creek and Rolling Ridge, Benching, Relief Bridges

Table 28 shows the full array of measures that were considered, color coded by whether they were retained for evaluation or screened prior to analysis, and Figure 128 maps the evaluated measures.

Table 29. Screened and Retained Measures for Red Bluff/Longs Focus Area

Red Bluff/Longs	Screening Rationale
Levee/Floodwalls	Retained
Stream Channelization/modification	Screened; Portion of waterway designated as Wild and Scenic
Floodplain Relief/Benching	Retained
Improve Water Connection	Screened; Wetland impacts, critical habitat impacts, real estate concerns, acceptability issues, High cost, transfer of risk
Elevation	Retained
Acquisition	Retained
Watershed Storage	Screened; Environmental impacts, land owner constraints, agency concerns

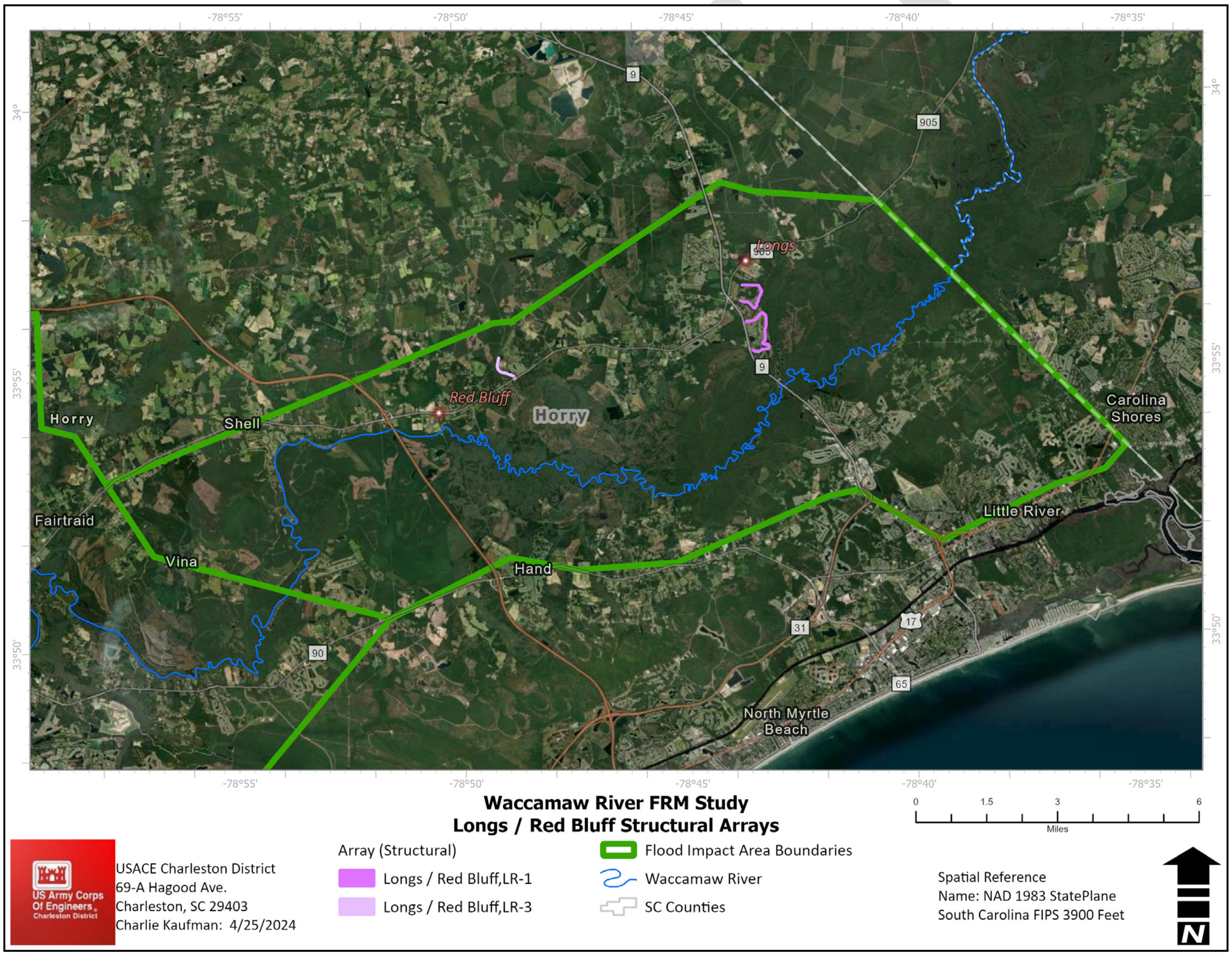


Figure 127. Longs/Red Buff Evaluated Measures

8.3.1.1 LR1: Levee/Floodwall along Buck Creek at Rolling Ridge and Cox Lane

Floodwalls can impede the natural exchange of water between surface water bodies and groundwater systems. This reduced interaction can hinder the recharge of groundwater aquifers, which are important sources of drinking water and support for ecosystems. Floodplains serve as natural buffers during flood events by absorbing excess water and reducing flood peaks. Floodwalls can disconnect the floodplain from the main river channel, reducing its ability to absorb and store floodwaters. This loss of connectivity can exacerbate flooding downstream and increase flood risk in surrounding areas. Floodwalls can fragment and isolate wetland ecosystems, disrupting their hydrological connectivity with adjacent water bodies. This fragmentation can degrade wetland habitats, reduce biodiversity, and impair the ecosystem services they provide, such as water filtration and flood control.

In some cases, floodwalls can create backwater effects upstream, where water levels rise higher than they would naturally during flood events. These elevated water levels can inundate surrounding areas that would not have flooded otherwise, leading to unexpected flood impacts and property damage. Overall, while floodwalls can provide protection against flooding in urban areas, their construction and maintenance can have significant negative impacts on hydrology and hydrogeology, as well as on the surrounding ecosystems and communities. It's important for planners and engineers to consider these impacts when designing flood protection infrastructure and to explore alternative approaches that minimize adverse effects on natural systems.

However, in this case, floodwalls have several positive effects on hydrology in regard to flood control. Floodwalls help in controlling the flow of water during periods of heavy rainfall or storm surges. By confining the water within specific boundaries, floodwalls reduce the risk of flooding in adjacent areas, protecting communities and infrastructure. Floodwalls channel water flow, directing it away from sensitive areas such as residential neighborhoods or agricultural land. This controlled flow can prevent erosion and sedimentation in waterways, maintaining their ecological health.

In this situation, containing floodwaters, the floodwall can minimize erosion along riverbanks and coastal areas. This preservation of soil helps maintain the stability of ecosystems and protects against loss of land and property. Floodwalls can prevent contaminants carried by floodwaters from spreading into surrounding areas. By confining the water within defined channels, floodwalls can facilitate the implementation of water treatment measures, leading to improved water quality downstream. The floodwall can be integrated into comprehensive water management systems, allowing for better regulation of water levels in rivers, lakes, and other water bodies. This can help mitigate the impact of both floods and droughts, ensuring a more reliable water supply for various uses.

The floodwall along Longs/Red Bluff protects critical infrastructure such as roads, bridges, and utilities from damage caused by flooding. This safeguarding of infrastructure reduces maintenance costs and minimizes disruptions to transportation and communication networks. By providing a physical barrier against flooding, floodwalls reduce the risk of property damage and loss of life during extreme weather events. This can lead to lower insurance premiums for residents and businesses located in flood-prone areas, as well as greater overall resilience to climate-related hazards. Overall, the implementation of floodwalls can contribute to more sustainable and resilient hydrological systems, benefiting both human communities and the natural environment.

Structurally the floodwall consists of a sheet pile floodwall or earthen levee, in two distinct segments, along the right bank of Buck Creek adjacent to the Aberdeen community continuing north to Rolling Ridge drive. Floodwall/levee height is estimated at 5-11 ft and approximately 2 miles long. From the center line of the wall on each side, a perpetual 25-foot-wide easement is required for maintenance, plus a 10-foot-wide temporary easement during construction, totaling 70 feet. Where the wall hugs a waterway, the 70 feet will be taken on one side of the wall for construction. Pump stations would be required in conjunction with the flood wall/levee to alleviate interior flooding. These features are positioned, either permanently or temporarily, at the low points along the structure. The proposed location of the structures is in or adjacent to Aberdeen Country Club, Cox Lane, and Rolling Ridge Drive.

Some considerations and assumptions are that Buck Creek routinely floods during intense rainfall events. During storm events, road closures frequently cutoff this area from local resources, blocking access to grocery stores, pharmacies and other essential needs for the senior population in the area. A 5-11ft high wall above the existing grade would provide 1% AEP flood protection, wall height would vary and tie into high ground on each end. Taller sections of the levee would be constructed as T-wall and require a more extensive foundation. Proximity to Buck Creek limits the space for this measure, therefore, acquisition of a portion of the Aberdeen golf course and other private property may be required for implementation of this measure. Floodplain encroachment and pre-construction site clearing pose possible environmental impacts. There are 4 centrifugal pumps on protected side of the wall to capture the ponded water in the region in the cost estimate. These pumps were not included in the hydraulic modeling however, they were captured in the cost and economic estimations.

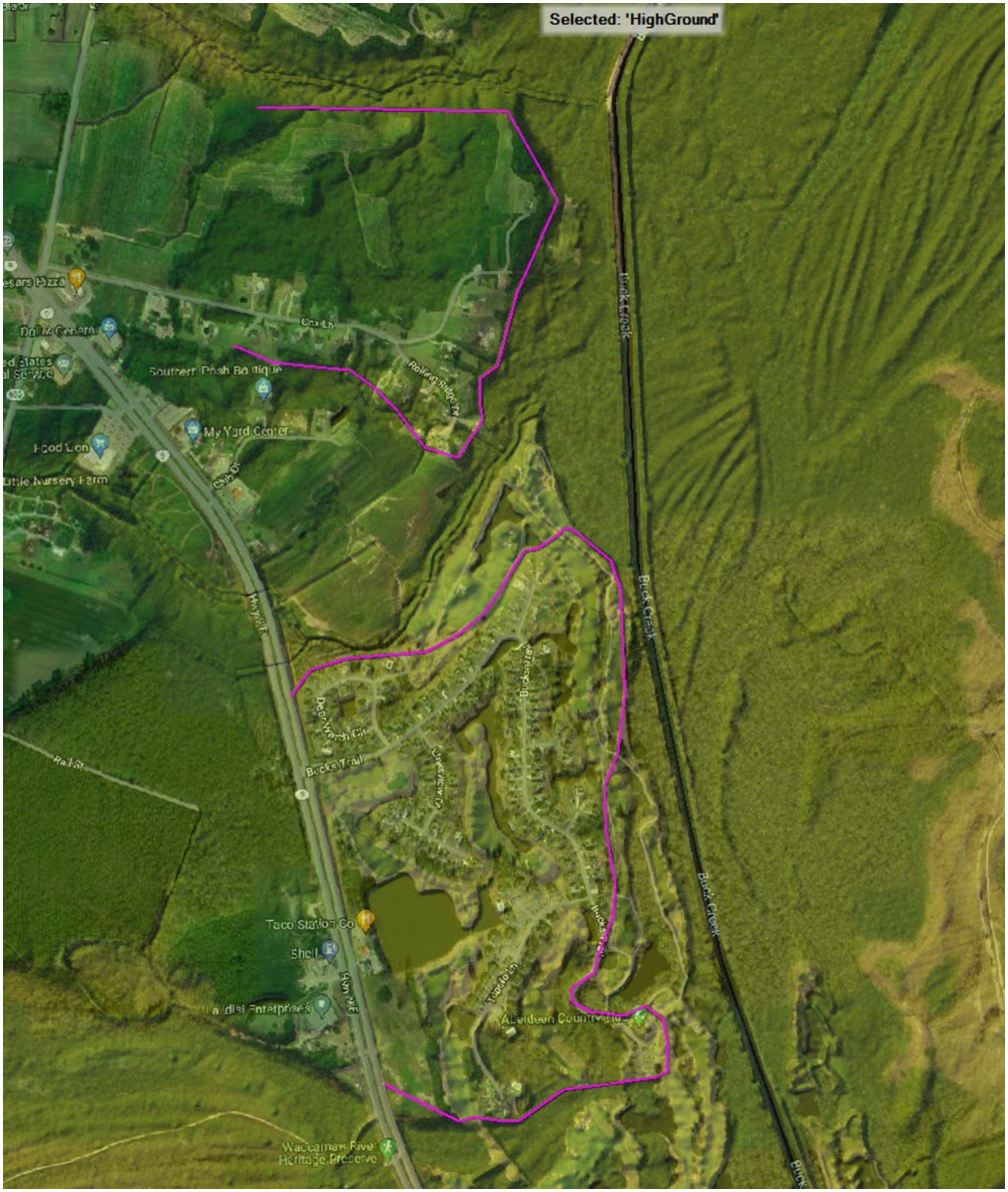


Figure 128. Floodwall (in pink) in Longs along Buck Creek

Figure 130 shows the FWP and FWOP modeling of the floodwall in Longs. The darker blue represents the flooding and depth with and without the wall modeled for 1% AEP events. There is an overall reduction in depth with the structural inventory, however that did not supersede the cost of the wall. The wall is from high ground to high ground which extended the length of the wall.

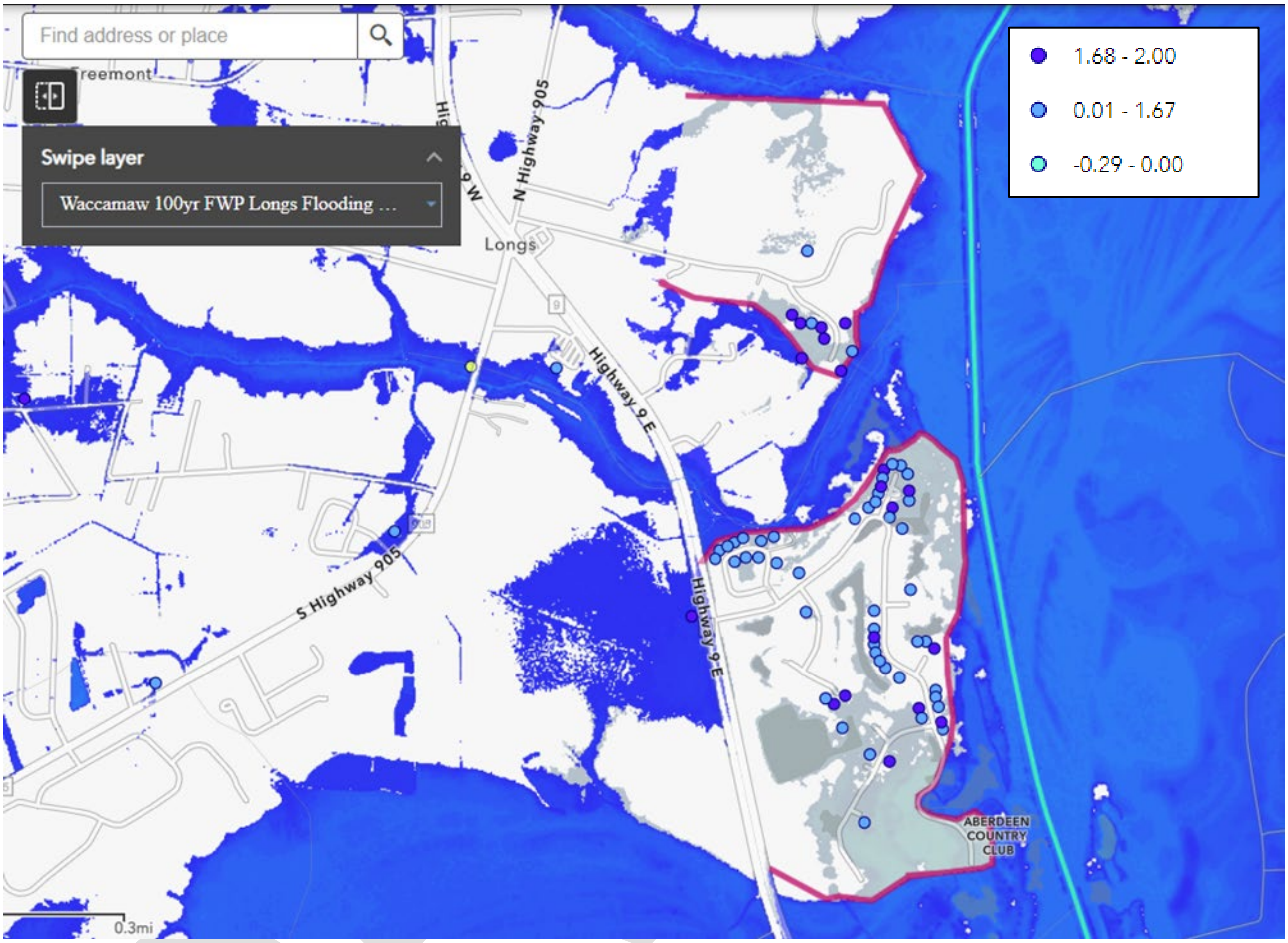


Figure 129. FWP (Blue) and FWOP (grey) modeled Floodwall in Longs for 1% AEP. Structures are indicated with the dots with varying depths of protection.

8.3.1.2 LR3: Benching and Relief Bridges

Streambank benching consists of using excavation methods upstream of HWY 905 along Simpson Creek. Activity proposed to open channel and allow stream connection back to the floodplain surrounding Simpson Creek. Benching extents to be determined. A relief bridge is anticipated for Simpson Creek bridge as it passes under HWY 905.

Relief Bridges are proposed culverts/water connections in areas where conveyance is restricted by roadways, bridges, or similar abutments. These drainage improvements will be placed along the Hwy 905 and Simpson Creek intersection. Improvement activities include clearing streambanks under the bridge and installing culverts in the stream and within the abutments.

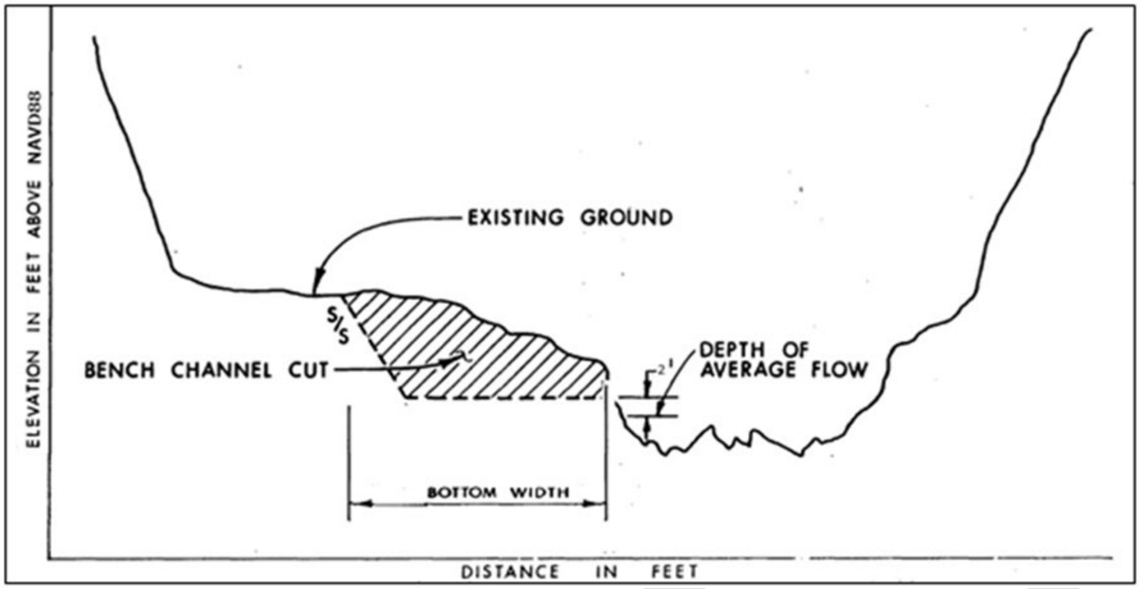


Figure 130. Typical Benching cross section

Benching of creeks, which involves cutting into the natural banks to create flat areas or benches, can have several negative impacts on hydrology. Benching can destabilize the creek banks, leading to increased erosion. The removal of vegetation and disturbance of soil structure weaken the banks' ability to resist erosion, resulting in sedimentation downstream and degradation of water quality. Creeks and their surrounding riparian zones provide critical habitat for a variety of plant and animal species. Benching reduces the available riparian habitat by removing vegetation and altering the natural features of the creek, leading to loss of biodiversity and disruption of ecological functions. Benching can compromise the stability of creek banks by removing natural vegetation that helps anchor the soil and absorb excess water. This can lead to bank collapse and channel widening, further exacerbating erosion and sedimentation issues.

Benching alters the natural flow dynamics of creeks by changing the channel geometry and cross-sectional area. This can lead to changes in water velocity, sediment transport, and channel morphology, potentially increasing the risk of flooding and impacting downstream ecosystems and infrastructure. Benching can disrupt the connection between surface water and groundwater systems by altering the natural hydrological processes. Reduced infiltration and groundwater recharge can lead to lowered groundwater levels, impacting local aquifers and water availability for both human and ecological needs. Benching reduces the extent of the natural floodplain by narrowing the creek channel and removing vegetation. This diminishes the flood storage capacity of the creek, increasing the risk of flooding during high-flow events and reducing the ability of the floodplain to provide important ecosystem services such as water filtration and groundwater recharge.

Overall, benching of creeks can have significant negative impacts on hydrology by disrupting natural processes, reducing habitat quality, and increasing the vulnerability of ecosystems and communities to flooding and erosion. It's important to carefully consider the potential consequences of creek modification projects and to prioritize strategies that minimize adverse effects on the natural environment.

While benching of streams can have negative impacts on hydrology, there are some situations where it may provide certain positive effects, albeit to a lesser extent. Here are a few potential positive impacts on hydrology for benching in streams. Benching can create a more stable and defined channel within the stream, which may enhance connectivity between the main channel and the floodplain during low to moderate flow conditions. This improved connectivity can facilitate the exchange of water, sediment, and nutrients between the stream and adjacent floodplain areas, supporting ecosystem health and productivity. Benching can create diverse habitat types along the stream corridor, including pools, riffles, and shallow areas. These habitat variations can support a wider range of aquatic species and increase overall biodiversity within the stream ecosystem.

In this case, strategic benching can help stabilize eroding stream banks by providing a transition zone between the main channel and the floodplain. This transition zone can help absorb energy from flowing water, reduce erosive forces, and promote the establishment of riparian vegetation, ultimately enhancing bank stability and reducing sedimentation downstream. Benching can create opportunities for riparian restoration and enhancement efforts along the stream corridor. By establishing vegetation buffers and restoring natural hydrological processes, benching projects can improve water quality, provide wildlife habitat, and enhance the aesthetic value of the stream corridor. Management of Urban Stormwater Runoff: In urban areas, benching projects can be integrated with stormwater management practices to help mitigate the impacts of urbanization on hydrology. By incorporating features such as vegetated swales, infiltration basins, and bio-retention areas into the benching design, runoff volume and peak flows can be reduced, improving water quality and reducing the risk of flooding downstream.

It's important to note that the positive impacts of benching on hydrology are context-specific and depend on factors such as site conditions, project objectives, and stakeholder priorities. Careful planning, site assessment, and implementation are essential to maximize the potential benefits of benching while minimizing negative consequences on stream hydrology and ecosystem functions. Additionally, thorough monitoring and adaptive management are necessary to evaluate the effectiveness of benching projects over time and make any necessary adjustments to optimize outcomes.

This alternative is formulated to increase conveyance in the proposed protection areas by reducing flood elevations and backwater effects. Increased water velocity may result in stream scouring and erosion. Enhancement of culverts/water connection at the HWY 905 intersection with Simpson Creek where bottlenecking occurs could potentially be a collaboration project with SCDOT. Environmental impacts associated with stream encroachment and removing fill from the streambanks apply. The project consists of a 140 width with a 1:1 slope and a max width of 200 ft, with a total cutoff 714,373 cu yd.

Figure 131. Location of the benching along Simpson Creek.

The most frequent design storms, 0.5-AEP through 0.02-AEP, appeared to best utilize the floodplain bench for flood conveyance. Their flood boundaries were confined by the natural terrace on the north, left overbank side of the river. This boundary was characterized by older developed residential neighborhoods. The channel bench’s added flood conveyance had a diminishing effect to WSEL reduction as the design storm frequency was lowered. This effect meant that when flood inundation did eventually reach the more populated areas of the subdivision, within the 0.01-, 0.005-, and 0.002-AEP impacted areas, the added benefit from this measure was not as prominent.

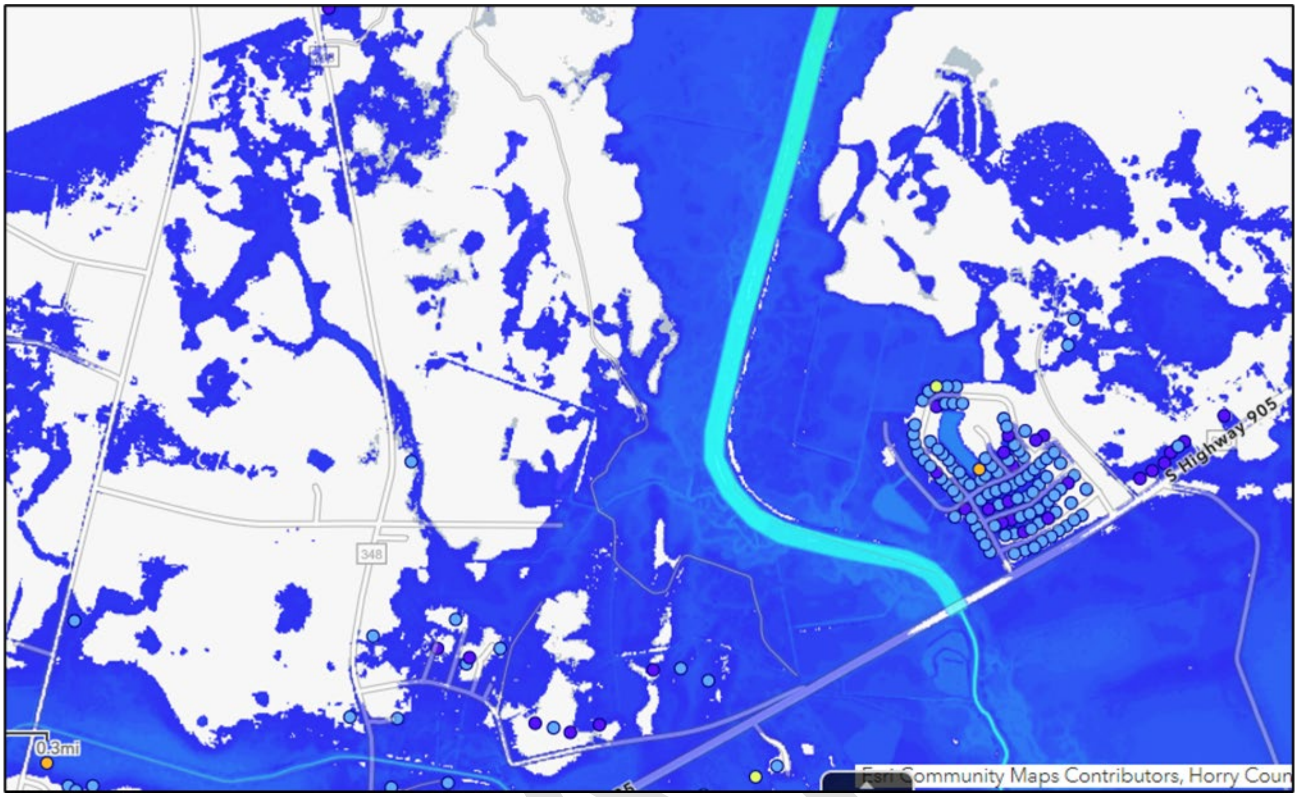


Figure 132. Depths for FWP 1% AEP for Benching in Simpson Creek

In general, while this measure was effective at reducing flood elevations for the more frequent design storms, it was unable to provide significant WSEL and depth reductions during the more severe events, which was assumed to contain the majority of FWOP damages. Despite these concerns, it was decided that this measure would be carried forward for detailed economic assessment.

8.3.1.3 LR6: Combined Modeling of all structural measures for Longs/ Red Bluff

Sheet pile floodwall or earthen levee, in two distinct segments, along the right bank of Buck Creek adjacent to the Aberdeen community continuing north to Rolling Ridge Drive. Flood wall/levee height is estimated at 5-11 ft and approximately 2 miles long. From the center line the wall on each side, a perpetual 25-foot-wide easement is required for maintenance, plus a 10-foot-wide temporary easement during construction, totaling 70 feet. Where the wall hugs a waterway, the 70 feet will be taken on one side of the wall for construction. Pump stations would be required in conjunction with the flood wall/levee to alleviate interior flooding. These features are positioned, either permanently or temporarily, at the low points along the structure.

Streambank benching using excavation methods upstream of HWY 905 along Simpson Creek. Activity proposed to open channel and expand overflow capacity of Simpson Creek. Benching dimensions to be determined during feasibility design. These drainage improvements will be placed along the Hwy 905 and Simpson Creek intersection. Construction activities include clearing stream under the bridge and installing culverts in the stream and within the abutments. Proposed protection (Levee/Floodwall): Property in or on Aberdeen Country Club to Rolling Ridge Drive. Proposed protection (relief Bridges/benching): Residents on Parker drive and McNeil Chapel Rd. Could potentially benefit residents on Jefferson Rd and Mountain Drive.

This alternative is structured to increase conveyance in the proposed protection areas by reducing flood elevations and backwater effects. Buck Creek routinely floods during intense rainfall events. A 5-11ft high wall above the existing grade would provide 1% AEP flood protection and is proposed in the Aberdeen community. This wall height would vary and tie into high ground at both ends. A sheet pile wall would require a more extensive footing/foundation (height exceeds 5ft). Changes in water flow may result in stream scouring and erosion. Relief bridges are proposed along HWY 905 between Todd Swamp and Simpson Creek where bottlenecking occurs. Floodplain encroachment and pre-construction site clearing pose possible environmental impacts. Proximity to Buck Creek limits the space for construction of a floodwall/levee, therefore, acquisition of a portion of the Aberdeen golf course and other private property may be required for implementation of this risk management plan. Environmental impacts associated with stream encroachment and removing fill from the streambanks apply.

Overall, the combined measures provided flood protection and there was an overall reduction in depth with the structural inventory, however that did not supersede the cost of the wall. The wall is from high ground to high ground which extended the length of the wall, and made it more costly. The benching provided some reduction in water surface elevation but not significant enough to justify the cost of the production, thus resulting in a non-positive BCR.

8.3.2 Conway Structural Array of Alternatives

Conway is the centermost portion of the Waccamaw River with the most urbanized region. Formulating measures for this region proved to be a difficult task, however one structural measure for Conway was retained, relief bridges. Figure 133 shows the outline of the focus area on Conway and the retained and evaluated structural measures.

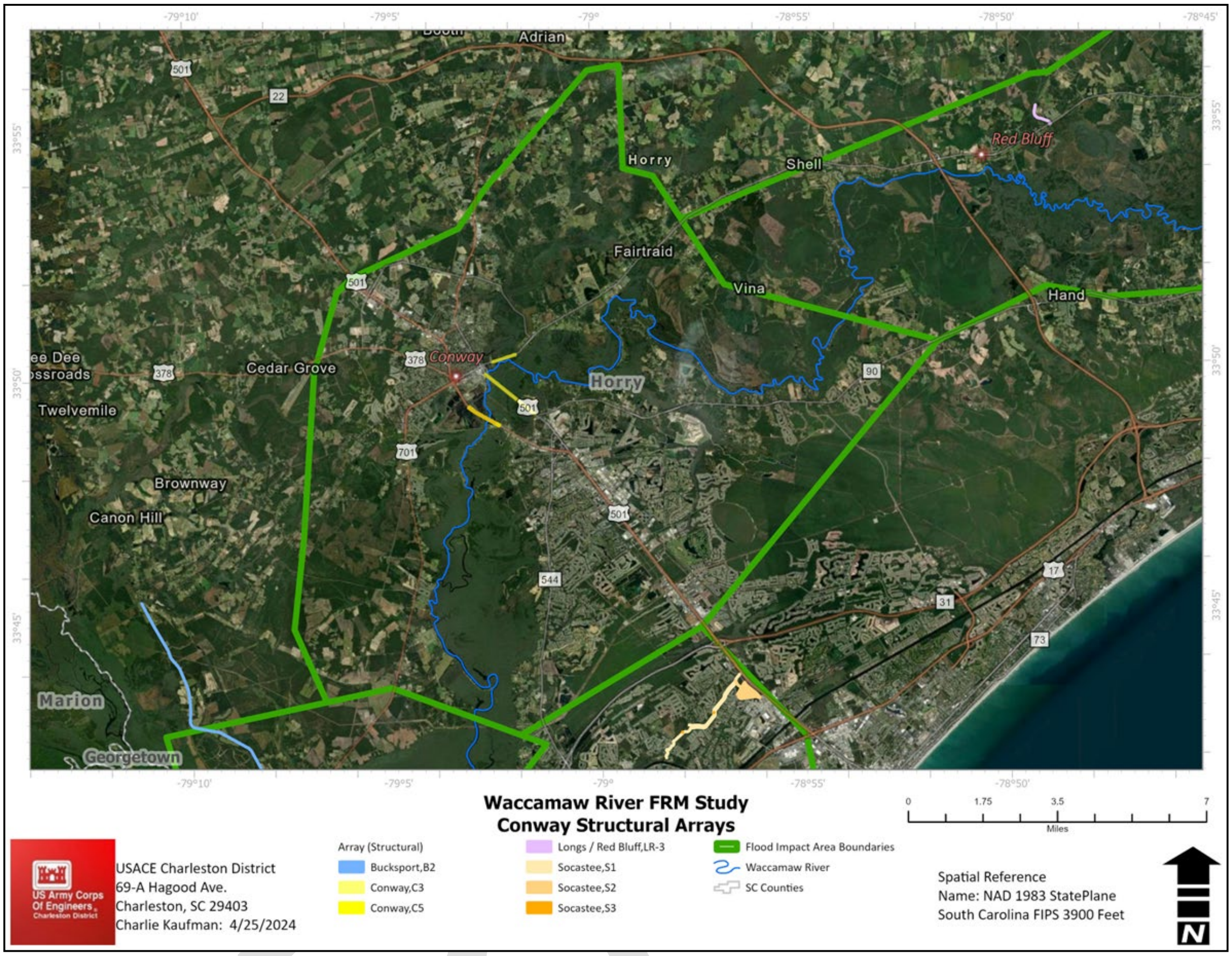


Figure 133. Conway Structural Arrays

Table 29 shows the measures retained and screened measures for Conway. Floodwalls and Ring Levee were proposed however there was no high ground to tie into, within a reasonable distance without cutting off a significant part of the channel. Retention and detention ponds were screened as well because of the environmental impacts of Lake Busbee. Relief Bridges were retained since they would not impact the wild and scenic portion of the Waccamaw, rather allow for the flow to convey somewhat naturally without overtopping the road.

Table 30. Screened and Retained Measures for Conway Focus Area

Conway	Screening Rationale
Floodwalls	Screened; High cost, environmental impacts, real estate concerns
Ring Levee	Screened; High cost, environmental impacts, real estate concerns
Increase capacity of Lake Busbee	Screened; Previous industrial activities, environmental concerns, HTRW issues, not enough storage capacity, recreation impacts
Detention/Retention	Screened; Significant environmental impacts, high mitigation likely, HTRW issues
Clearing and Snagging	Screened; not effective
Road Elevation	Screened; low effectiveness, real estate concerns, stormwater improvements would be needed
Relief Bridges	Retained
Elevation	Retained
Acquisition	Retained
Watershed Storage	Screened; Environmental impacts, landowner constraints, agency concerns
Flood warning System	Screened; Horry County emergency response notification system is up to date, unable to identify improves that would reduce risk

Relief bridges, also known as grade separation structures, are designed to elevate one transportation route over another to avoid intersections or conflicts between traffic flows. While they offer several benefits such as improved traffic flow, safety, and reduced congestion, they can also have hydrologic and hydraulic effects, both positive and negative. Relief bridges can minimize the risk of flooding by allowing water to flow more freely underneath, especially during heavy rainfall or flood events. By providing a larger opening for water to pass through, they can reduce the chances of water backing up and causing localized flooding. By maintaining a clear path for water flow, relief bridges can help stabilize the natural channels underneath. This can prevent erosion and sediment buildup, maintaining the integrity of the watercourse and reducing the risk of channel shifting or bank erosion. Relief bridges can increase the hydraulic capacity of waterways by providing a wider and deeper opening for water to pass through. This can improve overall drainage and reduce the likelihood of overtopping during high-flow events.

However, relief bridges can alter the natural flow patterns of watercourses by introducing barriers to flow. This alteration can disrupt the natural movement of sediment and aquatic habitats, potentially leading to ecological impacts downstream. The increased velocity of water passing through relief bridge openings can lead to higher levels of erosion in the channel bed and banks downstream. This erosion can undermine the stability of the watercourse and adjacent infrastructure, potentially leading to maintenance issues and increased long-term costs. Relief bridges may also create areas where sediment accumulates, particularly at the entrance and exit points of the bridge openings. Over time, this sediment buildup can reduce the hydraulic capacity of the watercourse, increase flood risk, and necessitate costly maintenance efforts to remove accumulated sediment.

Overall, while relief bridges offer significant benefits in terms of traffic efficiency and safety, their construction and presence can have notable hydrologic and hydraulic effects on surrounding waterways. Proper design, mitigation measures, and ongoing maintenance are essential to minimize negative impacts and maximize the positive contributions of relief bridges to both transportation networks and hydrological systems.

The structural measure evaluated in Conway is to add relief bridges/culverts at 501 Business, 501 Bypass, and 905 to increase conveyance through these areas where potential bottlenecking is occurring. The exact location is still being determined with the County, however, the modeled location is in excess of 500 ft of the bridge abutments, which meets SCDOT regulations. The proposed protection is for the relief bridges/culverts at 501 and 905 to increase conveyance through these areas where potential bottlenecking is occurring.

Edward E. Burroughs relief bridges would most likely consist of culverts due to the proximity of the existing bridge. The proposed protections include decreasing the flood depths and size of the floodplain upstream of the Edward E. Burroughs highway along the Waccamaw River. This relief bridge would convey more water away from the inundated zone.

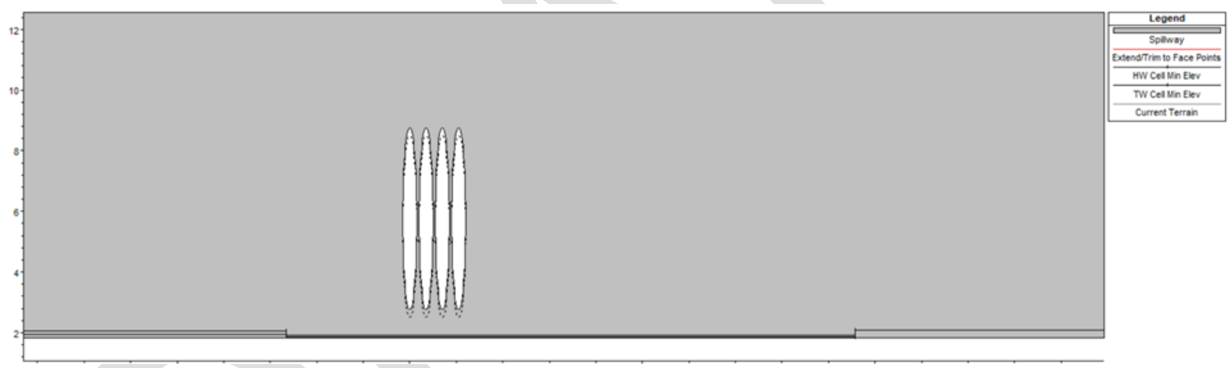


Figure 134. Geometry in HEC RAS Cross section of the relief bridge for 501 B

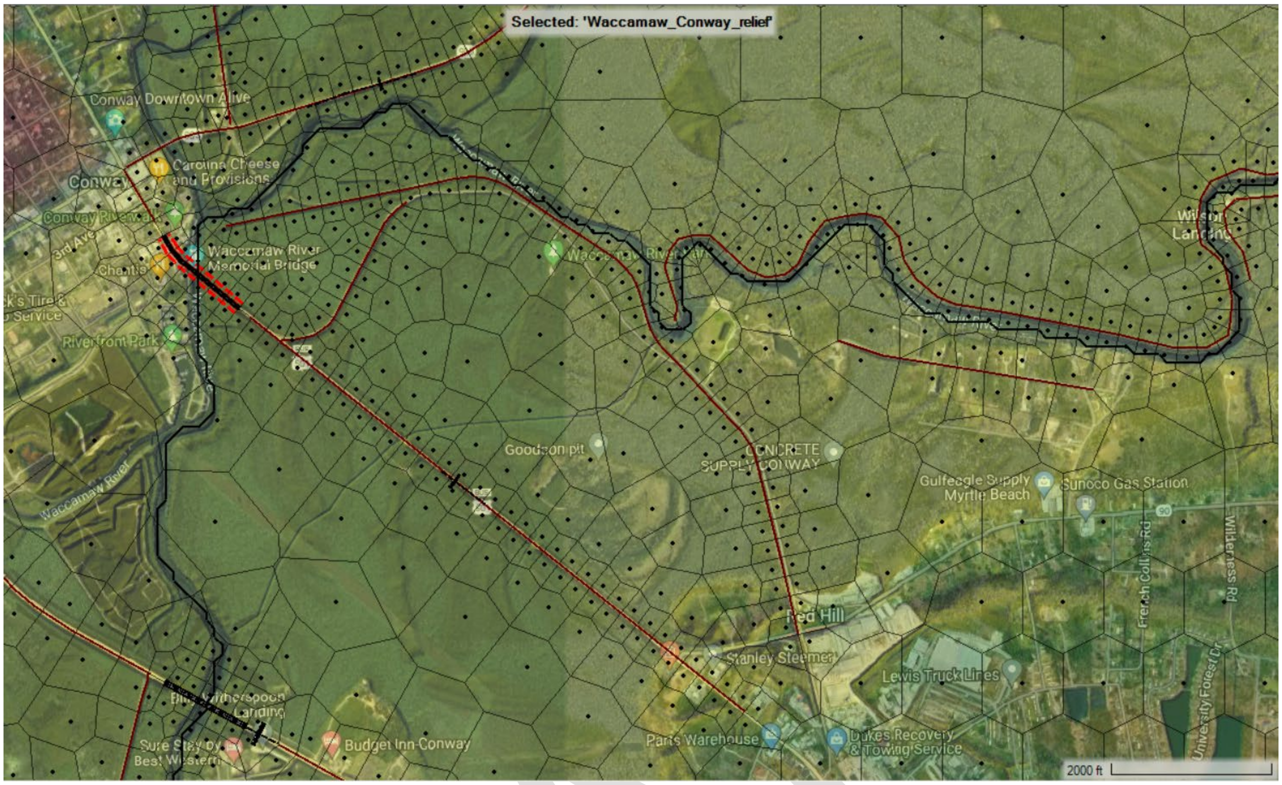


Figure 135. Three locations of the relief bridges in Conway; 905, 501B, and 501

Highway 501 Business and Highway 501 cross the Waccamaw River, and Highway 905 crosses Crabtree Swamp. The embankments cut through the natural floodplain and cause backwater effects that propagate upstream.

Figure 138 shows the 1% AEP water depths in Conway after evaluating the relief bridges measures. The relief bridges were combined into a single model because any single relief bridge did not show a significant decrease in WSE. Since the three bridges were near one another, the three relief bridges were included into the FWP model. The relative low cost of the relief bridges conveyed a positive BCR. As indicated in Figure 137, with the location of the cross section in Figure 136, there is a reduction in Water Surface Elevation of 1.08 ft downstream of Highway 501.

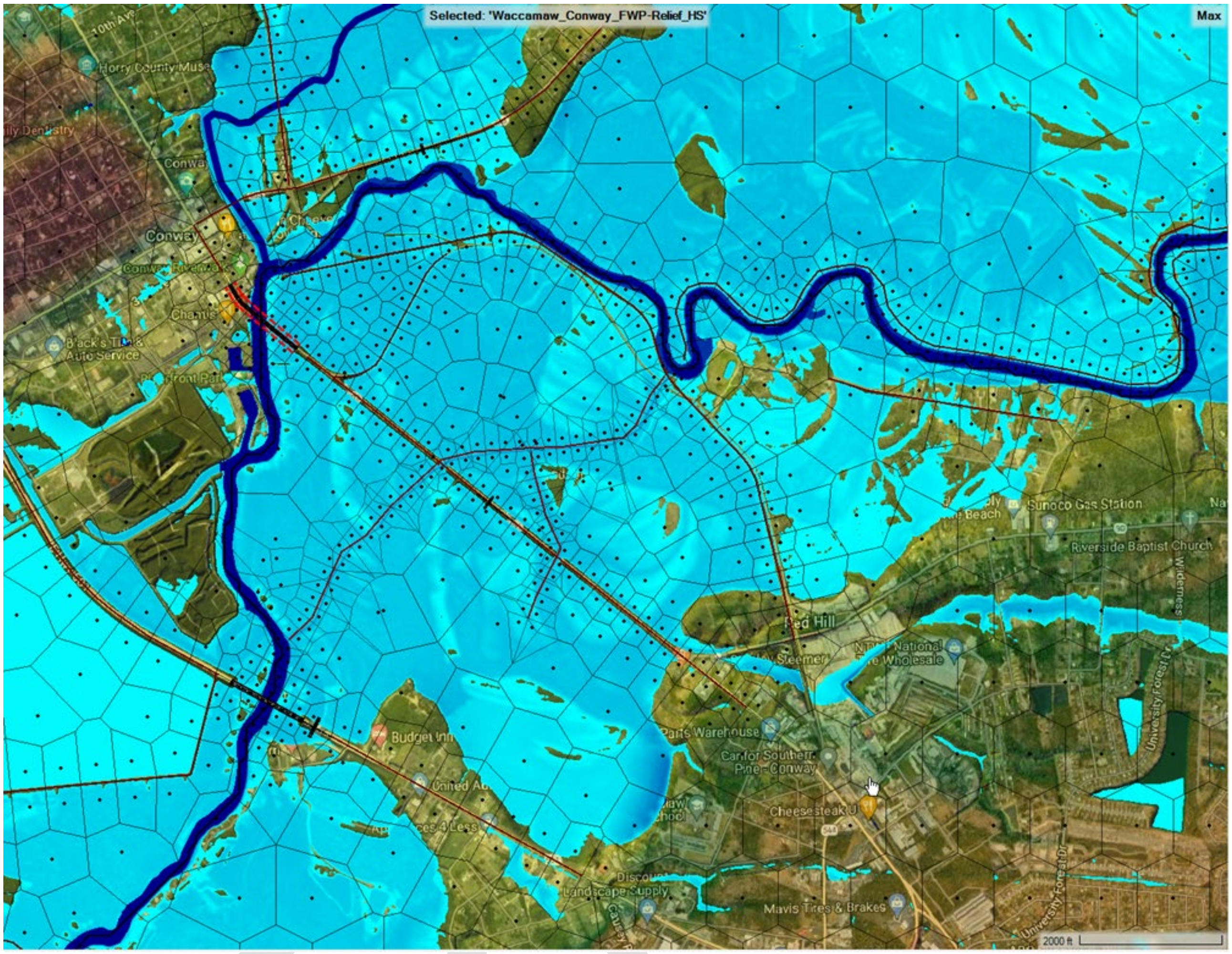


Figure 136. 1% AEP depth of the FWP in Conway after evaluating the measure in HEC RAS

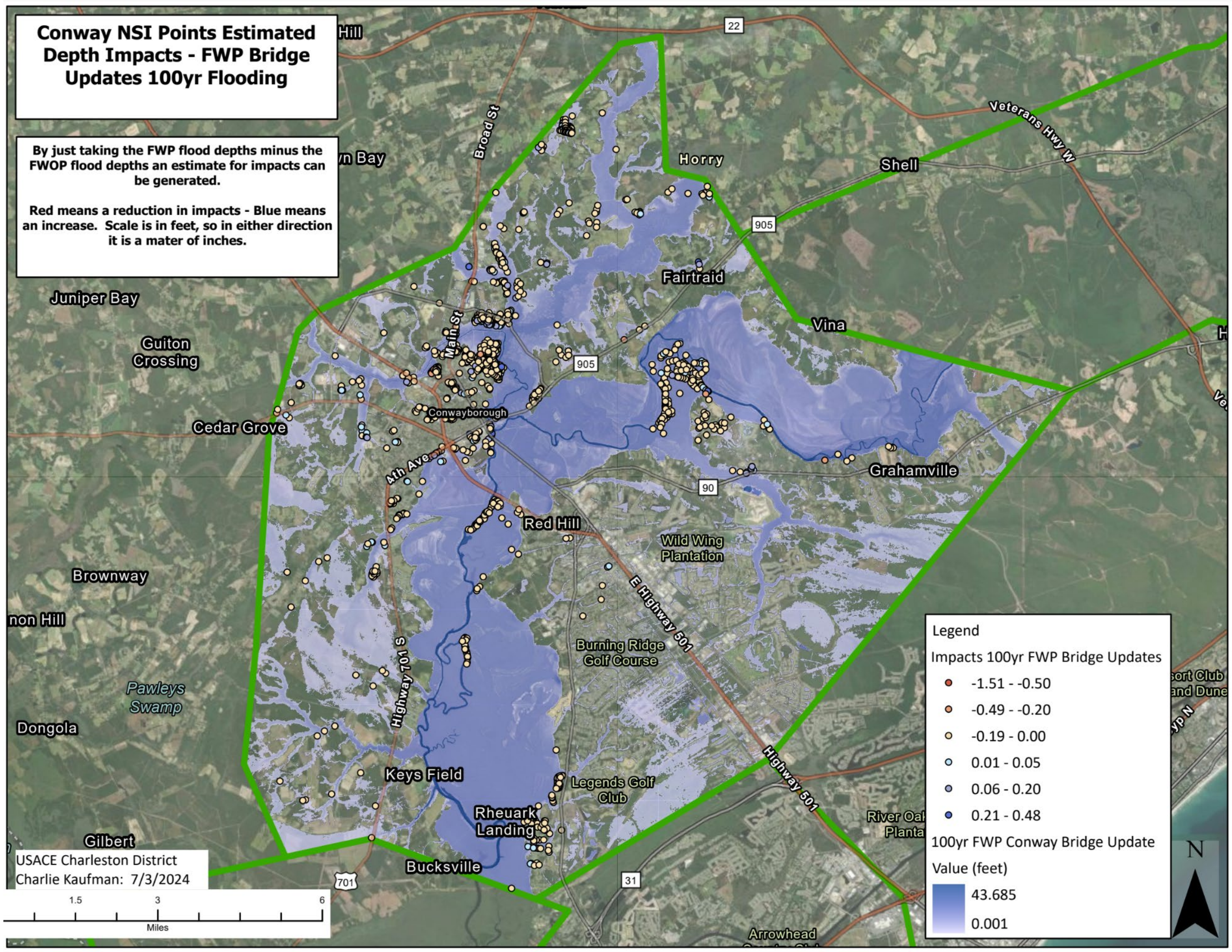


Figure 137: Conway NSI estimated Depth Impacts

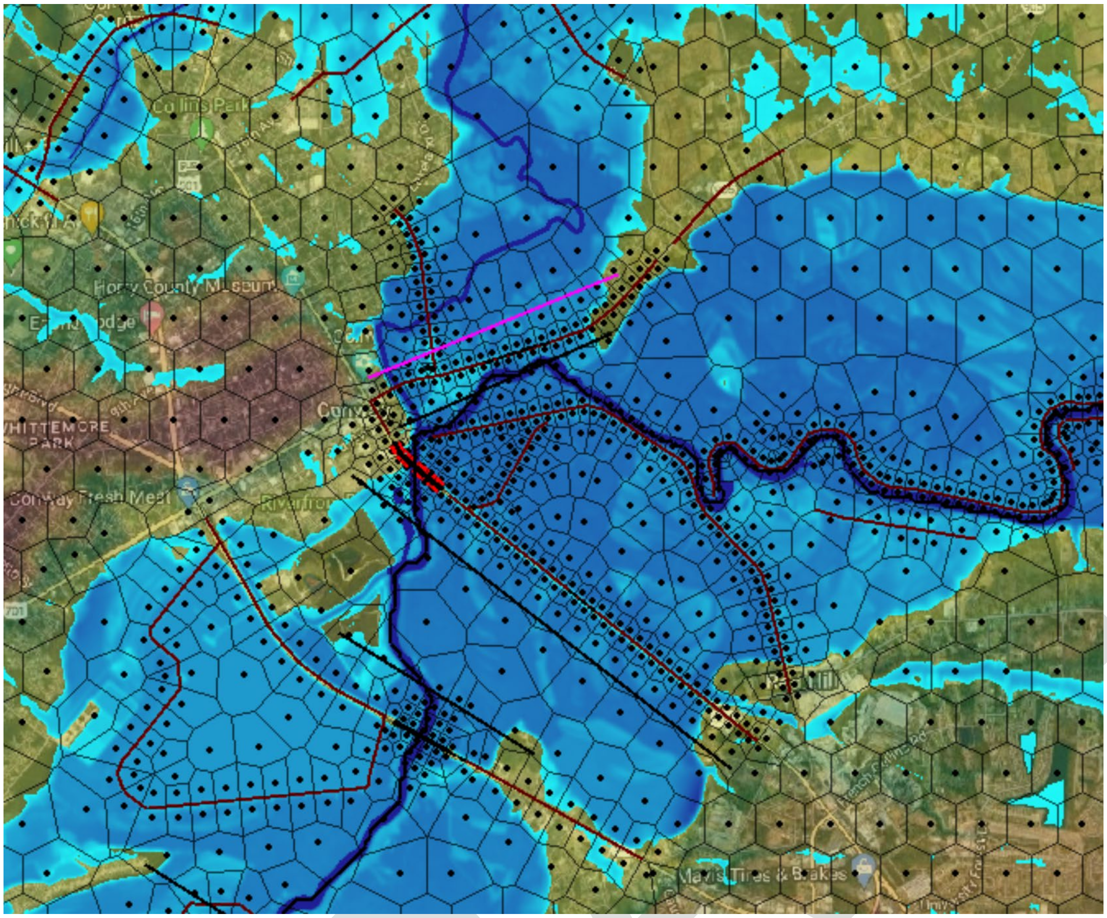


Figure 138: Location of the cross-section water surface elevation comparison upstream of highway 905.

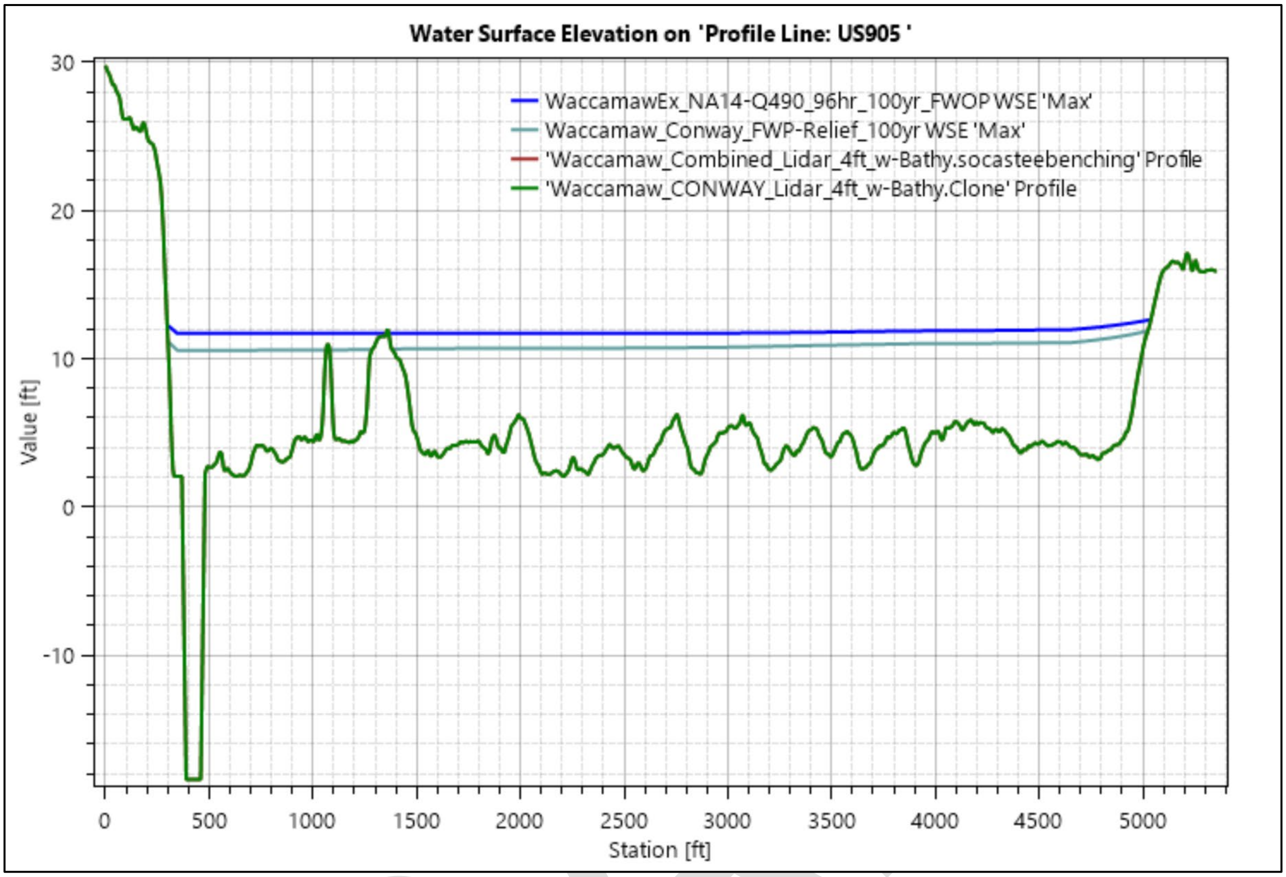


Figure 139: Water Surface Profile cross section comparison for upstream of 905.

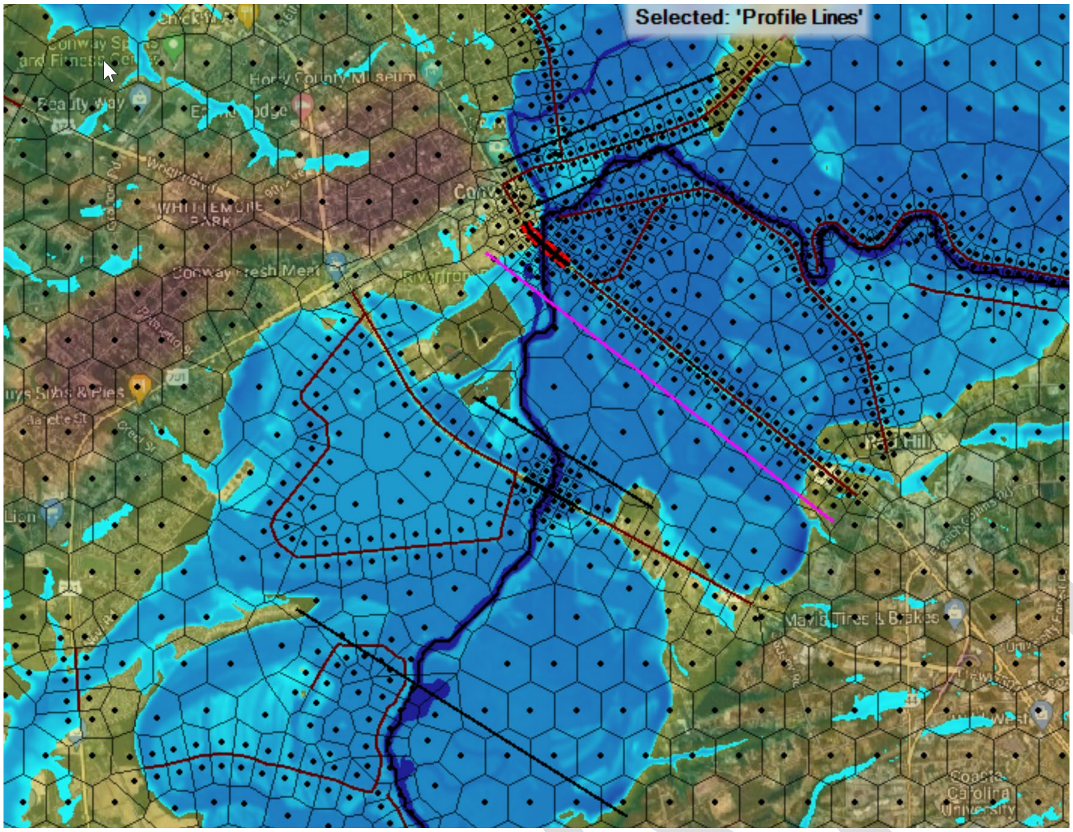


Figure 140:Location of the cross-section water surface elevation comparison upstream of 501.

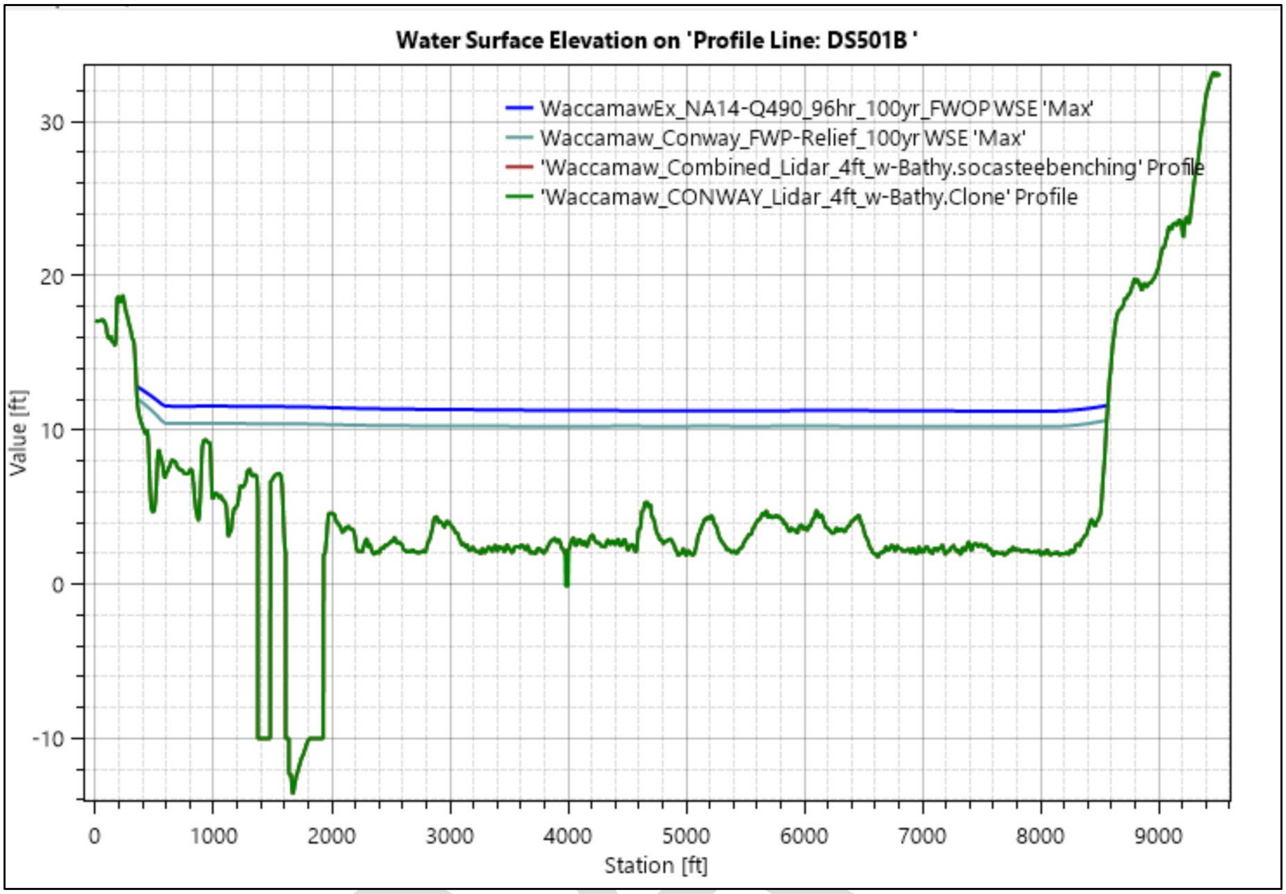


Figure 141: Water Surface Profile cross section comparison for upstream of 501.

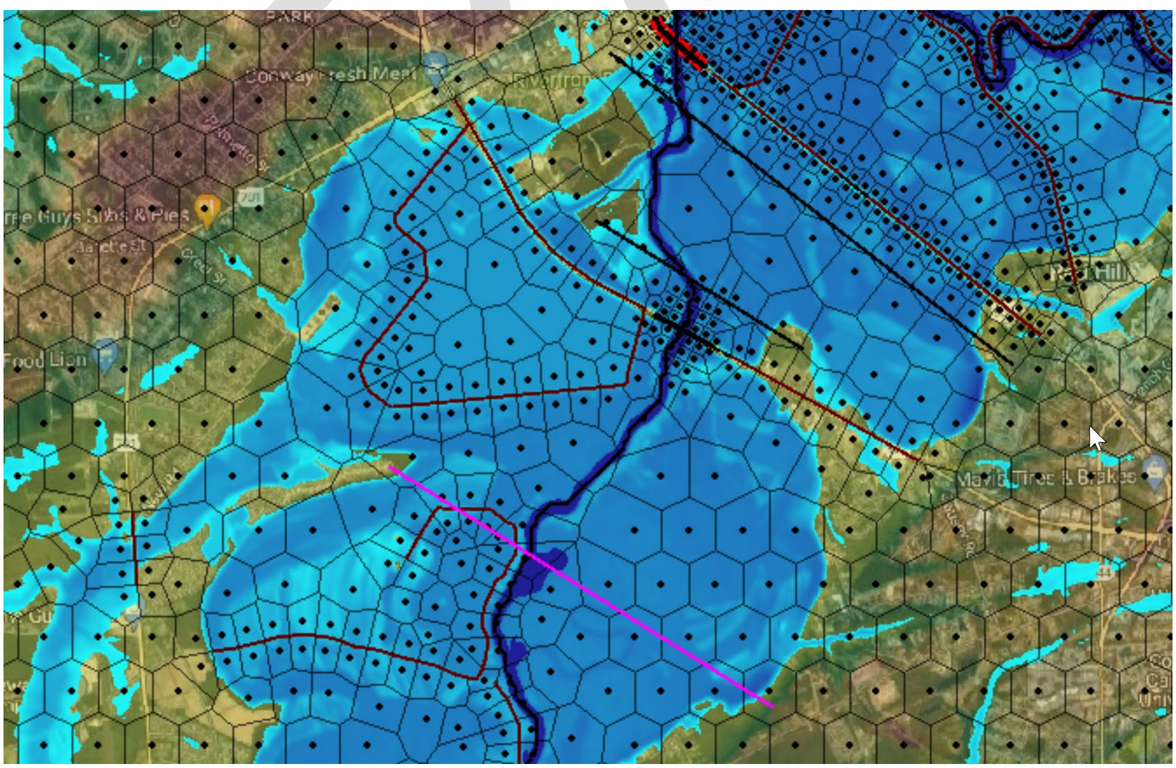


Figure 142: Location of the cross-section water surface elevation comparison downstream of 501.

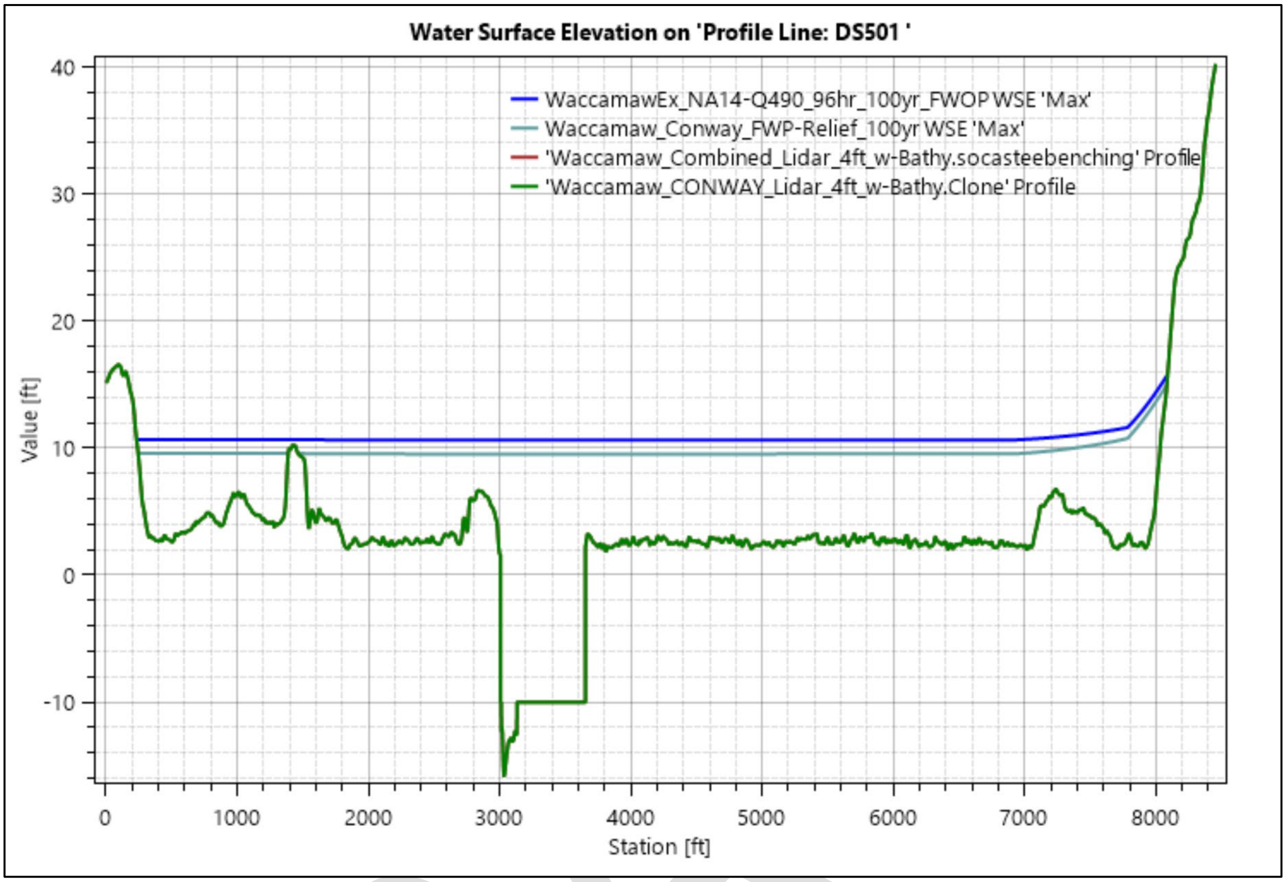


Figure 143: Water Surface Profile cross section comparison for downstream of 501.

Table 31: WSE differential FWOP vs. FWP for Relief Bridges in Conway

Location	Reduction in WSE (ft)
Upstream Highway 905	1.01
Downstream Highway 905	1.16
Upstream Highway 501B	1.18
Downstream Highway 501B	1.09
Upstream Highway 501	0.89
Downstream Highway 501	1.10

Table 30 shows the differential in water surface elevation for cross sections both up and downstream of each relief bridge. Each cross section had a reduction in water surface elevation in excess of 1 foot in most locations up and downstream of the relief bridges. Figure 142 shows the structural locations with increase and decrease in water surface elevation.

8.3.3 Socastee Structural Array of Alternatives

The following structural measures were evaluated for the Socastee Focus area:

- S1 – Floodwall
- S2 – Detention Pond with Channel to Socastee Creek
- S3 – Barrier Removal
- S4 – Floodwall, Barrier Removal, Detention Pond with Channel to Socastee Creek

Socastee is adjacent to the Intracoastal Waterway, approximately four miles east of the confluence with the Waccamaw River. Socastee is an established community that consists of a mixture of older subdivisions from the twentieth century as well as new construction. Socastee is more developed than the other target communities (in the 90th percentile of population density compared to other South Carolina areas) and consists of a mixture of residential neighborhoods and subdivisions, commercial businesses, and public infrastructure, such as schools and churches. The three evaluated structural measures were along the ICW project of Socastee Creek (Figure 144).

Figure 144. Socastee Structural Arrays

Table 31 shows the full array of measures considered for the Socastee Focus Area. The retained measures are the floodwalls, detention/retention and channel, and barrier removal, and the final is all three measures combined. These measures are described in the following sections.

Table 32. Screened and Retained Measures for Socastee Focus Area

Socastee	Screening Rationale
Floodwalls	Retained
Detention/Retention	Retained
Barrier Removal	Retained
Benching	Screened, low effectiveness, environmental impacts

8.3.3.1 S1: Floodwall

Two sheet pile floodwalls along the outer banks of Socastee Creek. Perpendicular to Edwards Burrough Hwy these floodwalls are estimated to be 5-9ft in height; with the right bank extending ~2.3 miles and the left bank extending ~3 miles. From the center line the wall on each side, a perpetual 25-foot-wide easement is required for maintenance, plus a 10-foot-wide temporary easement during construction, totaling 70 feet. Pump stations would be required in conjunction with the flood wall/levee to alleviate interior flooding. These features are positioned, either permanently or temporarily, at the low points along the structure.

The proposed protection is for the Forestbrook community, McCormick and Burcale Rd. A 5-9ft high wall above the existing grade would provide 1% AEP flood protection, and this wall height would vary and tie in at high ground. Construction access and staging may include temporary impacts to private property immediately adjacent to the creek. From the center of the wall on each side, a 25-foot-wide perpetual easement is required for maintenance, plus a 10-foot-wide temporary construction easement.

Floodwalls, while effective at protecting against flooding in urban areas, can have several negative impacts on hydrology and hydrogeology. Floodwalls can disrupt the natural flow patterns of rivers and streams by confining the water within a narrow channel. This alteration can lead to changes in sediment transport, erosion, and deposition downstream. Additionally, it can disrupt the natural migration patterns of aquatic species. The construction of floodwalls has the potential to increase the velocity of water flow along the river or stream, leading to increased erosion of riverbanks and streambeds. This erosion can destabilize the surrounding ecosystem and infrastructure, leading to further damage during flooding events.

Floodwalls can impede the natural exchange of water between surface water bodies and groundwater systems. This reduced interaction can hinder the recharge of groundwater aquifers, which are important sources of drinking water and support for ecosystems. Floodplains serve as natural buffers during flood events by absorbing excess water and reducing flood peaks. Floodwalls can disconnect the floodplain from the main river channel, reducing its ability to absorb and store floodwaters. This loss of connectivity can exacerbate flooding downstream and increase flood risk in surrounding areas. Floodwalls can fragment and isolate wetland ecosystems, disrupting their hydrological connectivity with adjacent water bodies. This fragmentation can degrade wetland habitats, reduce biodiversity, and impair the ecosystem services they provide, such as water filtration and flood control.

In some cases, floodwalls can create backwater effects upstream, where water levels rise higher than they would naturally during flood events. These elevated water levels can inundate surrounding areas that would not have flooded otherwise, leading to unexpected flood impacts and property damage. Overall, while floodwalls can provide protection against flooding in urban areas, their construction and maintenance can have significant negative impacts on hydrology and hydrogeology, as well as on the surrounding ecosystems and communities. It's important for planners and engineers to consider these impacts when designing flood protection infrastructure and to explore alternative approaches that minimize adverse effects on natural systems.

However, in this case, floodwalls have several positive effects on hydrology regarding flood control. Floodwalls help in controlling the flow of water during periods of heavy rainfall or storm surges. By confining the water within specific boundaries, floodwalls reduce the risk of flooding in adjacent areas, protecting communities and infrastructure. Floodwalls channel water flow, directing it away from sensitive areas such as residential neighborhoods or agricultural land. This controlled flow can prevent erosion and sedimentation in waterways, maintaining their ecological health.

Figure 145. Evaluated Structural measure of floodwalls along Socastee Creek in Socastee, with grid refinement

In this situation, containing floodwaters, the floodwall can minimize erosion along riverbanks and coastal areas. This preservation of soil helps maintain the stability of ecosystems and protects against loss of land and property. Floodwalls can prevent contaminants carried by floodwaters from spreading into surrounding areas. By confining the water within defined channels, floodwalls can facilitate the implementation of water treatment measures, leading to improved water quality downstream. The floodwall can be integrated into comprehensive water management systems, allowing for better regulation of water levels in rivers, lakes, and other water bodies. This can help mitigate the impact of both floods and droughts, ensuring a more reliable water supply for various uses.

The floodwall along Socastee protects critical infrastructure such as roads, bridges, and utilities from damage caused by flooding. This safeguarding of infrastructure reduces maintenance costs and minimizes disruptions to transportation and communication networks. By providing a physical barrier against flooding, floodwalls reduce the risk of property damage and loss of life during extreme weather events. This can lead to lower insurance premiums for residents and businesses located in flood-prone areas, as well as greater overall resilience to climate-related hazards. Overall, the implementation of floodwalls can contribute to more sustainable and resilient hydrological systems, benefiting both human communities and the natural environment.

8.3.3.2 S2: Detention Pond

Detention ponds, also known as retention basins or stormwater management ponds, can have several positive impacts on hydrology. One of the primary purposes of detention ponds is to mitigate flooding by temporarily storing excess stormwater runoff during heavy rain events. By slowing down the flow of stormwater and releasing it at a controlled rate, detention ponds help reduce peak flows in downstream watercourses, thereby minimizing the risk of flooding in surrounding areas. Detention ponds serve as effective sedimentation basins, allowing suspended solids and pollutants carried by stormwater runoff to settle out before the water is discharged into receiving water bodies. Additionally, the detention time provided by these ponds facilitates the natural processes of filtration, biological uptake, and chemical transformation, leading to improved water quality downstream.

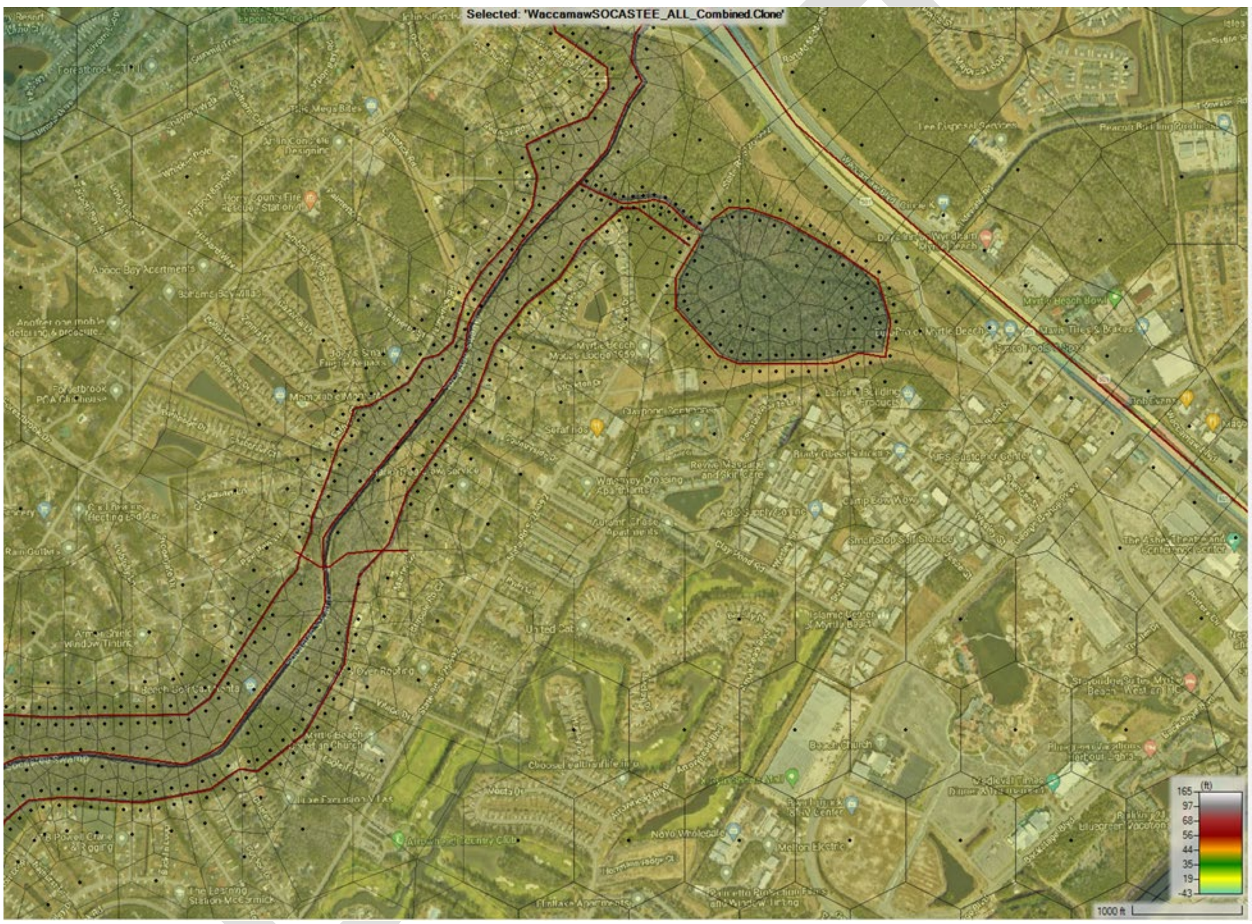


Figure 146. Refinement of the HEC-RAS grid and topography of the detention pond and channel in Socastee

Detention ponds can contribute to the recharge of groundwater aquifers by allowing infiltrated stormwater to percolate into the underlying soil and replenish the groundwater table. This helps maintain baseflow in streams and rivers during dry periods, supports groundwater-dependent ecosystems, and ensures the availability of groundwater resources for drinking water supply and irrigation.

Well-designed detention ponds can function as valuable aquatic habitats, providing shelter, foraging areas, and breeding grounds for various species of aquatic plants and animals. The creation of wetland vegetation within and around detention ponds further enhances habitat diversity and promotes biodiversity, supporting the establishment of resilient ecological communities. Detention ponds can enhance the aesthetic appeal of urban landscapes by incorporating natural features such as native vegetation, walking trails, and wildlife viewing areas. These amenities provide opportunities for passive recreation, such as walking, birdwatching, and nature photography, thereby fostering community engagement with the natural environment and promoting public appreciation for water resources. The presence of detention ponds can help moderate water temperatures in urban environments by providing shading and evaporative cooling effects. This can mitigate the urban heat island effect, reduce thermal pollution in downstream water bodies, and create more favorable conditions for aquatic organisms that are sensitive to temperature fluctuations.

Overall, detention ponds play a crucial role in managing stormwater runoff, improving water quality, enhancing aquatic habitats, and providing recreational opportunities, thereby contributing to the sustainable management of water resources in urban and suburban areas. Effective planning, design, and maintenance are essential to maximize the positive impacts of detention ponds on hydrology and ecosystem health while minimizing potential adverse effects.

The proposed and evaluated pond and channel dimensions are; Pond depth 15ft, 3:1 side slope; Channel bottom width 20ft, 1:1 side slope, 10ft depth; Burcale Pond Cut: 991,864 cu yd; Burcale Channel Cut: 14,094 cu yd. Geotech report from the Fire Station nearby indicated soft to firm fat clays (CH) ranging from 7 to 7.5ft below the surface. Very dense sands encountered at depths 8-10ft below the surface, and interbedded silts, clays, and sands for the remainder of the pond depth.

From the nearby soil report, water was not encountered in the hand auger borings at the time of drilling to a depth of 4 feet below the surface. Water levels within the cone soundings were interpreted from pore pressure readings to range from approximately 3 to 4 feet below the existing ground surface. This site is favorable for the development of shallow perched groundwater conditions due to the clayey upper soils. The cost estimate will need to consider dewatering.

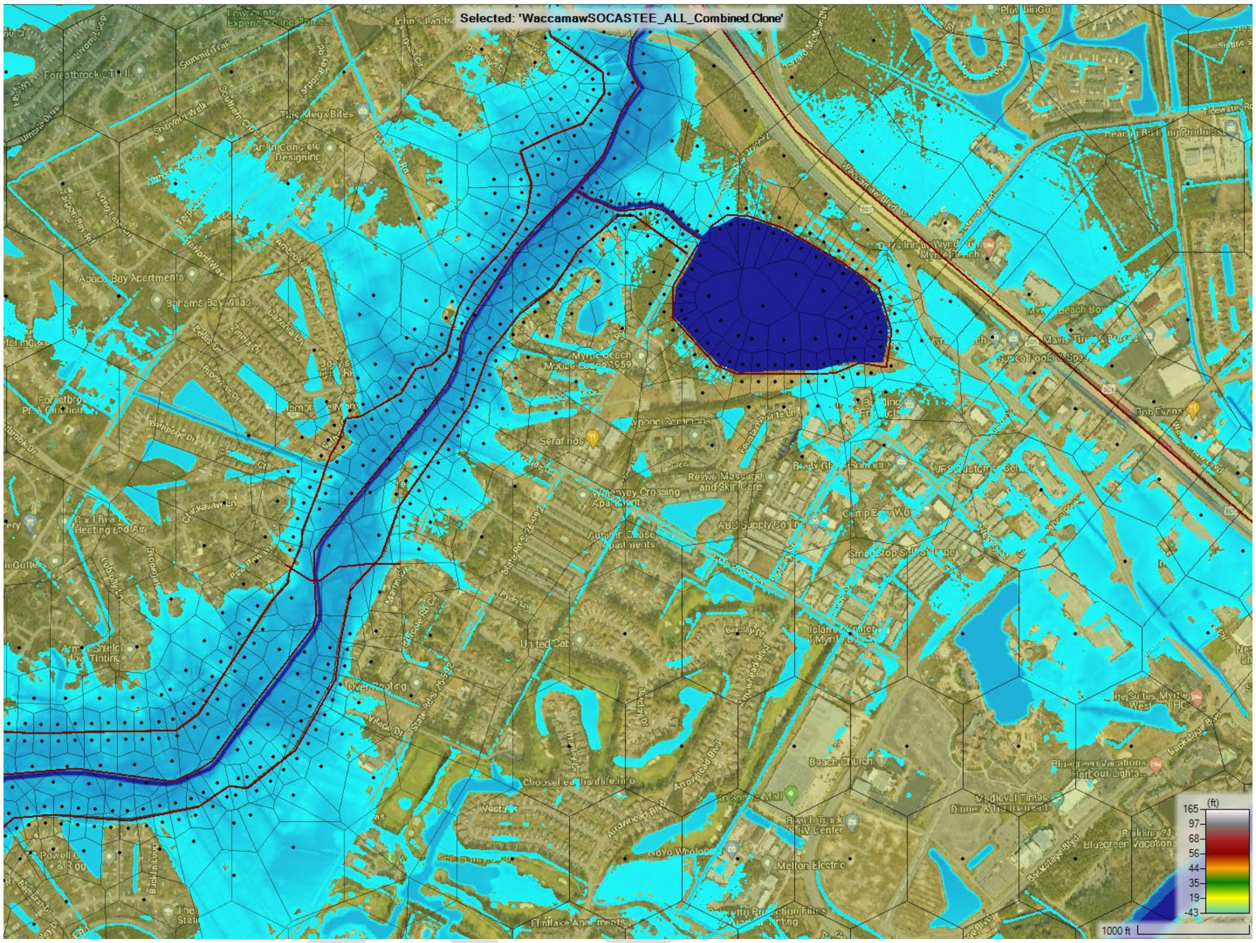


Figure 147. 1% AEP depth with the evaluated measure of detention pond along Socastee Creek

The Detention Pond with Channel to Socastee Creek is proposed on the left bank of Socastee Creek, immediately south of Edward E Burroughs Hwy, a detention pond impounded by levees/flood barriers is proposed. This plan involves occupying up to 55-acres. An existing tributary will be channelized to act as a diversion channel for a passively controlled release into Socastee Creek. Depth of the detention pond is unknown currently. Given the existing stream and lower topography, this plan may include pumps and or gates features to prevent backwater spillage.

The proposed protection is north central Socastee. This area is land locked by Edward E Burrough Hwy, private, and commercial property. Construction and maintenance access may require easements and acquisition. Currently assuming a passive system for water retention and releases. Clearing and dredging are anticipated to develop the detention basin site. Construction activities associated with excavation such as site clearing, fill removal/placement, and restoration are required. Suitable fill material may be repurposed for pond impoundment (requires soil sampling). Environmental impacts associated with habitat modification may apply, including the potential for irreversible conversion of farmland to nonagricultural use.

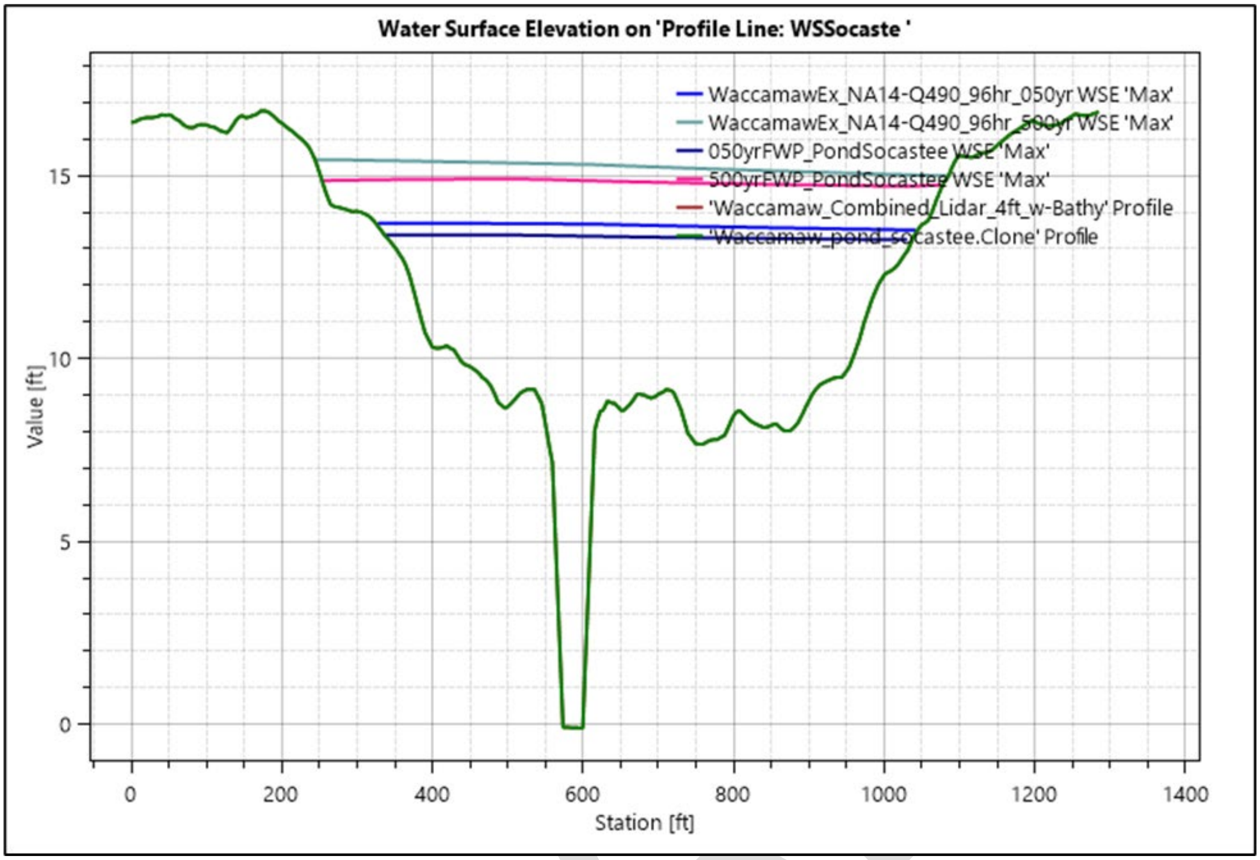


Figure 148. Comparison of Max WSE for FWOP and FWP including the Detention Pond

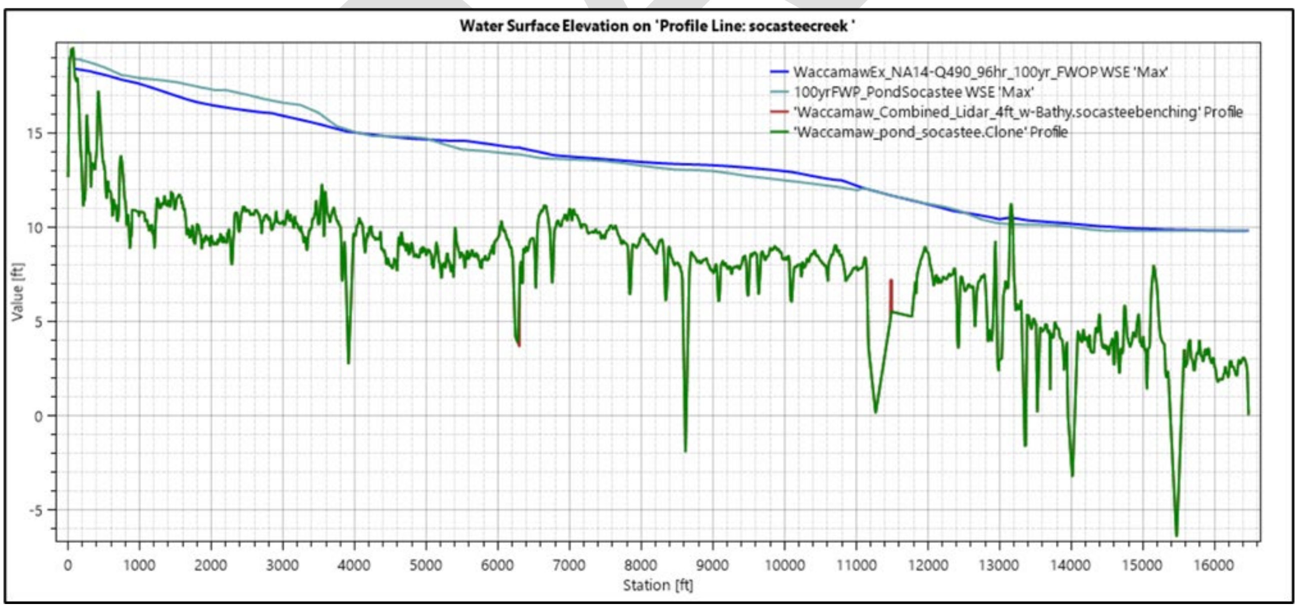


Figure 149. Comparison of the WSE along Socastee Creek FWOP and FWP

8.3.3.3 S3: Weir Removal Socastee

The Socastee Creek Federal Project currently has two existing weirs along Socastee Creek – Both 40ft wide and 10ft high – constructed from concrete and sheet pile. They are protected by a layer of rip-rap 2 ft thick and 50 ft wide on both the upstream and downstream sides. The weirs were designed to maintain the groundwater table as it existed before construction of the weirs to preserve the natural habitat of the study area by mitigating wetland loss. However, increased development in this area means that the natural habitat may not be present as anticipated. Water currently flows around the weirs, eroding the area and causing damage to the weir structures. Removing the weirs would increase conveyance in the adjacent flood impact area. Figure 152 shows the locations of the potential weir removals. The proposed measure is intended to decrease flood elevations at upstream homes along Socastee Creek (Figure 151).

Figure 150. Locations of potential weir removals along Socastee Creek

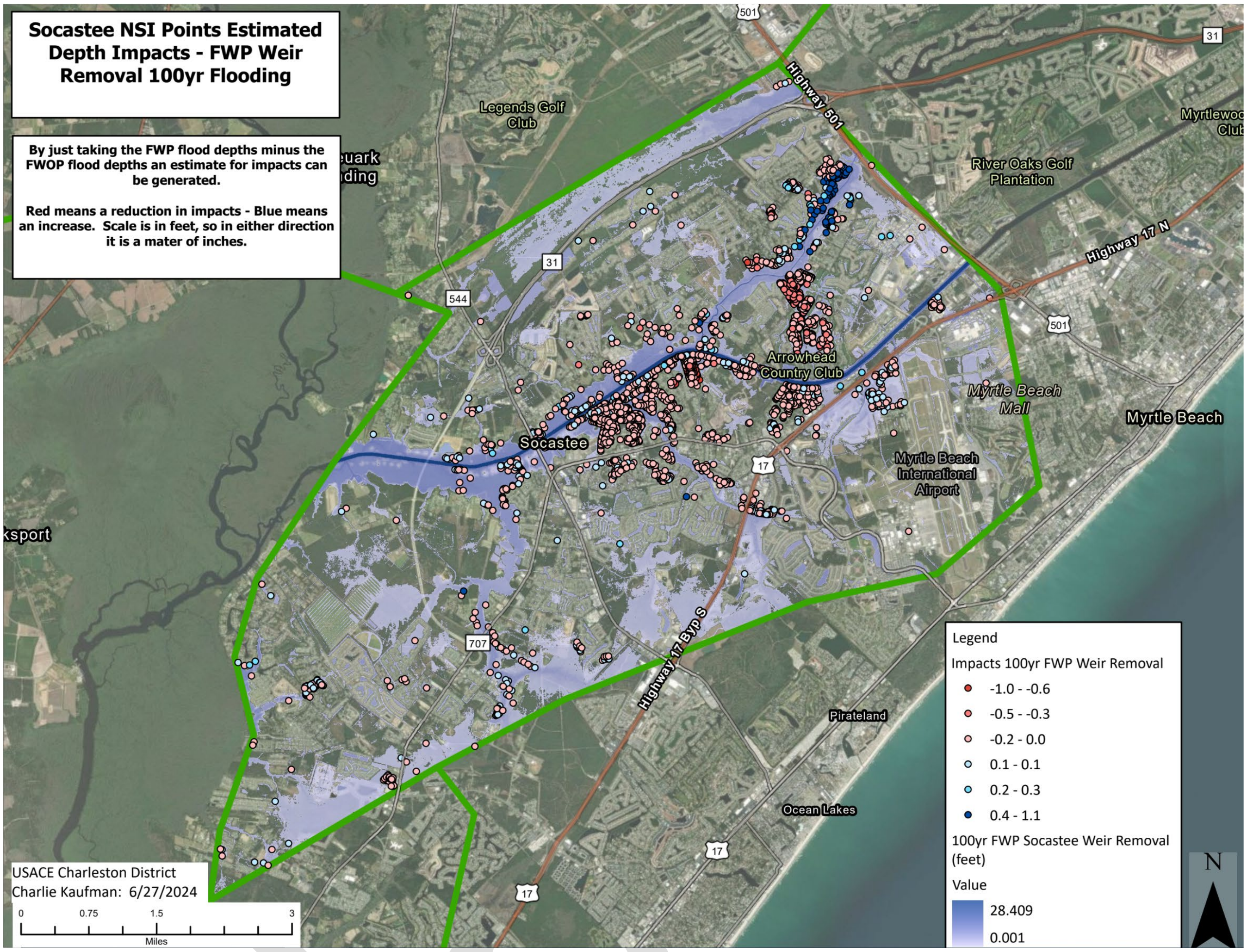


Figure 151. Structures displayed as decreased depths with the inclusion of the weir removal

In addition to increasing conveyance, the removal of weirs can have several positive impacts on hydrology. Weir removal allows the stream or river to return to its natural flow regime, including variations in flow intensity and frequency. This restoration of natural hydrological patterns can benefit aquatic ecosystems by providing suitable habitat conditions for native flora and fauna, promoting nutrient cycling, and supporting biodiversity. Weirs can act as barriers to fish migration, particularly for species that need to move upstream to spawn or access important habitat areas. Removing weirs restores connectivity along the river or stream, allowing fish to freely move between different sections of the watercourse and access essential spawning grounds, nursery areas, and feeding habitats. Weirs can disrupt the natural transport of sediment downstream, leading to sediment accumulation and channel degradation upstream of the structure. Removal of weirs restores the natural sediment transport processes, promoting the movement of sediment through the river or stream system and helping to maintain channel morphology, substrate diversity, and aquatic habitat quality.

Weir removal can lead to the recovery of riparian vegetation along the stream or riverbanks. Without the presence of the weir, natural flooding and erosion processes can occur, creating opportunities for the establishment of native riparian vegetation species. Healthy riparian vegetation provides numerous benefits, including stabilizing stream banks, filtering pollutants, and providing wildlife habitat. Weirs can fragment river and stream networks, reducing hydrological connectivity between upstream and downstream reaches. Removing weirs restores the natural connectivity of the watercourse, allowing water, sediment, and nutrients to flow freely throughout the river system. This improved connectivity can enhance ecosystem resilience, support ecological processes, and facilitate the movement of aquatic organisms. Weir removal can enhance recreational opportunities for activities such as kayaking, canoeing, and fishing. Restoring the natural flow regime and channel morphology of the river or stream can create more diverse and dynamic recreational experiences, attracting visitors and stimulating local tourism economies.

Overall, the removal of weirs can have significant positive impacts on hydrology by promoting ecosystem health, restoring natural processes, and enhancing the ecological and recreational value of river and stream ecosystems. However, it's important to carefully assess the potential consequences and engage stakeholders in the decision-making process to ensure that weir removal projects are implemented effectively and sustainably. Figure 152 depicts the difference in water depths at each structure. The red shading depicts the negative depth, meaning the lowering of the water depth and blue shading is the increase in depth. Upstream of the weirs indicate the increase in water depths and downstream shows a decrease.

Figure 152: Location of the cross sections used in comparison for FWOP vs. FWP weir removal of the WSE

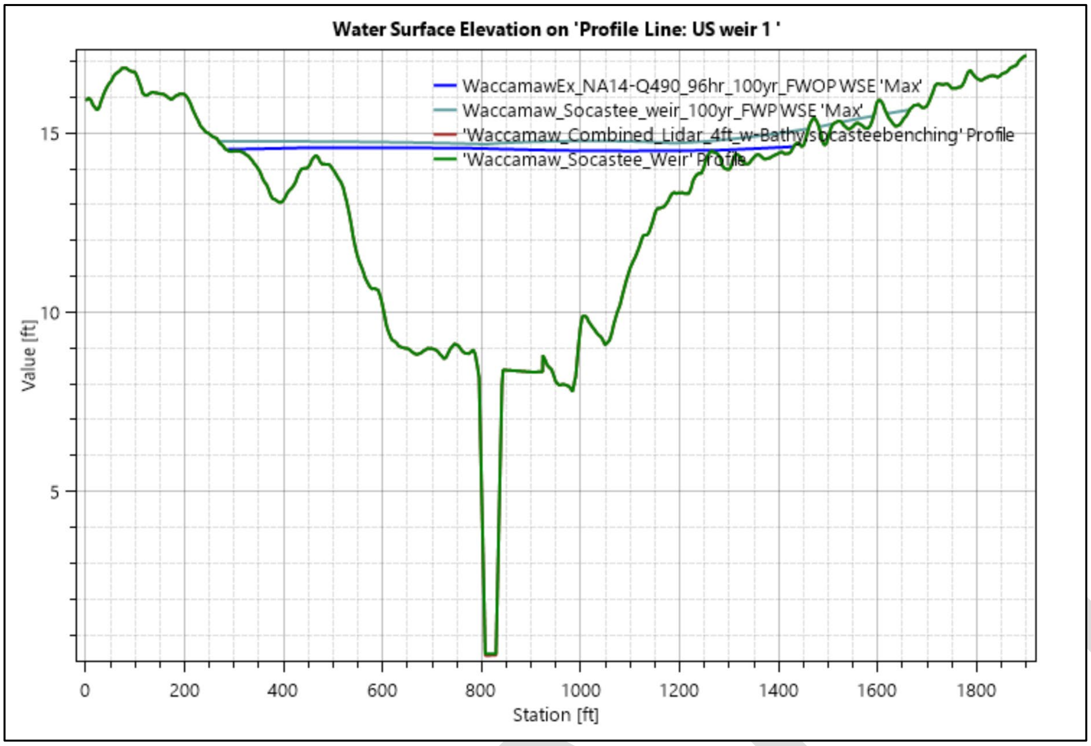


Figure 153: Cross section WSE comparison of the FWOP and FWP weir removal U/S weir 1

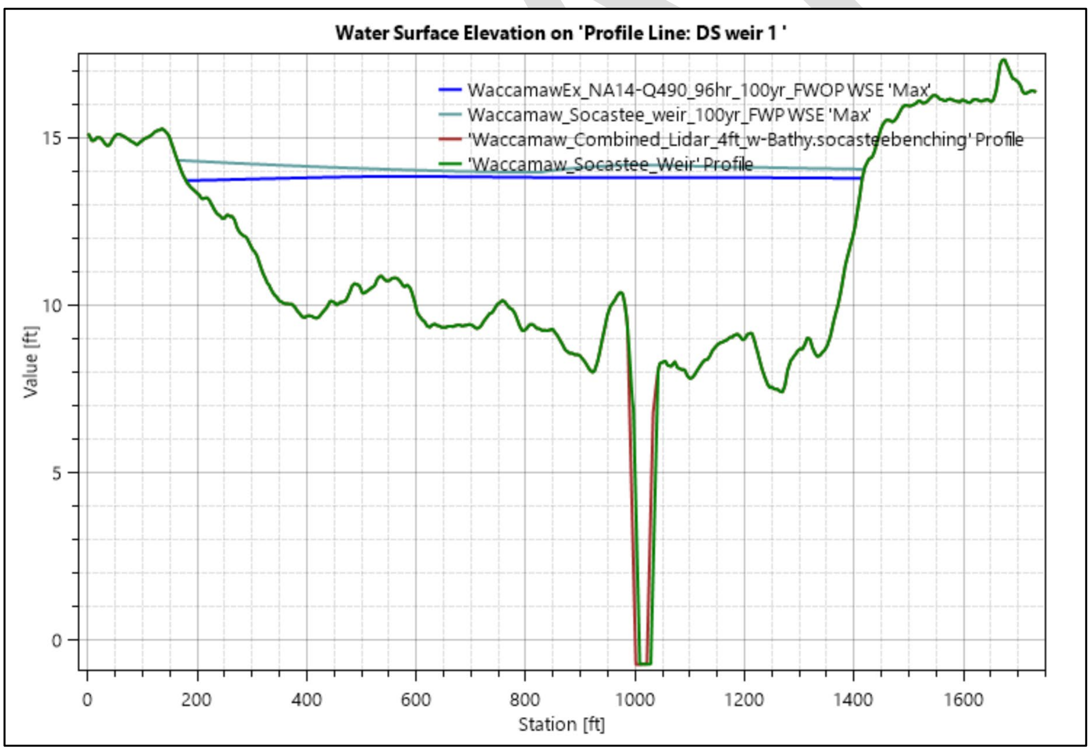


Figure 154:Cross section WSE comparison of the FWOP and FWP weir removal D/S weir 1

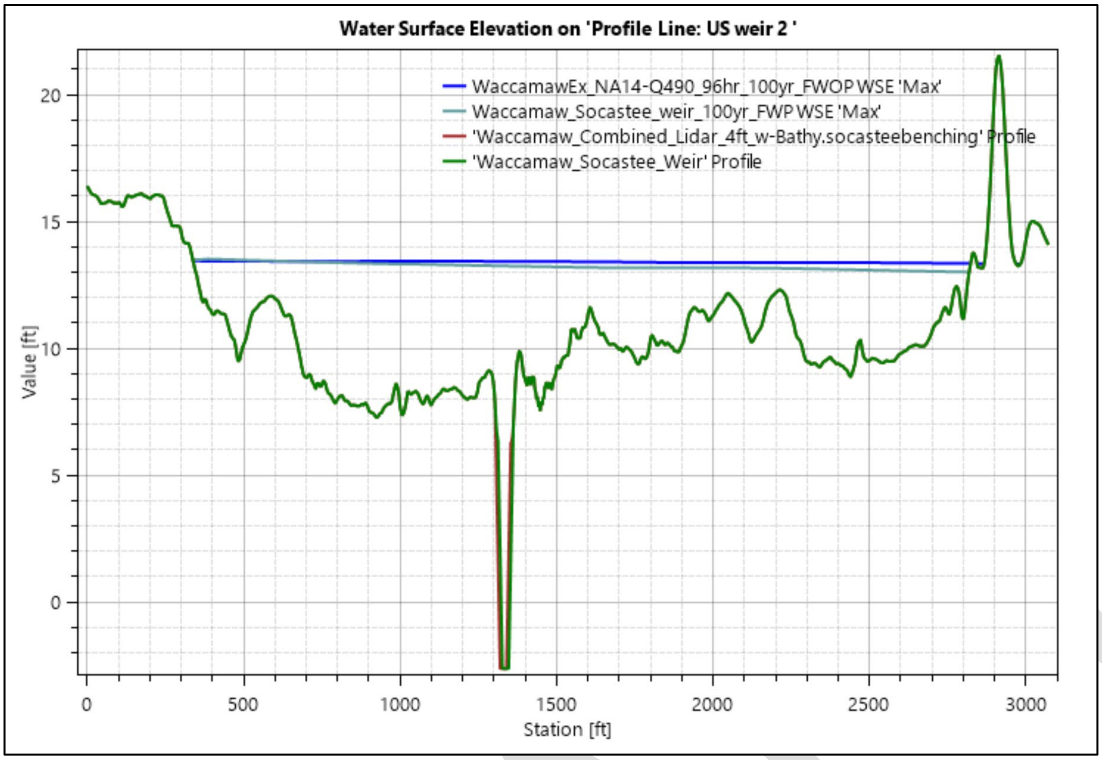


Figure 155: Cross section WSE comparison of the FWOP and FWP weir removal U/S weir 2

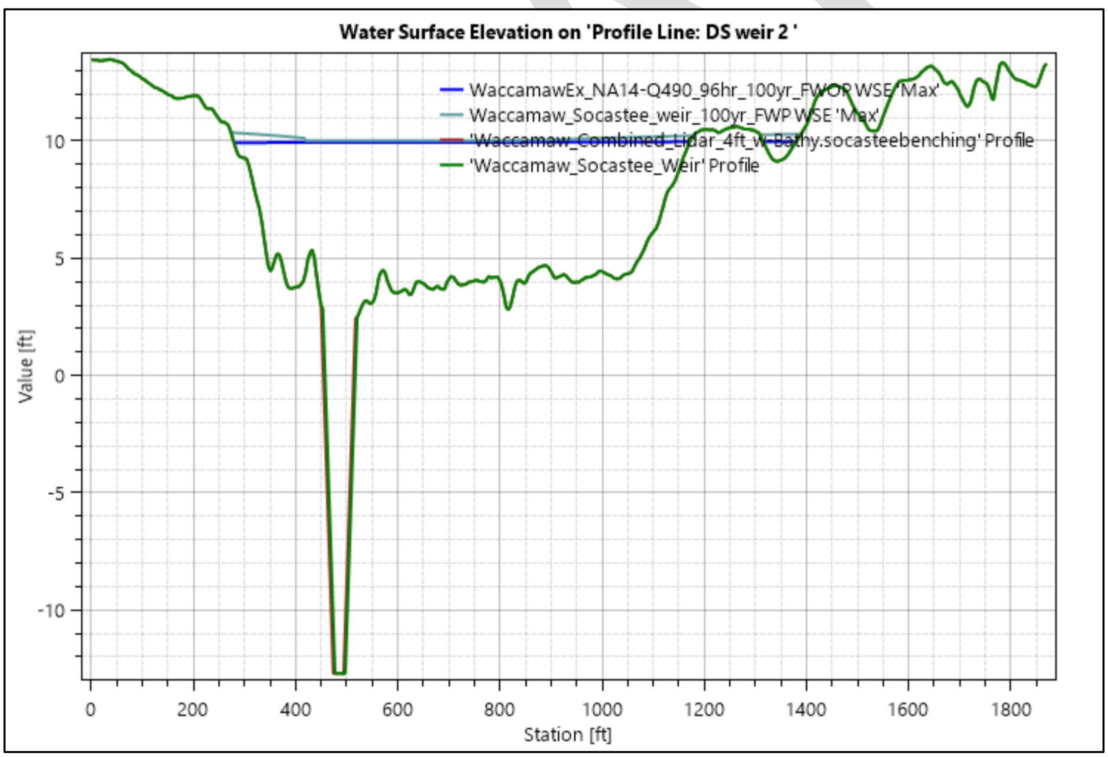


Figure 156: Cross section WSE comparison of the FWOP and FWP weir removal D/S weir 2

Table 33: Differential in WSE with FWOP and FWP weir removal

Cross section location	Differential in WSE (ft)
Upstream Weir 1	-0.22
Downstream Weir 1	-0.27
Upstream Weir 2	0.3
Downstream Weir 2	0.12

Table 33 shows the differential of the water surface elevation at various cross sections both up and downstream of the weirs to be removed for the 1% AEP event. The negative values indicate an increase in WSE and the positive values indicate a decrease in WSE. There is an induction of flooding on the upstream portion of Socastee Creek with the removal of the weirs in the magnitude of approximately 0.25 ft. This induction of flooding is being considered for potential screening of the measure. However, at this time, there is an extensive decrease in water surface elevation further downstream of the weir removals, relieving flooding in properties in excess of 1 ft in some locations.

Some considerations and assumptions are that the floodplain encroachment and pre-construction site clearing pose possible environmental impacts. For removal of the weir a perpetual 25-foot-wide easement is required for maintenance on both sides, plus a 10-foot-wide temporary easement during construction, totaling 70 feet.

8.3.3.4 S4: Floodwall, Detention Pond with Channel to Socastee Creek and Weir Removal

This measure combines measures S1, S2, and S3:

- Install two sheet pile floodwalls along the outer banks of Socastee Creek
- Install a detention pond on the right bank of Socastee Creek, immediately south of Edward E Burroughs Hwy
- Removal of two existing weirs from Socastee Creek Federal Project

All considerations for the previous measures apply to the combined measure. The combined measures did not provide significant enough reductions in water surface elevations across the entire focus area. Therefore, the combined measure will not be pursued and will be screened out moving forward. Figure 157 shows 1% AEP depths for the combined measure.

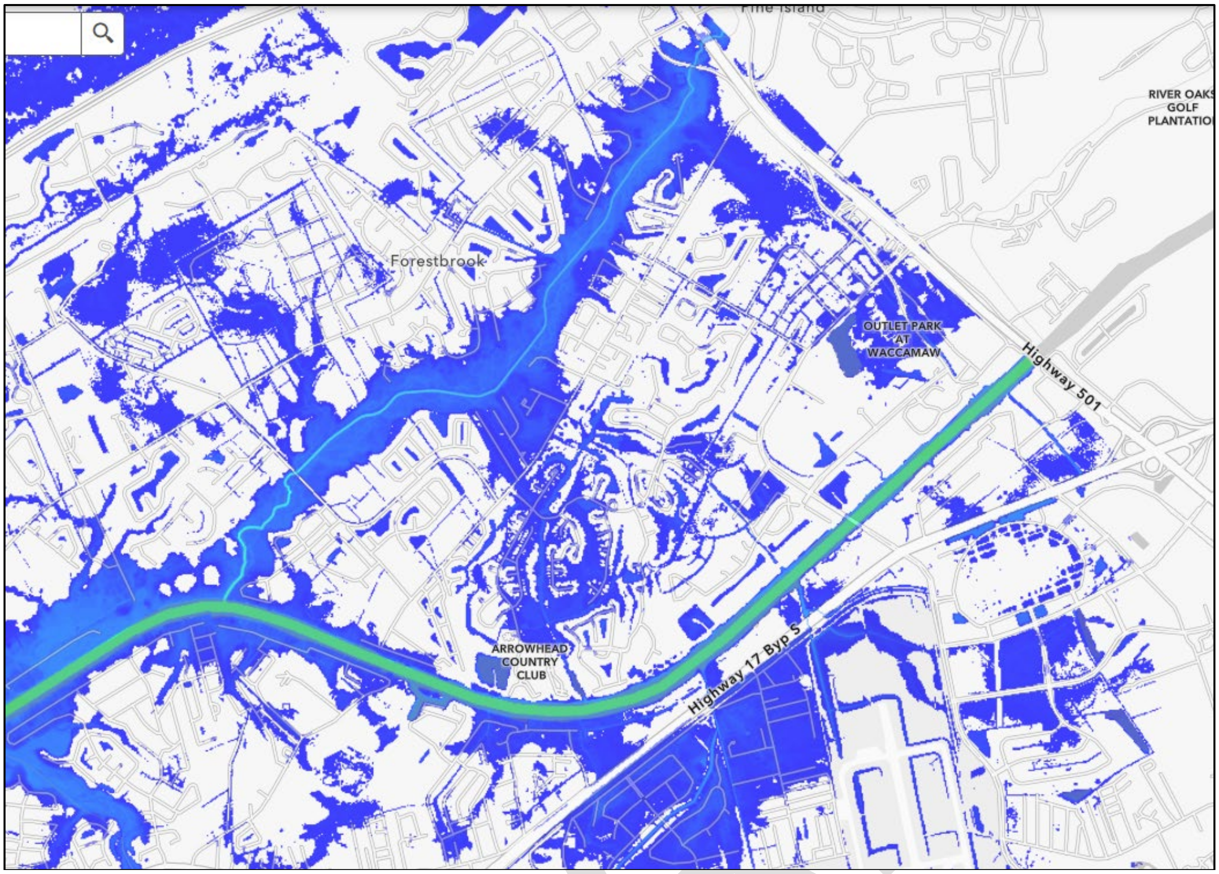


Figure 157. Depth for 1% AEP for the combined measures

8.3.4 Bucksport Structural Array of Alternatives

The following structural measures were evaluated for the Bucksport Focus area:

- B1: Floodgate
- B2: Pee Dee Hwy Elevation

Bucksport is the most downstream focus area community, located in southwestern Horry County and nestled between the Great Pee Dee and Waccamaw Rivers, just to the north and east of their confluence. To the west of Bucksport, these two major rivers are connected by Bull Creek, a former channel of the Great Pee Dee. This community is bordered on three sides by the expansive floodplain and wetlands of the Waccamaw National Wildlife Refuge. Overall, Bucksport is low-lying, particularly in developed areas where elevations rarely exceed 17 feet above sea-level.

This plan involves installation of a floodgate parallel to the confluence of Cowford Swamp and Bull Creek and the road raising of Pee Dee Highway. A floodgate is expected to slow backwater to the Pee Dee River by restricting backflow through Cowford Swamp. The two evaluated structural measures were along Cowford Swamp and Pee Dee Highway, which is pictured in figure 147.

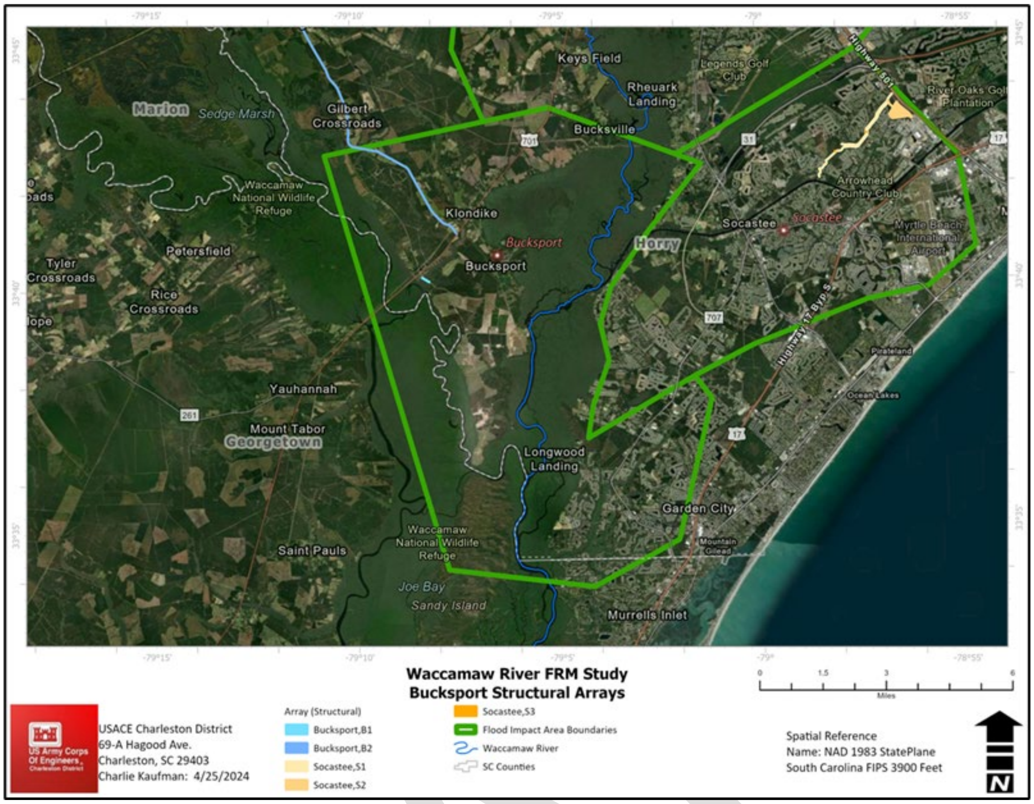


Figure 158. Bucksport evaluated structural measures

Table 33 shows the full array of measures considered for the Bucksport Focus Area. The two retained structural measures are the floodgate and road elevations measures.

Table 34. Screened and Retained Measures for Bucksport Focus Area

Bucksport	Screening Rationale
Floodgate	Retained
Road Elevation	Retained
Elevation	Retained
Acquisition	Retained
Watershed Storage	Screened; Environmental impacts, landowner constraints, agency concerns
Flood Warning System	Screened; Horry County emergency response notification system is up to date, unable to identify improves that would reduce risk

8.3.4.1 B1: Floodgate

Floodgates, which are structures designed to control the flow of water in rivers, canals, and coastal areas, can have several hydrologic impacts. Floodgates are used to regulate the flow of water in rivers and canals, particularly during periods of high-water levels or flooding. By opening or closing the gates, water managers can control the discharge rates, thereby mitigating flood risks downstream or ensuring sufficient water supply for irrigation, navigation, and other purposes. The operation of floodgates can alter the natural flow patterns of rivers and water bodies, leading to changes in water levels, flow velocities, and sediment transport processes. Depending on the design and operation of the floodgates, these alterations can have significant impacts on aquatic ecosystems, including changes in habitat availability, migration routes, and spawning conditions for fish and other aquatic species.

Floodgates may influence sediment dynamics and water quality in rivers and estuaries by trapping or releasing sediment particles during their operation. When floodgates are closed, sediment deposition can occur upstream, leading to channel aggradation and potential impacts on flood conveyance capacity. Conversely, when floodgates are opened, sediment can be flushed downstream, affecting sedimentation patterns, erosion rates, and navigation channels.

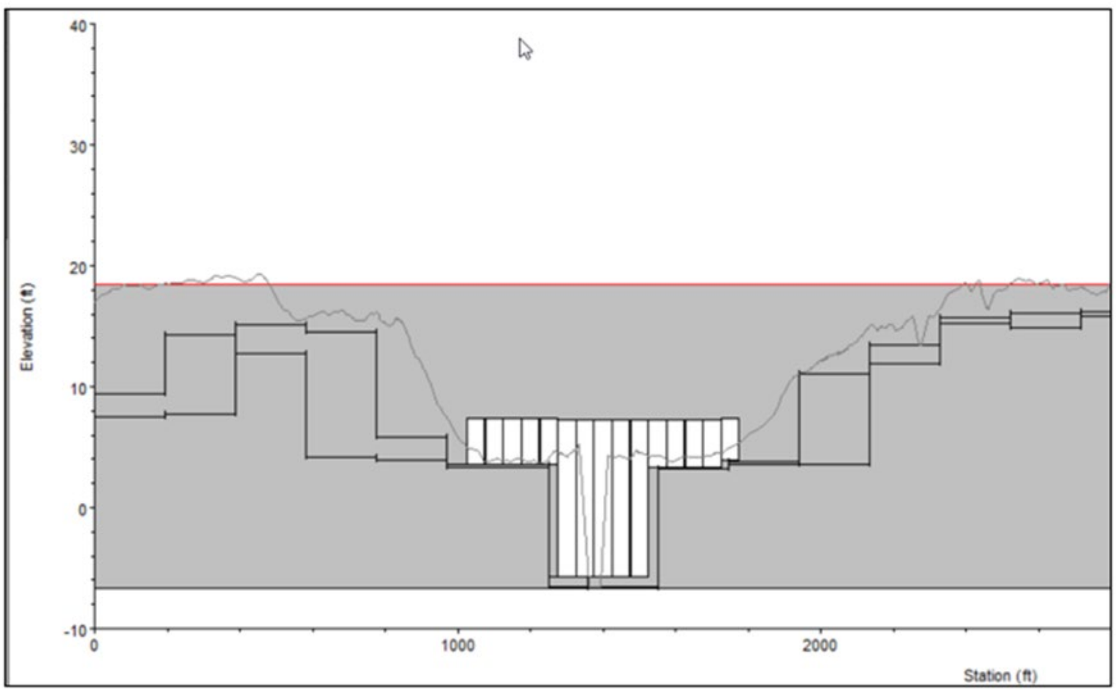


Figure 159. Cross section of geometry input parameters of the floodgate

The operation of floodgates can influence water quality parameters such as temperature, dissolved oxygen levels, nutrient concentrations, and pollutant transport. Changes in flow patterns, residence times, and mixing dynamics resulting from floodgate operation can impact the distribution and fate of contaminants, algae blooms, and other water quality indicators in rivers, estuaries, and coastal waters.

Floodgates can have both positive and negative ecological impacts, depending on their design, operation, and surrounding environmental conditions. While floodgates can provide habitat for certain species, such as wetland birds and aquatic vegetation, they can also disrupt natural hydrological regimes, alter habitat connectivity, and fragment aquatic ecosystems, leading to biodiversity loss and ecological degradation. Floodgates play a crucial role in managing water supply systems by controlling the release of water from reservoirs, impoundments, and diversion structures. By regulating the timing and volume of water releases, floodgates can ensure a reliable water supply for domestic, agricultural, industrial, and municipal uses, as well as for hydropower generation and ecological maintenance.

Overall, floodgates can have significant hydrologic impacts on rivers, estuaries, and coastal areas, influencing flow regimes, sediment dynamics, water quality, ecological processes, and water supply management. It's important to consider these impacts in floodgate design, operation, and management to minimize adverse effects on aquatic ecosystems, water resources, and communities downstream. Additionally, ongoing monitoring and adaptive management are essential to assess and mitigate the hydrological impacts of floodgate operations over time.

The function of that would permit flow from Cowford Swamp to the Pee Dee River, but in anticipation of high-water levels, the gate would be closed. Under normal conditions the flap gate would remain open. Situated between 701 HWY and Big Bull Landing on Marine Park Road, this structure is estimated to be 0.6 miles in length and 13ft above surface water levels. The exact location and footprint remain undefined. From the center line of the gate/wall on each side, a perpetual 25-foot-wide easement is required for maintenance, plus a 10-foot-wide temporary easement during construction, totaling 70 feet.

The proposed areas for protection are the communities on or near Frazier Road, Bucksport Road, and Railroad Drive. Some considerations and assumptions are that the flood stage for the Bucksport USGS gage is 19ft. The floodgate would need to be 6ft above existing water level to protect from the 1% AEP (annual exceedance probability-100year) and more frequent events. Pooling north of the Big Bull Landing is anticipated when the flood gate is closed. Permitting for the Big Bull Landing elevation project has been initiated by Horry County. Stream and floodplain impact to Cowford Swamp and Bull Creek are expected. Proposed to work in conjunction with the Big Bull Landing elevation project. The elevated roadway would require supplemental drainage facilities such as additional gates and pumps to prevent water build up behind the wall when the flood gate is closed.

Figure 160. Location of the Floodgate along Cowford Swamp

8.3.4.2 B2: Pee Dee Highway Road Raising

Elevating Pee Dee Hwy provides reliable access to residences during flooding events and minimizes overflow from the Pee Dee River.

Raising a roadway can have several hydrologic benefits, particularly in areas prone to flooding or waterlogging. Elevating a roadway can facilitate better drainage by allowing water to flow freely underneath, reducing the risk of standing water on the road surface. This helps prevent road damage and improves driving conditions during heavy rain. Raising a roadway above flood-prone levels can mitigate the risk of flooding during heavy rainfall or storm surges. By keeping the road above the water level, transportation routes remain accessible, ensuring continuity in emergency services and facilitating evacuation if necessary.

Elevating a roadway can help maintain natural drainage patterns by allowing water to flow unimpeded beneath the road. This prevents the disruption of natural watercourses and reduces the need for extensive artificial drainage systems, which can be costly to install and maintain. Elevating a roadway minimizes direct contact between road runoff and nearby water bodies, reducing the risk of water pollution. This helps protect aquatic ecosystems by preserving water quality and minimizing habitat degradation. Raising a roadway can enhance its resilience to future climate change impacts, such as sea-level rise and increased precipitation. By elevating critical transportation infrastructure, communities can better adapt to changing hydrological conditions and reduce the risk of costly damage from extreme weather events.

Overall, raising a roadway can provide significant hydrologic benefits by improving drainage, reducing flooding, preserving natural watercourses, protecting aquatic ecosystems, and enhancing long-term resilience to climate change. Currently the Pee Dee Hwy has significant low points along the highway that allow flood water to overflow and cover the road, preventing ingress and egress during flood events. This plan involves elevating approximately 7 miles of the Pee Dee Hwy, starting at US 701 Hwy and terminating at Pauley Swamp Road. To reduce flood risk for a 1% AEP event the Pee Dee Hwy would need to be raised by 3-7ft (existing road elevation varies). Auxiliary drainage features to minimize pooling east of the roadway may be required. The protected area includes the eastern side of the Pee Dee Highway in Bucksport.

Some considerations and assumptions are that the flow over from the Pee Dee River is a major source of flooding in Bucksport. The downstream area of the Pee Dee Highway often floods in storms above 4% AEP. The current elevation of the Pee Dee Hwy ranges from 15-19 ft NAD27. The raising might require drainage to prevent water build up behind the highway. Environmental impacts may include altered hydrology and floodplain dynamics upstream and downstream of highway.

Figure 161. Identification of the Road raising in Bucksport

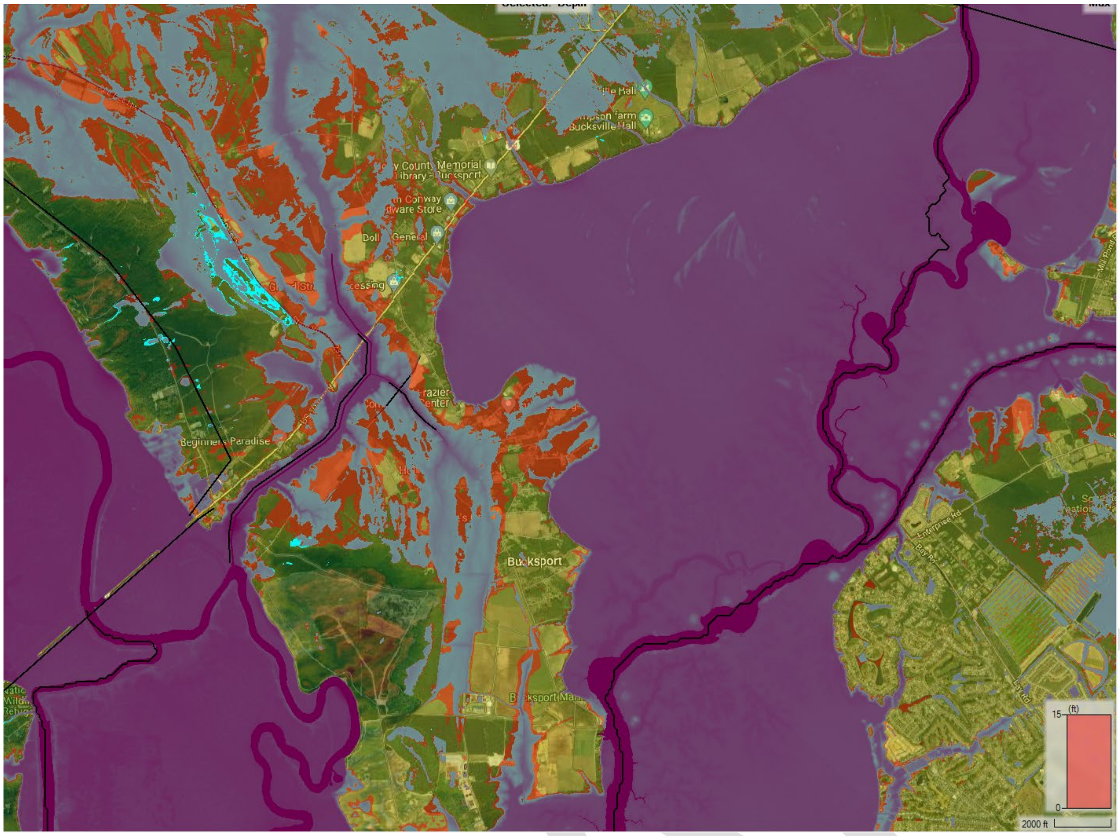


Figure 162. Combined measures in Bucksport showing the depth FWOP and FWP

A few measures screened in this section were located in the tidally influenced coastal area of the Waccamaw River basin. Upon partial plan formulation completion and engineer analyses, the ability to fully capture the complex combination of riverine and coastal influences in driving flood damages was weighed against the constraints of the original allotted time and effort for the Waccamaw River basin study. In-depth, compound event analysis is not warranted because coastal hazards from hurricanes and extreme extratropical storms can include storm surge, waves, wind, rainfall, compound coastal-inland flooding, and extreme tides, among others. Climate change and sea level rise are expected to significantly exacerbate coastal flooding in the upcoming decades. These coastal hazards can threaten the lives of millions of people living in coastal regions, and devastate coastal communities and infrastructure, resulting in profound adverse social, economic, and environmental impacts. The Waccamaw portion of Horry County was not significantly impacted by coastal effects, however appropriate coastal modeling tools would be required in a separate study to adequately formulate for alternatives in this tidally influenced area with sufficient technical details pursuant to USACE 3x3x3 study guidelines further downstream.

8.4 Green Infrastructure and Floodplain Restoration

The inclusion of these measures was predicated on the successful application of more traditional FRM measures (ex. channel modification, bridge modification, etc.). Historically, for these types of measures economic benefits are not as direct, and their intended outcomes can carry more uncertainty due to their limited implementation throughout the USACE FRM portfolio, especially for non-coastal FRM. Ultimately, it was decided that if traditional measures produced a healthy benefit-to-cost ratio, some of that could be absorbed to allow implementation of a more natural and nature-based measure. Therefore, consideration and evaluation of viability for these nature-based measures were assumed to take place during measure refinement, once there is a higher degree of confidence in their successful implementation. If a structural project's benefit-to-cost ratio was slightly below unity, nature-based measures would still be pursued. However, if ratios were well below 1.0 for more traditional measures, these nature-based measures would also be screened from further consideration.

8.5 Refined Structural Alternatives

Upon completion of FWP economic analysis for the preliminary alternatives, it was determined that only two of the 13 structural alternatives produced a benefit-to-cost ratio above 1.0. Specifically, overall perceived damages under FWOP conditions revealed significant challenges in the ability for structural measure refinement to cause an alternative plan to reach a benefit-to-cost ratio of 1.0. The two measures were the relief bridges in Conway and weir removal in Socastee. The two measures are standalone and do not need to be incorporated together in order to get the positive BCR. The BCR for the Relief Bridges was 5.48 and the Weir removal was 10.67.

9.0 Flood Risk Management Uncertainty

9.1 Background

The following description of uncertainty related to FRM was developed by the USACE Kansas City (NWK) and South Atlantic Mobile (SAM) districts as part of a recent FRM feasibility study (SAM, 2021) and the Neuse River Basin FRM Study. While the NWK study area was significantly smaller than that of the Waccamaw River FRM study, the Neuse River was similar in size, and the primary drivers of uncertainty are similar for all.

There are many sources of uncertainty contributing to the analyses involved in flood risk management studies. Fuguitt and Wilcox (1999) distinguish between the two types of uncertainty: future unknowns and data inaccuracy/measurement error. Future unknowns, in the case of this study, may be encountered in forecasting future watershed development, future storm water management, meteorology supporting synthetic storm development, or the effect of climate change on local hydrology. Measurement uncertainty may be encountered in supporting data (i.e., topography) and model calibrations, whereby error may be associated with reported data (i.e., stage and discharge). As flood risk management analyses deal with natural systems, the frequency and severity of risk drivers warranting investigation are most often random. Flood events can be examined as the results of a meteorological risk-driver, basin development, storm water management practices, and hydraulic characteristics. In the area of study, the meteorological risk driver is considered heavy rainfall produced from frontal or dissipating tropical events. Both, the frequency and severity of the risk driver and its response (flooding in this case) have associated uncertainties.

Previous methods of accounting for the consideration of uncertainty (and associated risk) included freeboard and safety factor application, over-designing, and analyzing long-term performance (USACE, 1996a). In response to such practice, USACE developed a risk-based analysis approach to flood risk analyses by analytically incorporating the consideration of risk and uncertainty in evaluations and decision making (USACE, 1996b). In practice these considerations are made through modeling flood damages with the Hydrologic Engineering Center's Flood Damage Analysis (HEC-FDA) system, whereby expected probability distributions for critical study decision tools are developed from extensive sample-testing. The use of HECFDA to assess damage-frequency in combination with calibrated hydraulic inputs works to reduce uncertainties associated with flood risk analyses and overall plan performance.

9.2 Frequency and Stage-Discharge Uncertainty

In accordance with EM 1110-2-1619, Risk-Based Analysis for Flood Damage Reduction Studies, uncertainties pertaining to frequency-discharge and stage-discharge were described using methodologies provided in Chapters 4 and 5 of the referenced EM. Estimation of frequency-discharge uncertainty was based on equivalent record lengths, as provided in Table 4-5 of EM 1110-2-1619. Table 34 shows equivalent record lengths for selected rivers in the Waccamaw basin.

Table 35. Equivalent Record Lengths

Hydrologic Study Model	Equivalent Record Length (yr)
Waccamaw River Mainstem	30
Buck Creek	20
Pee Dee River	30

Stage-discharge uncertainty was assessed by methods provided in Chapter 5 of EM 1110-2-1619. Standard deviations of hydraulic roughness coefficients used in the study models were determined from reference Figure 5-4 in Figure 163 below.

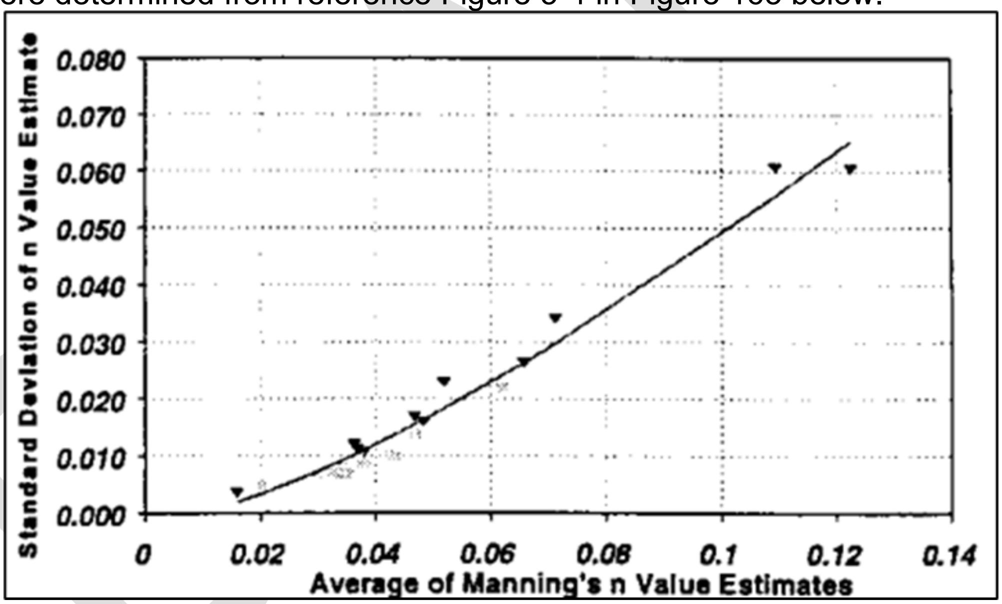


Figure 163. EM 1110-2-1619 Figure 5-4

Each unique Manning's N value within the HEC-RAS models was plotted along the x- axis and a standard deviation value was extracted from a Microsoft Excel trendline equation fitted to Figure 163. This resulted in up to roughly 30 unique standard deviation values for the larger Waccamaw River mainstem model which ranged from 0.013 to 0.121. A series of sensitivity analyses was then performed for each of the hydraulic models to generate upper and lower limit water stages based on the minimum and maximum standard deviation value applied to every Manning's N value. EM 1110-2-1619, Equation 5-7 was used to initially calculate the model uncertainty for each HEC-RAS reach and then averaged such that each HEC-FDA reach was assigned a specific model uncertainty value (S_{model}) in feet. The calculated S_{model} was then compared against the minimum standard deviation of error in stage within EM 1110-2-1619, Table 5-2.

Natural uncertainty ($S_{natural}$) was calculated partially based on the presence of representative streamflow gages within specific HEC-FDA reaches. The general standard deviation equation was used with data from USGS field measurements plotted against a fitted trendline in Microsoft Excel. Due the broad scale and number of separable study models, not all reaches possessed useable streamflow gages, therefore, $S_{natural}$ was also based on Equation 5-5 of EM 1110-2-1619 for study model reaches that lacked said gages. Final total uncertainty (S_{total}) was the summation of model uncertainty (Smodel) plus natural uncertainty ($S_{natural}$). A total uncertainty value was calculated for each HEC-FDA reach, represented by the 0.01-AEP event. For design events more frequent than 0.01, total uncertainty was based on the ratio of peak discharge to the 0.01-AEP. For design events less frequent than 0.01, total uncertainty was held constant. Total uncertainty values per HEC-FDA reach for the 0.01-AEP event across all focus areas are shown in Figure 164 through 167.

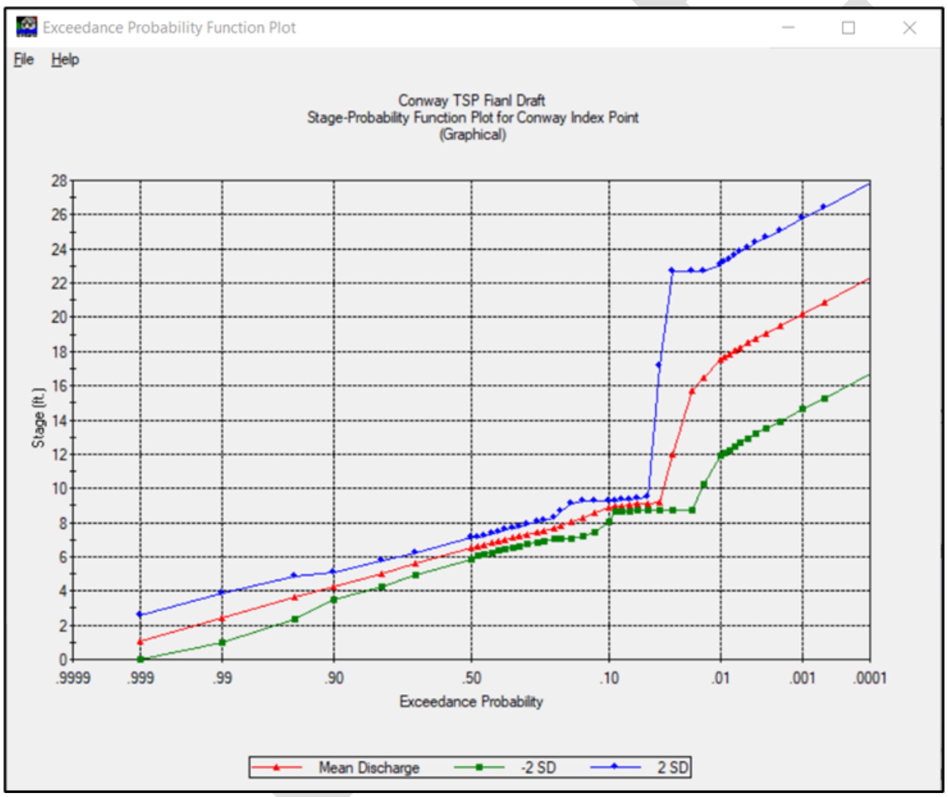


Figure 164. Stage Probability function for Conway

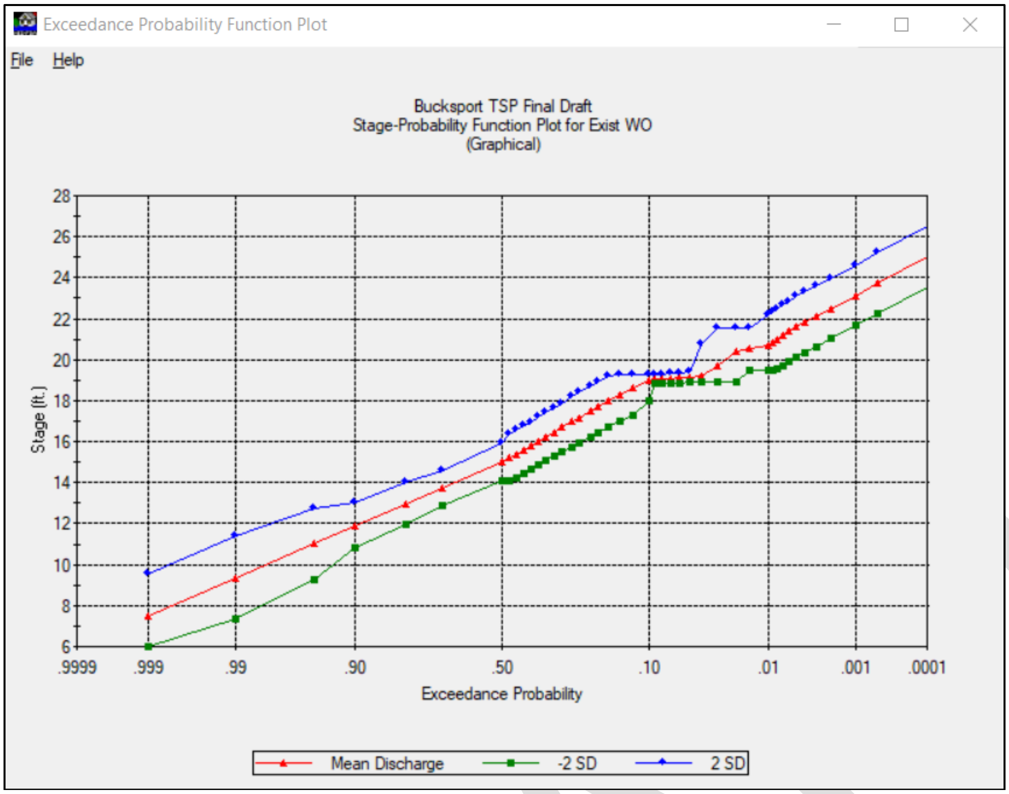


Figure 165. Stage Probability function for Bucksport

Figure 166. Stage Probability function for Socastee

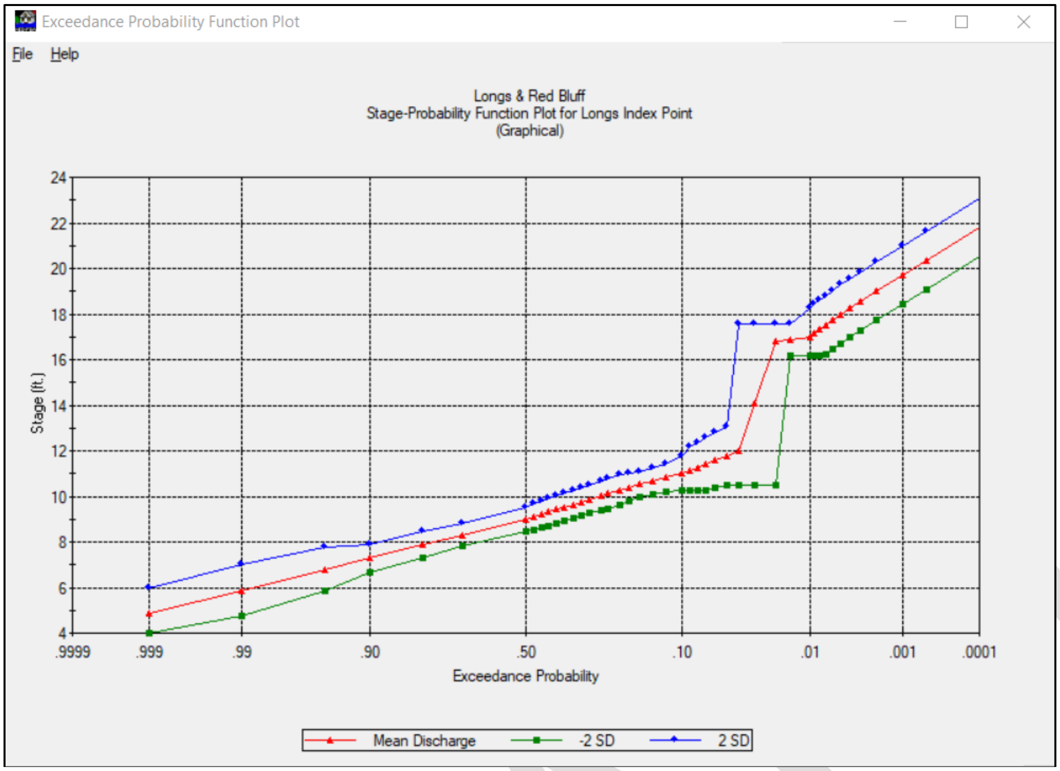


Figure 167. Stage Probability function for Longs/Red Bluff

Reference the Economics Appendix for the uncertainty assessed for the hydraulic and economic modeling.

10.0 Summary and Conclusions

10.1 Observed Summary and Conclusions

The proposed structural measures were evaluated using HEC RAS and were compared to the FWOP model, both hydraulically and economically. In Longs, the evaluated structural measures were a floodwall along Buck Creek and benching along Simpson Creek. In Conway, three relief bridges at the three major bridges were evaluated. A floodwall, detention/retention pond and removal of weirs were evaluated in Socastee. And Pee Dee Highway road raising and floodgate were proposed for Bucksport. Each focus area also evaluated the implementation of nonstructural measures acquisition and elevation of homes. Overall, the implementations of the evaluated structural measures lowered the water surface elevations in some locations near and around the weir locations. There was a reduction of water in homes, but most were not fully removed from the inundation area.

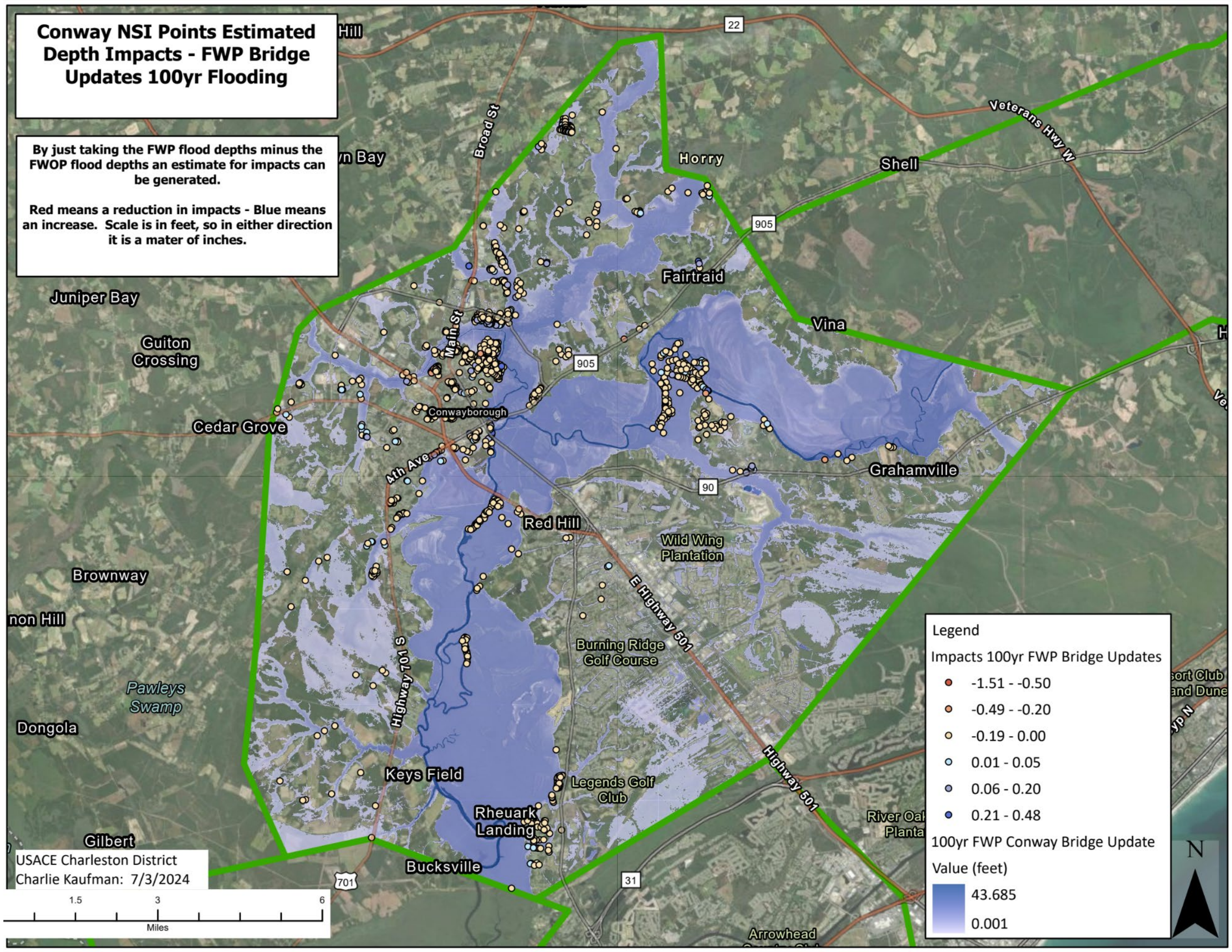


Figure 168: Conway NSI estimated Depth Impacts

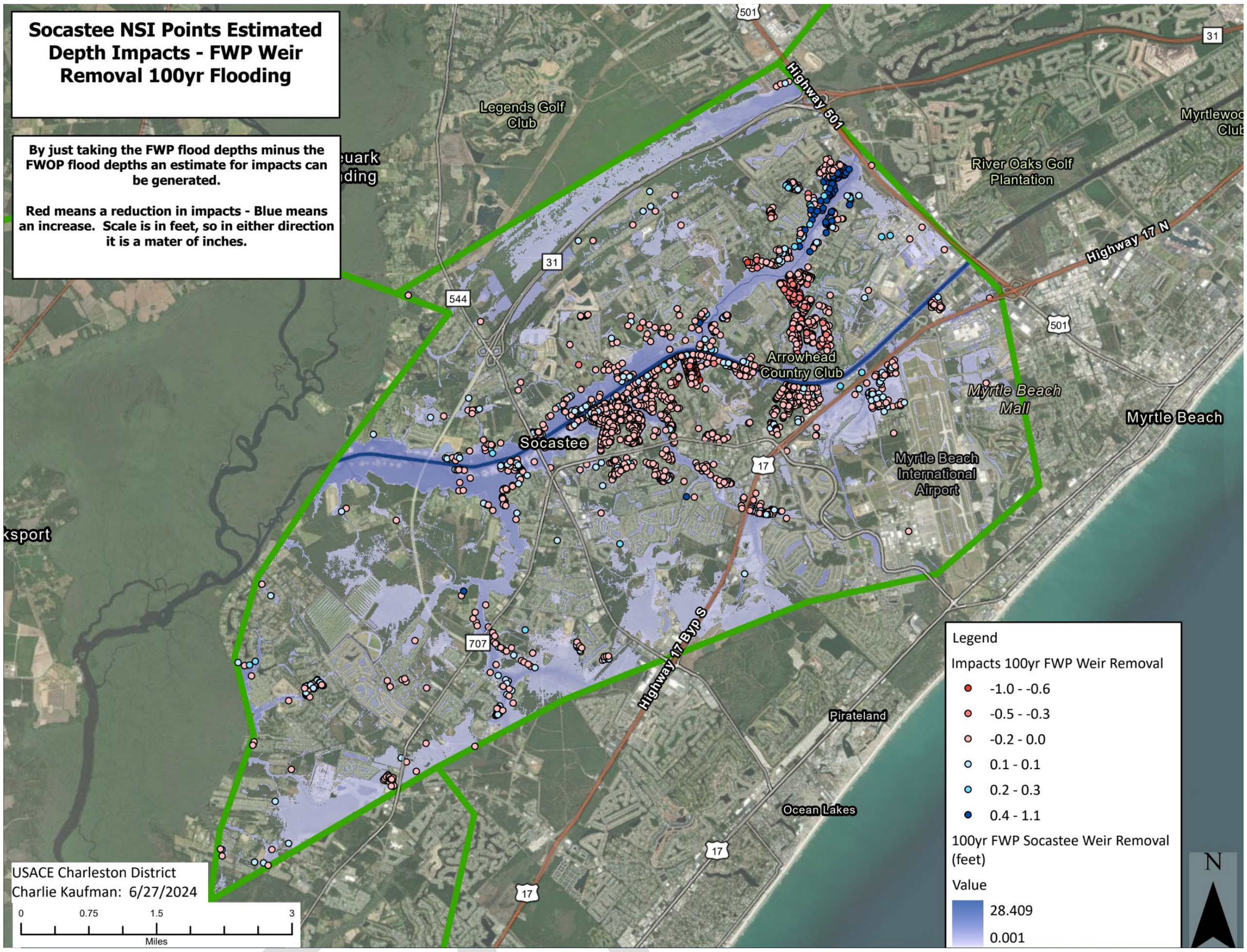


Figure 169: Socastee NSI points Estimated depth impacts with incorporation of the weir Removal.

Figure 168 and 169 depicts the difference in water depths at each structure for both the Relief Bridges (cross drains) in Conway and the Weir removal in Socastee. The red shading depicts the negative depth, meaning the lowering of the water depth and blue shading is the increase in depth. Upstream of the weirs indicate the increase in water depths and downstream shows a decrease.

10.2 Projected Trends Summary and Conclusions

The literature review projects a strong consensus that air temperatures will rise in the study area and across the country over the next century. Most studies forecast a rise in mean annual air temperature by about 2 to 4 ºC by the second half of the 21st century for the South Atlantic-Gulf Region. However, predictions regarding changes in precipitation are more uncertain, with the studies reviewed showing an even split between anticipating increases and decreases in future annual precipitation. When it comes to streamflow projections, the outcomes are mixed as well, with some models suggesting decreases and others indicating increases for the region, based on the combination of Global Climate Models (GCMs) with macro-scale hydrological models. In summary, flooding in the project area is due to extensive rainfall throughout the year, multi-day rainstorms leading to saturated soils, the warmer Atlantic Ocean is contributing to the increased rainfall and an increase in intensity and frequency of hurricanes. The projected changes and impacts to the Waccamaw River Watershed include an increase of rainstorms and extreme rainfall events causing flooding that puts people and infrastructure at risk.

Analysis of the Climate Hydrology Assessment Tool's range of model results shows a distinct upward trend in higher projections, while lower projections remain stable over time. The disparity in model outcomes widens over time, reflecting growing uncertainty the further the projections extend from the starting point. This uncertainty stems from various factors, including the initial conditions set for the GCMs, differences among GCMs themselves, and the choice of Representative Concentration Pathways (RCPs). Further uncertainties arise from the process of climate model downscaling, limited temporal resolution, and the hydrologic models themselves, as evidenced by the broad range of results depicted in Figures 3-3 to 3-4 in Appendix B.

The USACE Vulnerability Assessment tool, applied to the project area, did not identify any exceptional vulnerabilities when compared with other Hydrologic Unit Codes (HUCs) nationwide. Despite not ranking in the top 20% of vulnerable HUCs, this does not negate the potential impacts of climate change on the region. The assessment pointed out flood risk vulnerabilities related to changes in flood runoff and the extent of urban areas within the 500-year floodplain.

A sensitivity analysis was conducted increasing the intensity of 1% AEP rain event and found that the model was sensitive to the increase. A 14.6% increase in total rainfall for a 96-hour event produced a rise in water surface elevation of more than 2 feet for the Waccamaw River at Conway, SC. Stronger hurricanes coupled with extreme precipitation will destroy or damage public and private buildings and property. Increased inland flooding caused by extreme precipitation events will further increase economic and agricultural losses after an event. Vulnerable populations are most at risk of flooding and may have difficulty evacuating when necessary. These results were not used to choose the recommended plan and are not included in the Economics and Benefits analysis.

Sea level projections for the Waccamaw River basin, based on the USACE Sea Level Tracker tool and data from the Springmaid Pier, SC NOAA station, predict varying increases by 2085 and 2135. By 2085, sea level rises the Low-rate Sea level increase over the life of the project (from 2035 to 2085) was 0.17 m (0.55 ft), the Intermediate Sea level increase was 0.40 m (1.32 ft), and the High sea level increase was 1.15 m (3.76 ft). For predicted SLC through year 2135, the Low-rate Sea level increase (from the start of the project in 2035 to 2135) was 0.33 m (1.08 ft), the Intermediate sea level increase was 0.83 m (2.72 ft), and the High sea level increase was 2.64 m (8.67 ft) underscoring the broad range of possible future sea level scenarios.

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11.0 References

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Waccamaw River,

Horry County, South Carolina

Flood Risk Management Study

Draft Integrated Feasibility Report and Environmental Assessment

Appendix A2. Climate Change and Sea Level Change

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Table of Contents

1. Introduction - Inland Climate Factors for the Waccamaw River Watershed

1.1 Introduction and Background

This is an evaluation of potential climate vulnerabilities facing the Waccamaw River Watershed. This assessment was performed to highlight existing and future challenges facing the project's ability to mitigate flood risk in response to past and future climatic changes, in accordance with the guidance in Engineering Construction Bulletin (ECB) 2018-14, revised 19 Aug 2022. Background information on the project can be found in the main report, and background information on climate-affected risks to projects and assessments thereof can be found in the ECB.

USACE projects, programs, missions, and operations have generally proven to be robust enough to accommodate the range of natural climate variability over their operating life spans. However, recent scientific evidence shows that in some places and for some impacts relevant to USACE operations, climate change is shifting the baseline about which that natural climate variability occurs and may be changing the range of that variability as well. This is relevant to USACE because the assumptions of stationary climate conditions and a fixed range of natural variability, as captured in the historic hydrologic record may no longer apply. Consequently, historic hydrologic records may no longer be appropriately applied to carry out hydrologic assessments for flood risk management in watersheds such as the Waccamaw Basin.

1.2 Waccamaw River Watershed Description

The Waccamaw River is a 140-mile-long river, located in southeastern North Carolina and eastern South Carolina in the flat Coastal Plain. It drains an area of approximately 1,110 square miles (2886 km²) in the coastal plain along the eastern border between the two states into the Atlantic Ocean. Along its upper course, it is a slow-moving, blackwater river surrounded by vast wetlands, passable only by shallow-draft watercraft such as canoe. Along its lower course, it is lined by sandy banks and old plantation houses, providing an important navigation channel with a unique geography, flowing roughly parallel to the coast.

It enters South Carolina and flows southwest across Horry County, past Conway. Near Burgess, it is joined from the northwest by the Great Pee Dee River, which rises in north central North Carolina. It continues southwest, separated from the ocean by only five miles (8 km) in a long tidal estuary. The long narrow point of land along the ocean formed by the lower river is called Waccamaw Neck. At Georgetown it receives the Black River (South Carolina) from the north, then turns sharply to the southeast and enters the ocean at Winyah Bay, approximately five miles (8 km) north along the coast from the mouth of the Santee River. Inland communities across the state are at risk from flooding due to extreme precipitation throughout the entire year. The Waccamaw River basin has a temperate climate with moderate winters and warm humid summers. Rainfall is well distributed throughout the year; however, rainfall is greatest near the coast, and decreases as the terrain transitions from Coastal Plain to Piedmont regions. The average annual precipitation over the Waccamaw River basin ranges from about 48 inches near Conway, SC up to 54 inches near Bucksport, SC. Rainfall is generally well distributed throughout the year, though it is greatest during the late spring to early fall when heavy localized rainfall and hurricanes are the most prevalent. The maximum monthly rainfall averages about 7 inches and occurs during July, whereas, the driest month is November with an average rainfall of 3.1 inches (NACSE, 2021).

1.3 Observed Trends from Literature Review

The Waccamaw River is in Water Resource Region (i.e., HUC-8 watershed) number 0304, the Pee Dee Region. A January 2015 report conducted by the USACE Institute for Water Resources (USACE 2015b) summarizes the available climate change literature for this region, covering both observed and projected changes. These include; Temperature, Precipitation and Hydrology.

1.3.1 Temperature

This report synthesizes findings from various studies investigating historical temperature trends, incorporating research on both national scales, which includes data relevant to Water Resources Region 03, and more focused regional analyses specific to this area. The subsequent discussion outlines insights from these studies.

In 2009, Wang et al. conducted a study on historical climate patterns across the continental United States, utilizing gridded mean monthly climate data (0.5 degrees x 0.5 degrees) from 1950 to 2000. Their research aimed to explore the relationship between the seasonality and regionality of temperature trends and variations in sea surface temperatures. The study found broadly positive, statistically significant trends in average air temperature across most of the U.S. (as illustrated in Figure 1-1). Within the South Atlantic-Gulf Region, the findings were more nuanced: the spring and summer months showed a general, albeit slight, warming trend across much of the region. However, during autumn, the southern part of the region experienced warming, while a slight cooling trend was observed in the north. Winter months revealed a more distinct east-west split, with the eastern part warming and the western part cooling. These findings were slightly contradicted by a subsequent study from Westby et al. (2013), which analyzed data from 1949 to 2011 and indicated a general trend of winter cooling across the region. The Third National Climate Assessment (NCA) report by Carter et al. (2014) offered a broader view, examining historical average annual temperatures for the southeast, a region that encompasses but is larger than the South Atlantic-Gulf Region. This larger area showed mild warming in the early 20th century, a cooling trend for several decades thereafter, and recent signs of warming again. Nevertheless, the NCA report noted an overall absence of a clear trend in the mean annual temperature over the last century for the region, without further investigation the calculation of the statistical significance and model sensitivity of the seasonal variations.

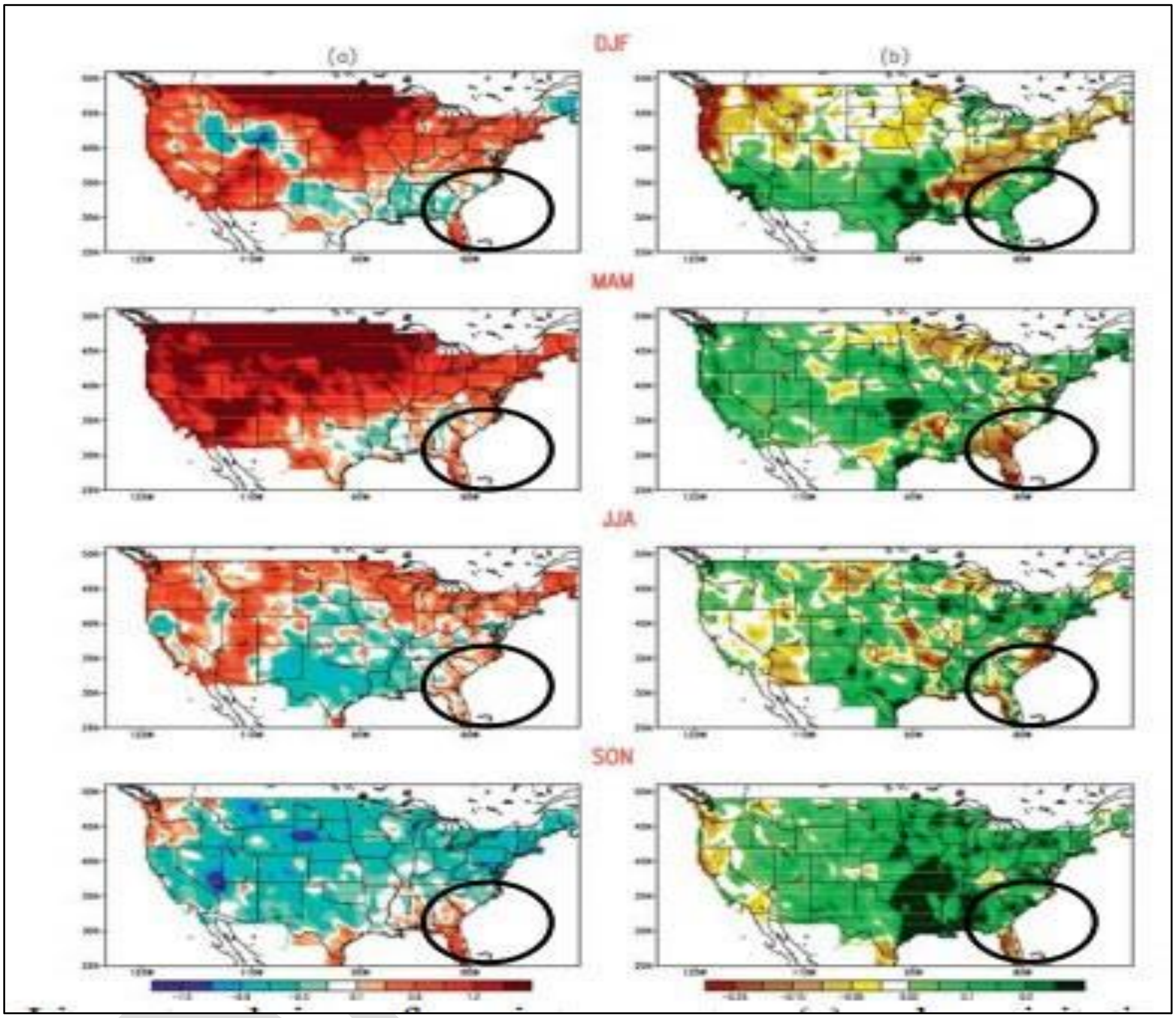
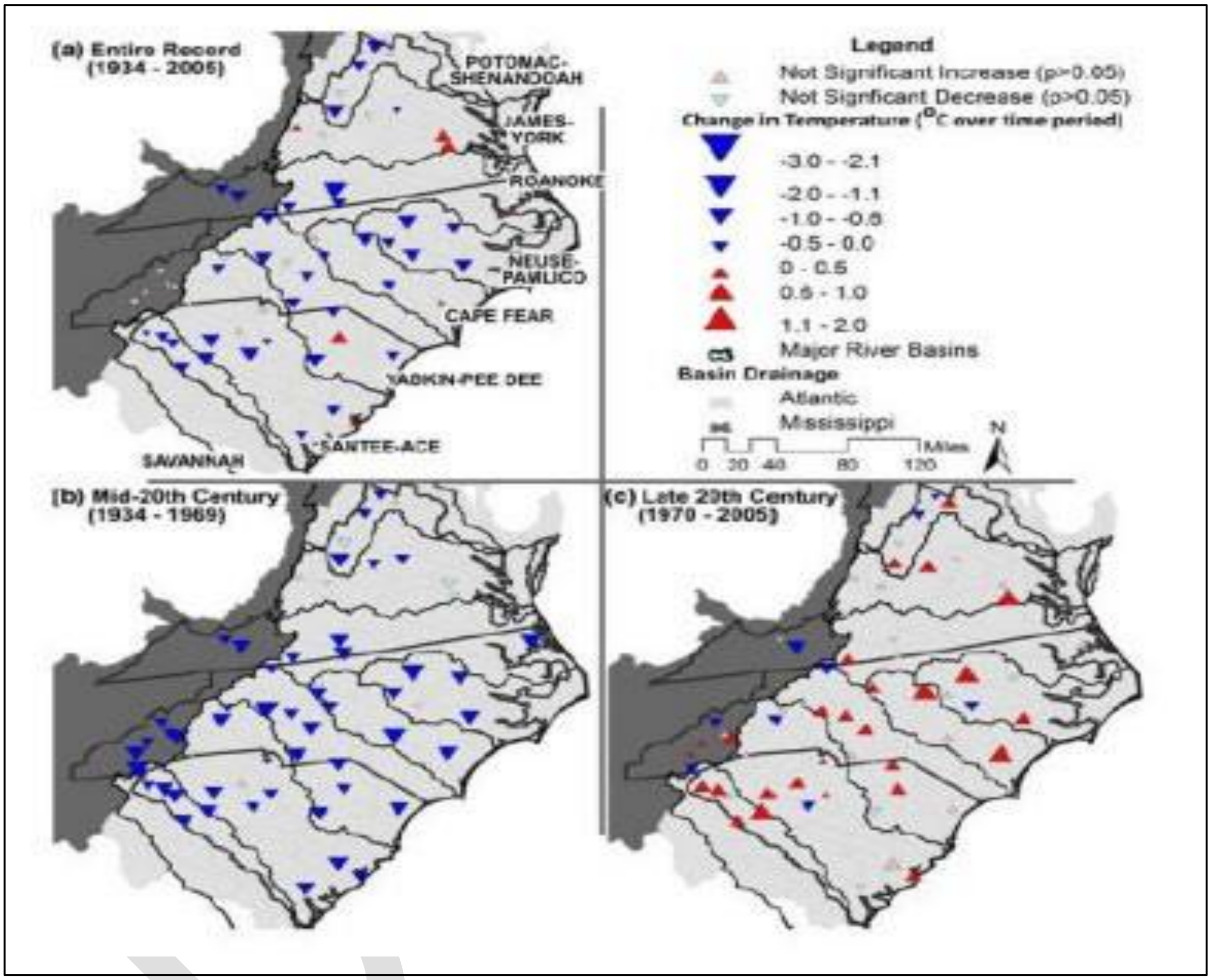


Figure 1-1. Linear trends in surface air temperature (a) and precipitation (b) over the United States, 1950 – 2000 (DJF= December, January, February; MAM= March, April, May; JJA= June, July, August; SON= September, October, November).

A 2012 study by Patterson et al. focused exclusively on historical climate and streamflow trends in the South Atlantic region. Monthly and annual trends were analyzed for several stations distributed throughout the South Atlantic-Gulf Region for the period 1934 – 2005. Results (Figure 1-2) identified a largely cooling trend for the first half of the historical period and the period as a whole. However, the second half of the study period (1970 – 2005) exhibits a clear warming trend with nearly half of the stations showing statistically significant warming over the period (average increase of 0.7 ºC). The circa 1970 “transition” point for climate and streamflow in the U.S. has been noted elsewhere, including Carter et al. (2014). Trends in overnight minimum temperatures (Tmin) and daily maximum (Tmax) temperatures for the southeast U.S. were the subject of a study by Misra et al. (2012). Their study region encompasses nearly the full extent of the South Atlantic-Gulf Region and used data from 1948 to

2010. Results of this study show increasing trends in both Tmin and Tmax throughout most of the study region. The authors attribute at least a portion of these changes to the impacts of urbanization and irrigation.

Figure 1-2. Historical annual temperature trends for the South Atlantic Region, 1934 – 2005. Triangles point in the direction of the trend, size reflects the magnitude of the change. Blue indicates a decreasing temperature trend. Red indicates an increasing temperature trend (Patterson et al., 2012)



In South Carolina specifically the temperatures have risen more than 1.2^{o}C since the beginning of the 20th century. Winter average temperatures have been increasing with the 2015-2020 period exceeding the levels of the 1930s and 1950s. Summer average temperatures in the 2005-2020 period have been the warmest on record.

- Most of North Carolina has warmed 0.6-1.2 degrees Fahrenheit in the last 100 years. The southeastern United States has warmed less than most of the nation.
- Tropical storms and hurricanes have become more intense during the past 20 years. Hurricane wind speeds and rainfall rates are likely to increase as the climate continues to warm.
- Increased rainfall may further exacerbate flooding in some coastal areas. Since

1958, the amount of precipitation during heavy rainstorms has increased by 27 percent in the southeast, and the trend toward increasingly heavy rainstorms is likely to continue.

1.3.2 Precipitation

In their 2005 study, Palecki et al. analyzed historical rainfall records from the continental United States, focusing on the period between 1972 and 2002. They leveraged NCDC's 15-minute precipitation data to identify trends in rainfall patterns. Their findings highlighted significant upticks in the intensity of winter storms (measured in millimeters per hour) and the overall precipitation during autumn in the lower areas of the South Atlantic-Gulf Region. On the flip side, a notable decrease in the intensity of summer storms was observed in the upper portions of this region.

McRoberts and Nielsen-Gammon, in their 2011 research, utilized a novel, consistent dataset to examine precipitation trends across various sub-basins in the United States, covering a lengthy period from 1895 to 2009. This extensive study uncovered generally upward trends in yearly precipitation across most of the United States, as depicted in Figure 1-3. Within the South Atlantic-Gulf Region, however, the trends were less consistent, with some areas experiencing minor drops in rainfall while others saw slight increases, leading to an inconclusive overall trend for the region based on this study.

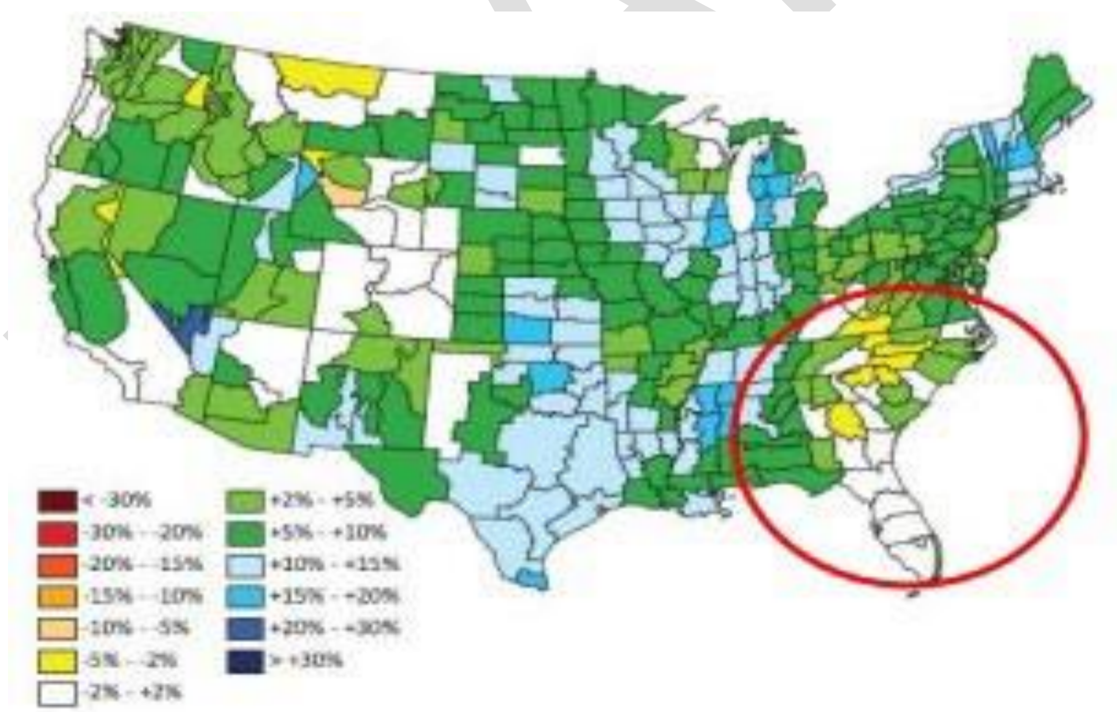
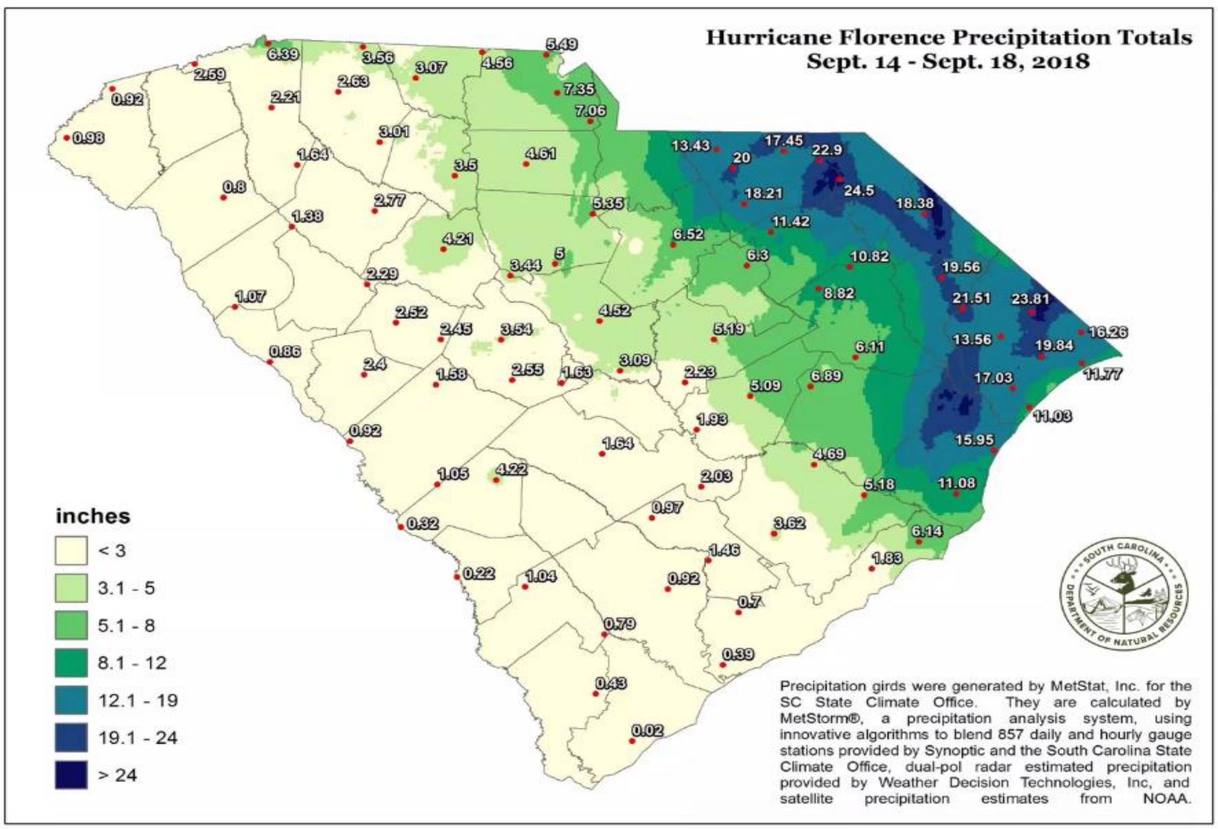


Figure 1-3. Linear trends in annual precipitation, 1895 – 2009, percent change per century. The South Atlantic-Gulf Region is within the red oval (McRoberts and Nielsen-Gammon, 2011).

A number of research initiatives have centered on analyzing variations in extreme precipitation events using updated historical records. These studies have scrutinized the severity, frequency, and duration of such weather phenomena. In their 2008 investigation, Wang and Zhang harnessed both recent historical data and downscaled models from Global Climate Models (GCMs) to probe into shifts in extreme precipitation across North America, with a specific focus on the alteration in the occurrence rate of the maximal daily precipitation event expected once every 20 years. Their examination spanned historical trends and future projections.

The research highlighted a statistically marked increase in the occurrence of these two-decade storm events within the southern and central United States, observed in both the historical records and future forecasts. Particularly in the South Atlantic-Gulf Region, a significant shift was observed in the frequency of these storms between the two periods of 1977–1999 and 1949–1976, indicating an increase in frequency ranging from 25% to 50%. Depiction of the rainfall totals from Hurricane Florence is shown in figure 1-4, generated by MetStat for SC State Climate office.

Figure 1-4. Precipitation Totals Hurricane Florence (SCDNR, 2022)



Despite these findings, the study reported a varied pattern in overall precipitation changes across the region during the studied interval. Some locations noted uptrends in precipitation, while others observed downtrends. Looking at the entire time span of the study, a greater number of sites showed slight increases in precipitation compared to decreases. Specifically, in North and South Carolina, there was no clear trend in yearly precipitation, though there was a general observation that rainfall tends to be higher during the summer months, according to a 2022 report by the National Centers for Environmental Information (NCEI).

1.3.3 Hydrology

In their 2008 study, Kalra et al. reported consistent declines in both the yearly and seasonal flow of streams across a wide array of measuring stations in the South Atlantic-Gulf Region, spanning the historical timeline from 1952 to 2001. This research also highlighted a notable shift during the mid-1970s, which coincides with a climate warming phase discussed in the temperature section (2.1). A similar conclusion was reached by Small et al. (2006), who analyzed HCDN data from 1948 to 1997, revealing significant downward trends in the annual minimum flow rates at several locations throughout the same region, although many sites showed no discernible trend either way.

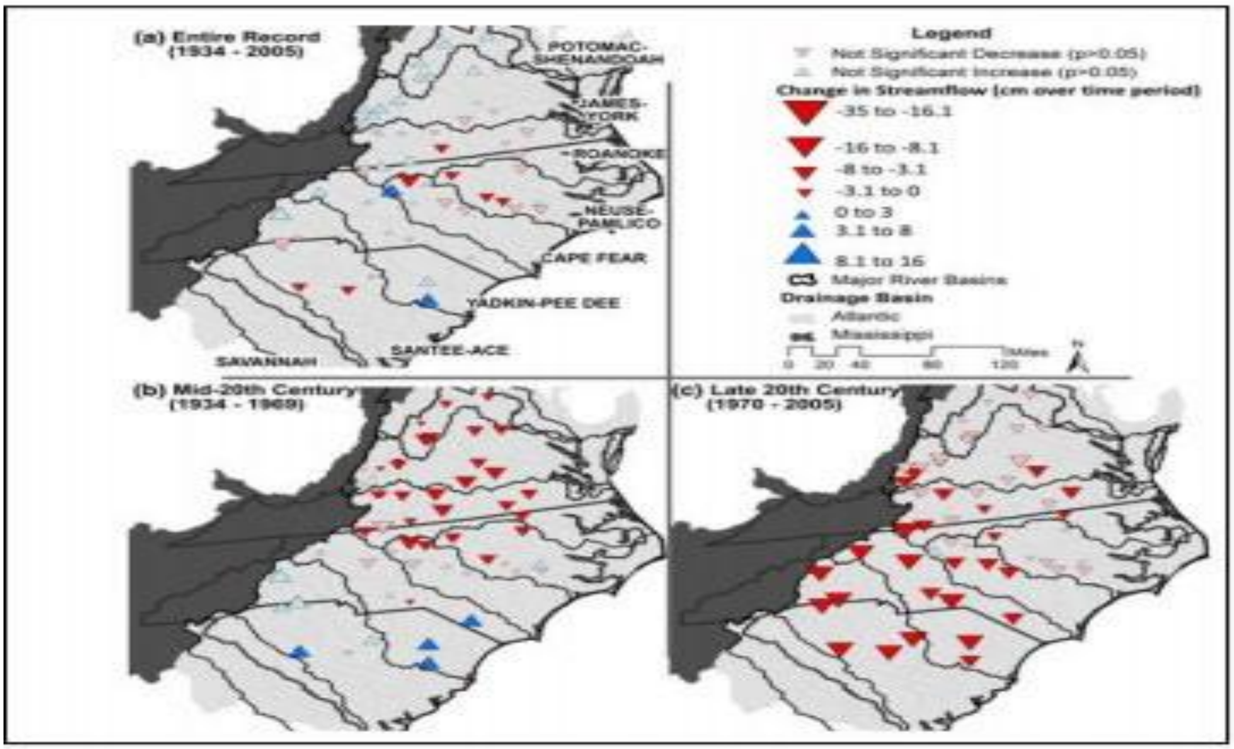


Figure 1-5. Observed changes in annual streamflow, South Atlantic Region, 1934 – 2005. Triangles point in the direction of the trend, size reflects the magnitude of the change. Blue indicates a decreasing streamflow trend. Red indicates an increasing streamflow trend. (Patterson et al., 2012).

Patterson et al. (2012) further identified a pivotal "transition" around 1970, along with marked decreases in streamflow in the South Atlantic-Gulf Region for the years 1970 to 2005, as depicted in Figure 1-5. The findings for the preceding years, from 1934 to 1969, were varied, with streamflows at some locations decreasing and at others increasing. These studies collectively emphasize the critical transition period of the 1970s in the context of regional streamflow variations.

1.3.4 Summary of Literature Review

Storm occurrences in the Waccamaw River basin are typically in the form of thunderstorms, northeasters, and hurricanes. The most severe floods of record over the basin have been associated with hurricanes. South Carolina lies in the path of tropical hurricanes as they move northerly from their origin north of the Equator in the Atlantic Ocean. These hurricanes usually occur in the late summer and autumn and have caused the heaviest rainfall and largest floods through the basin. These extreme hurricane events are characterized by heavy and prolonged precipitation.

Flooding in the project area primarily results from:

- Extensive rainfall throughout the year;
- Multi-day rainstorms leading to saturated soils;
- Warm Atlantic Ocean which is getting warmer contributing to the increased rainfall; and
- Increase in intensity and frequency of hurricanes.

These climate factors are the primary cause of floods that damage infrastructure in the project area and the focus of this climate hazard analysis.

2. Current Conditions

Large rainfall events can occur at any time of the year and cause flooding in the project area. Most recently, in 2024, a record average annual maximum 1-day precipitation total was set at Conway, SC at the municipal Airport. An average annual maximum 1- day record rainfall of 2.34 in. was set at Conway (CHAT tool). This breaks the previous record of 2.07 in. set in 2009, which is a 33% increase. This is the average annual maximum 1-day precipitation total for each epoch-scenario. The intensity of the 1-day event is a particularly good metric for estimating changes in flash and urban flooding exposure. Larger numbers indicate increased exposure.

Not only is the rainfall throughout the entire year a great concern, but the multiday storms also exacerbate the flooding issues within this region. The three-day maximum precipitation total for the Waccamaw River Watershed is 4.28 in. (Gade et al. 2020 “Indicator Values for the Waccamaw River Watershed”). Unlike 1-day precipitation, the three-day maximum precipitation measure can consider the effect of saturated soils on exacerbating flood risk by increasing the share of precipitation that runs off once the soil is saturated. Larger numbers indicate increased exposure. The saturated soils from the multiday storms only worsen the flooding in this area, because the rainfall cannot be absorbed into the soil, thus causing a larger and faster runoff.

The warmer Atlantic Ocean leads to an increase in moisture in the environment, thus more rainfall events. Climate change is likely causing parts of the water cycle to speed up as warming global temperatures increase the rate of evaporation worldwide. With more evaporation, there is more water in the air so storms can produce more intense rainfall events in some areas. This can cause flooding – a risk to the environment and human health.

Hurricanes are another source of flood risk in the project area. Communities along the Waccamaw River have experienced major flooding events over the past 25 years, with Floyd (1999), Joaquin (2015), Matthew (2016) and Florence (2018) all ranking among the most destructive storms in state history (Kunkle et al 2020). The damage from these storms was due primarily to flooding that resulted from the widespread heavy rains that accompanied the storms. Hurricane frequency for this watershed is 2.71% per year (Gade et al. 2022, "Indicator Values for the Waccamaw River Watershed"), which is the mean annual probability of being impacted by a hurricane, defined as being within 200 km buffer around the hurricane track.

Flooding puts people and infrastructure at risk. Energy infrastructure located along inland watersheds is vulnerable to flooding during heavy precipitation events. Heavy precipitation from more intense and frequent storms can cause significant damage to public and private structures such as homes, roads, utility services, etc. Vulnerable populations are most at risk of flooding and may have difficulty evacuating when necessary. Flooding poses a threat to archaeological and historic sites on floodplains across all three physiographic regions and within every river basin in the state. Increased or more frequent flooding may inundate and potentially destroy more cultural resources.

The Climate Hydrology Assessment Tool (CHAT) developed by USACE was utilized to examine trends in observed annual peak streamflow for the various gage locations shown in Table 83. The CHAT tool is used to fit a linear regression to the peak streamflow data in addition to providing a p-value indicating the statistical significance of a given trend.

The other gages that were analyzed via CHAT did not have a statistically significant linear trend. A few of the gages were not within the CHAT. There were no statistically significant trends detected in any gage that would indicate significant changes in observed streamflow due to climate change, long-term natural climate trends, or land use/land cover changes. These results will be further analyzed and checked with the nonstationary detection tool in the next section.

Figure 2-1. Trend Analysis of Longs, SC along the Waccamaw River for the timeframe 1946-2065 using the Nonstationary Tool USACE (Gade et al. 2020).

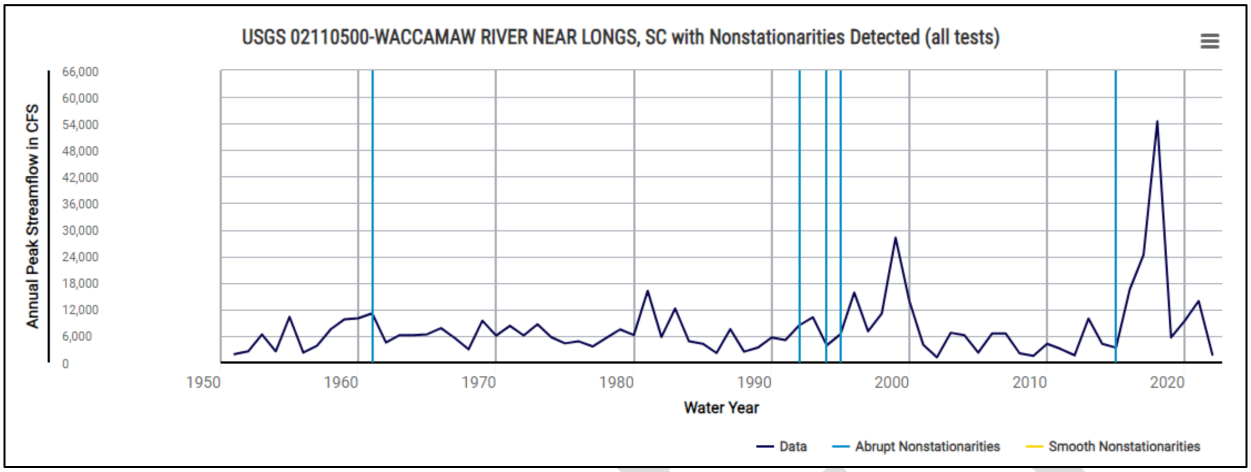


Figure 2-1 shows the trend analysis for the Waccamaw River at Longs, SC for the years 1951 to 2022. This location was chosen because it provided the appropriate historical data range and is located downstream of the North Carolina border at Longs, SC, at one of the final USGS gages along the Waccamaw River. As indicated by the Nonstationary Detection Tool developed by USACE there is no significant trend in this location.

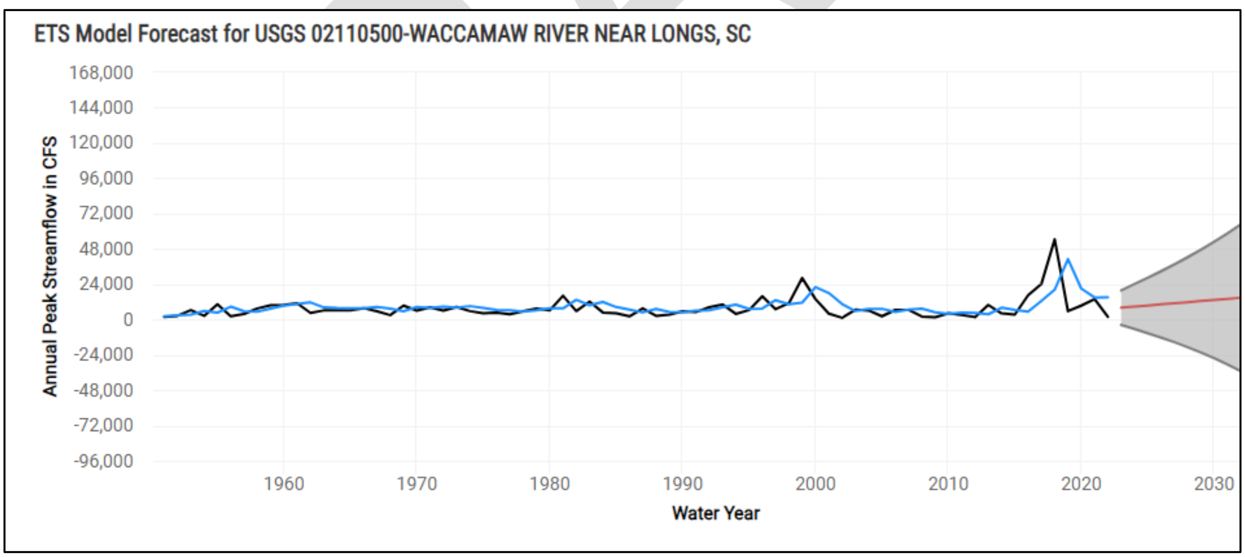


Figure 2-2. ETS Model Forecast of Longs, SC along the Waccamaw River for the timeframe 1946-2035 using the Nonstationary Tool USACE (Gade et al. 2020).

The annual peak instantaneous streamflow plot made available through the CHAT shows that there is a slight downward trend of streamflow vs. water year as shown in Figure 2-2.

3. Future Conditions

The intensity of the strongest rainfall is likely to increase with warming of the oceans and atmosphere, leading to greater damage to people, communities, our economy and natural resources from more intense hurricanes and accompanying flooding and precipitation. Sea surface temperature increased during the 20th century and continues to rise, enhancing precipitation in the project area. More frequent flooding will impact inland habitats, fisheries, and the protective services that natural areas provide to local communities.

The intense rainfall events are expected to increase in magnitude and frequency as well as the multi day rainfall events, which exacerbate the flooding issues in this region.

From 1901 through 2020, global sea surface temperature rose at an average rate of 0.14°F per decade (see Figure 3-1). Sea surface temperatures are projected to increase in the future, and these warmer temperatures are expected to contribute to increasing precipitation intensity in the project area. In addition, many storms draw moisture from the nearby Atlantic Ocean, and warming sea surface temperatures are expected to increase the available moisture, enabling larger storms to form and increasing the precipitation in the project area.

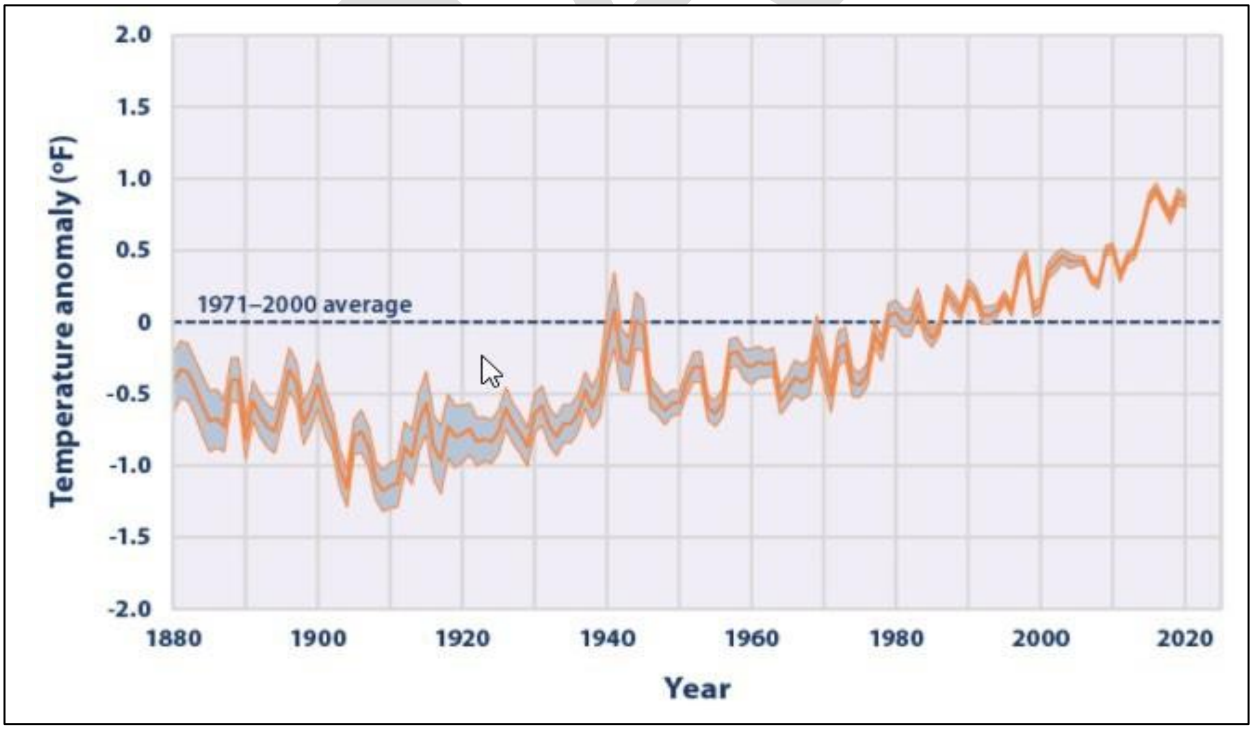


Figure 3-1. Average Global Sea Surface Temperature Change, 1881-2020. (NOAA, 2021).

An increase of the intensity of hurricane rainfall is a major concern for this area in a warmer climate. Heavy precipitation accompanying hurricanes and other weather systems is likely to increase, thus increasing the potential for flooding in inland areas, such as this area. For the Waccamaw River Watershed, the average number of days of extreme precipitation is expected to increase to an average of 4.94 days per year. This refers to the average annual number of days in which precipitation in the future is projected to exceed the amount that occurred 1% of the days in the historic period. This provides a measure of future increases in precipitation intensity that is relative to current conditions and can be used to assess how frequently heavy precipitation events may disrupt activities, and potentially overwhelm existing flood risk management infrastructure. Stronger hurricanes will destroy or damage public and private buildings and property. Increased inland flooding caused by extreme precipitation events will further increase economic and agricultural losses after a flooding event.

3.1 Nonstationarity Detection

The assumption that discharge datasets are stationary (their statistical characteristics are unchanging) in time underlies many traditional hydrologic analyses. Statistical tests can be used to test this assumption using techniques outlined in Engineering Technical Letter (ETL) 1100-2-3. The Nonstationarity Detection (NSD) tool is a web-based tool to perform these tests on datasets of annual peak streamflow at U.S. Geological Survey (USGS) stream gages. The primary objective of this study is to evaluate flood control operations, so the focus of this investigation is the high flow regime that is best represented by annual instantaneous peak flows.

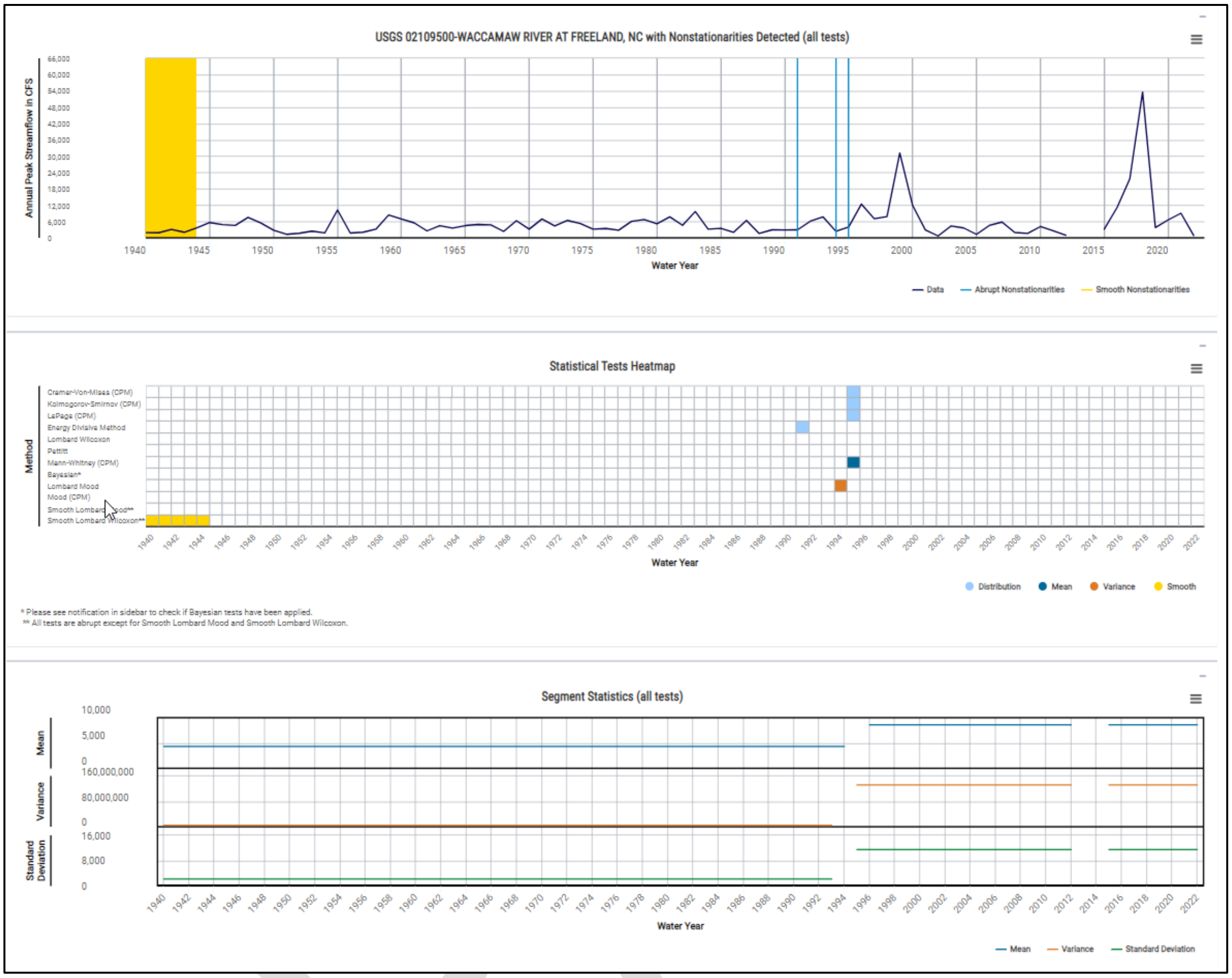


Figure 3-2. Nonstationarity Detection Tool USGS 02109500 Waccamaw River (Gade et al., 2022 Nonstationarity Detection Tool).

A nonstationarity can be considered “strong” when it exhibits consensus among multiple nonstationarity detection methods, robustness in detection of changes in statistical properties, and a relatively large change in the magnitude of a dataset’s statistical properties, which is shown in Figure 3-2). Many of the statistical tests used to detect nonstationarities rely on statistical change points, these are points within the time series data where there is a break in the statistical properties of the data, such that data before and after the change point cannot be described by the same statistical characteristics. Similar to nonstationarities, change points must also exhibit consensus, robustness, and significant magnitude of change.

3.2 Climate Hydrology Assessment Tool

The USACE CHAT can be used to assess projected, future changes to streamflow in the watershed. Projections are at the spatial scale of a HUC-4 watershed, with flows generated using the U.S. Bureau of Reclamation (USBR) Variable Infiltration Capacity

(VIC) model from temperature and precipitation data statistically downscaled from GCMs using the Bias Corrected, Spatially Disaggregated (BCSD) method. The USBR VIC model is set up to simulate unregulated basin conditions. The Waccamaw Watershed is in HUC 0304- Pee Dee. Figure 3-3 shows the range of output presented in the CHAT using 93 combinations of GCMs and representative concentration pathways (RCPs) applied to the generate climate-changed hydrology using the USBR VIC model. The range of data is indicative of the uncertainty associated with projected, climate-changed hydrology.

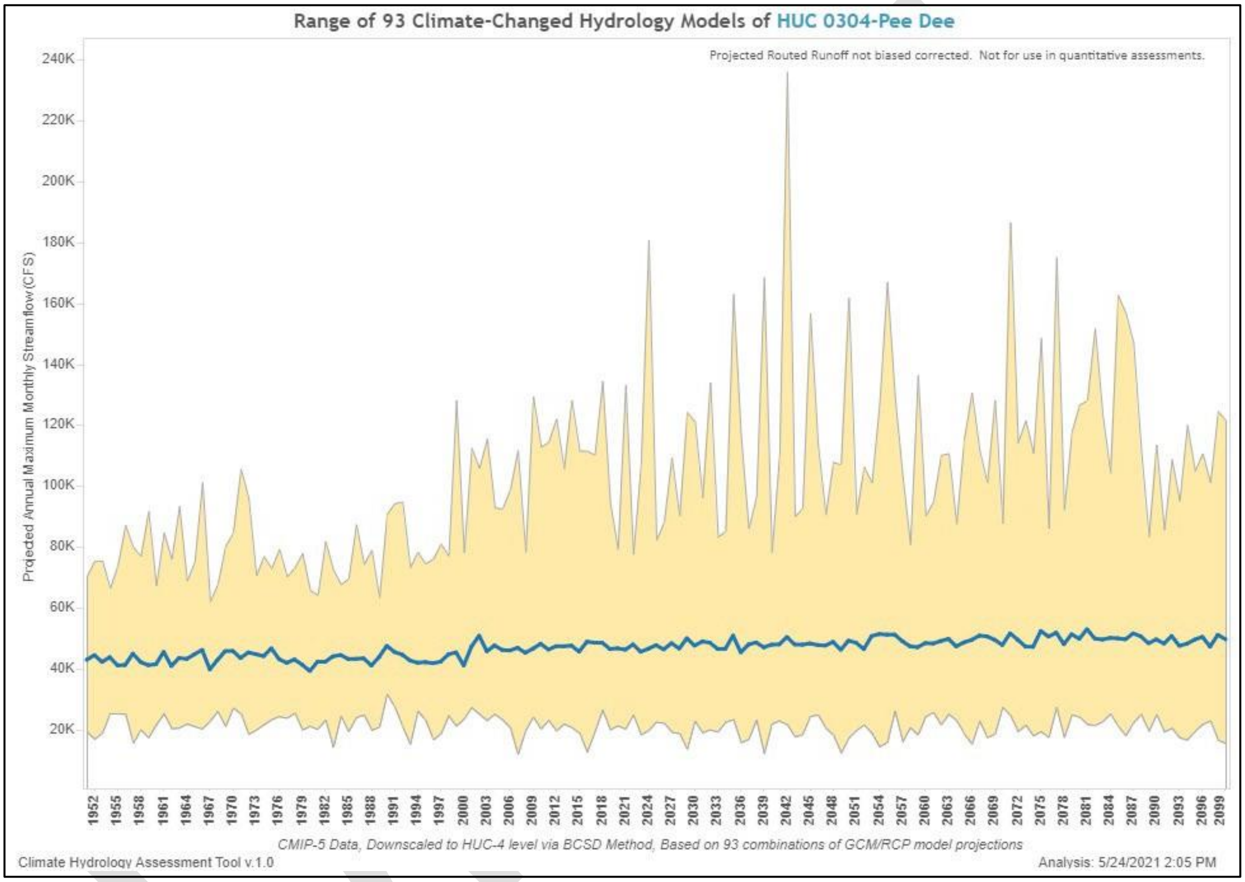
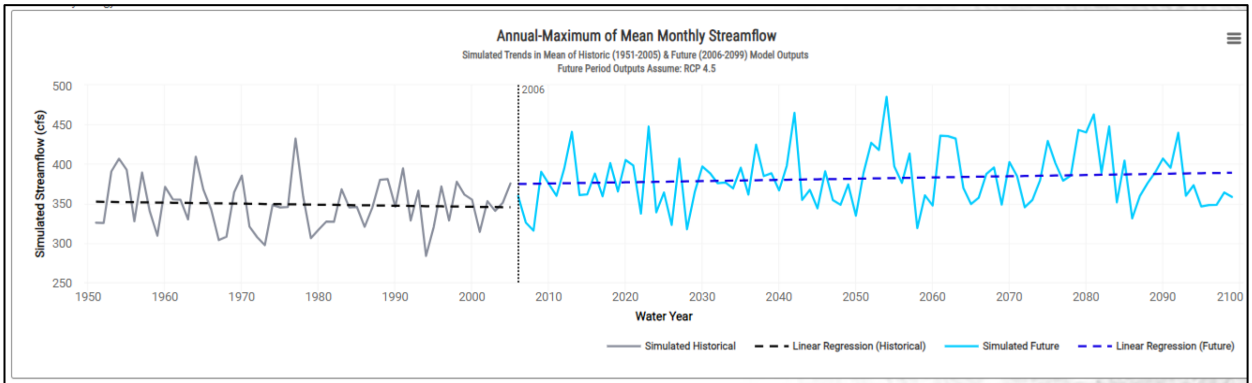


Figure 3-3. Range of 93 Climate-Changed Hydrology Models of HUC 0304-Pee Dee (Gade et al., 2020 Climate Hydrology Assessment Tool).

Figure 3-3 shows the climate changed hydrology models for the HUC that the Waccamaw River watershed is within. As indicated in the plot, the projected annual maximum monthly streamflow has increasingly intense events, but the trendline continues at a slight upward trend.



Simulated Trend Lines

Simulated Historical (1951 to 2005)

-0.1309
Traditional Slope

Simulated Future (2006 to 2099)

0.1534
Traditional Slope

Statistical Significance Tests (Historical)

Test	p-value
t-Test	0.628
Mann-Kendall	0.873
Spearman Rank-Order	0.863

Statistical Significance Tests (Future)

Test	p-value
t-Test	0.26
Mann-Kendall	0.396
Spearman Rank-Order	0.473

Figure 3-4. Trends in Mean of 93 Climate-Changed Hydrology Models of HUC 0304- Pee Dee (Gade et al., 2022).

Similarly in Figure 3-4, the mean of projected annual maximum monthly streamflow has an upward trend from 2000 to 2100. This shows the projected increase in streamflow for the Pee Dee HUC.

3.3 Vulnerability Assessment

The USACE Watershed Climate Vulnerability Assessment (VA) Tool facilitates a screening-level, comparative assessment of the vulnerability of a given business line and HUC-4 watershed to the impacts of climate change, relative to the other HUC-4 watersheds within the continental United States (CONUS). It uses the Coupled Model Intercomparison Project (CMIP5) GCM-BCSD-VIC dataset (2014) to define projected hydrometeorological inputs, combined with other data types, to define a series of indicator variables to define a vulnerability score.

Vulnerabilities are represented by a weighted-order, weighted-average (WOWA) score generated for two subsets of simulations (wet—top 50% of cumulative runoff projections; and dry—bottom 50% cumulative runoff projections). Data are available for three epochs. The epochs include the current time period ("Base") and two 30-year, future epochs (centered on 2050 and 2085). The Base epoch is not based on projections and so it is not split into different scenarios. For this application, the tool was applied using its default, National Standards Settings. In the context of the VA Tool, there is some uncertainty in all of the inputs to the vulnerability assessments. Some of this uncertainty is already accounted for in that the tool presents separate results for each of the scenario-epoch combinations rather than presenting a single aggregate result.

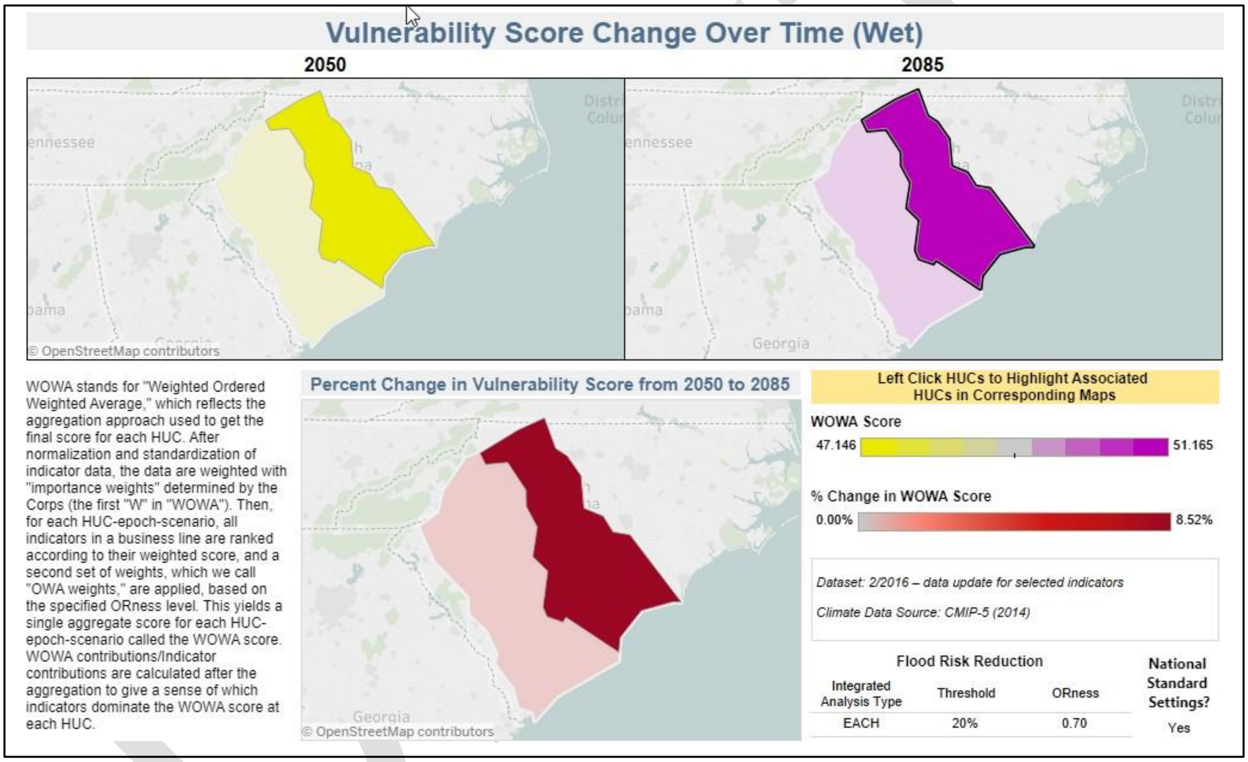


Figure 3-5. Vulnerability Score change over time for the Pee Dee watershed (Gade et al., 2020 Civil Works Vulnerability Assessment Tool).

As shown in Figure 3-5, the Trinity (HUC 1203) watershed is considered relatively vulnerable to climate change impacts for the flood risk reduction business line, being among the 20% most vulnerable watersheds for this business line in the CONUS (202 HUC04s). This is true for both the wet and dry scenarios and both the 2050 and 2085 epochs. The primary driver of this flood risk vulnerability for all scenarios and epochs is indicator 590: acres of urban area within the 500-year floodplain. Other important contributors at this location include runoff elasticity and flood magnification. Figure 3-5 shows a visualization of climate risk scores change over time for the Waccamaw River watershed region. The change in climate risk score changes over time from the year

2050 to 2085. The WOWA (Weighted Ordered Weighted Average) score is indicated as 47.146 in 2050 and 51.165 in 2085, with a change in score of 8.52% (Gade et. al. 2020).

4. Climate Sensitivity Analysis

The sensitivity of the Waccamaw River's hydrologic response to climate non-stationarity was tested using the methodology developed by the North Carolina Institute of Climate Studies (NCICS) for SERDP and NOAA. The results of the simulation indicate that climate non-stationarity could have a significant impact on future water surface elevations and flooding conditions within the Pee Dee and Waccamaw River basins. A 14.6% increase in total rainfall for a 96-hour event produced a rise in water surface elevation of more than 2 feet for the Waccamaw River at Conway, SC. It should be noted that the 90% confidence intervals for the rainfall values are large for the 100-year event, 10.70 to 15.93 inches for Atlas, 14 and 11.73 to 19.12 inches for the NCICS values. These results were not used to choose the recommended plan and are not included in the Economics and Benefits analysis. The results are provided in Appendix A- Hydrology and Hydraulics.

5. Sea Level Change Assessment

Sea level change (SLC) at the Waccamaw River was evaluated following the guidelines presented in USACE Engineer Pamphlet EP 1100-2-1 "Procedures to Evaluate Sea Level Change: Impacts, Responses and Adaptation" (30 Jun 2019). The purpose of the EP was to provide instructional and procedural guidance to analyze and adapt to the direct and indirect physical and ecological effects of projected sea level change on USACE projects and systems of projects needed to implement Engineer Regulation (ER) 1100-2-8162.

ER 1100-2-8162 "Incorporating Sea Level Change in Civil Works Programs" (15 June 2019) provides both a methodology and a procedure for determining a range of SLC estimates based on global sea level change rates, the local historic sea level change rate, the construction (base) year of the project, and the design life of the project. Three estimates are required by the guidance, a Low (Baseline) estimate representing the minimum expected SLC, an Intermediate estimate, and a High estimate representing the maximum expected SLC. The guidance will be used to evaluate the future sea levels, the impacts to the Waccamaw River project during a 50-year period, and to assess the risk associated with the SLC estimates.

The first step in evaluating sea level change was to identify a nearby NOAA water level gauge with a sufficiently long data record. The analysis was based on the NOAA tide gauge located in Springmaid Pier, Myrtle Beach, South Carolina (Station #8661070), seaward adjacent of Socastee (NOAA 2024b). The gauge is compliant and active with a historic record of 1976 to present, which includes a 2-month data gap in 1976, an 18-month data gap from September 1989 to April 1991, and a 10-month data gap in 2014. From Figure 5-1 the linear relative sea level trend for this gauge is 3.29 mm/yr (0.0108 ft/yr) with a 95% confidence interval of +/- 0.480 mm/yr (0.00157 ft/yr) based on monthly mean sea level data. For the 50-year analysis of 2035 to 2085 this is equivalent to an increase of 0.165 m (0.540 ft) in sea level. Regional sea level trends for stations on the central east coast are shown in Figure 52. Stations directly to the north of the project location show a lower sea level trend, while stations directly to the south show a higher sea level trend. Coastal dynamics for the project location are closer to the dynamics at the Springmaid Pier, SC location. Note that the nearby NOAA gauges at Southport (8659084) and Wrightsville Beach (8658163) are non-compliant with less than 50-years of data and with interrupted records.

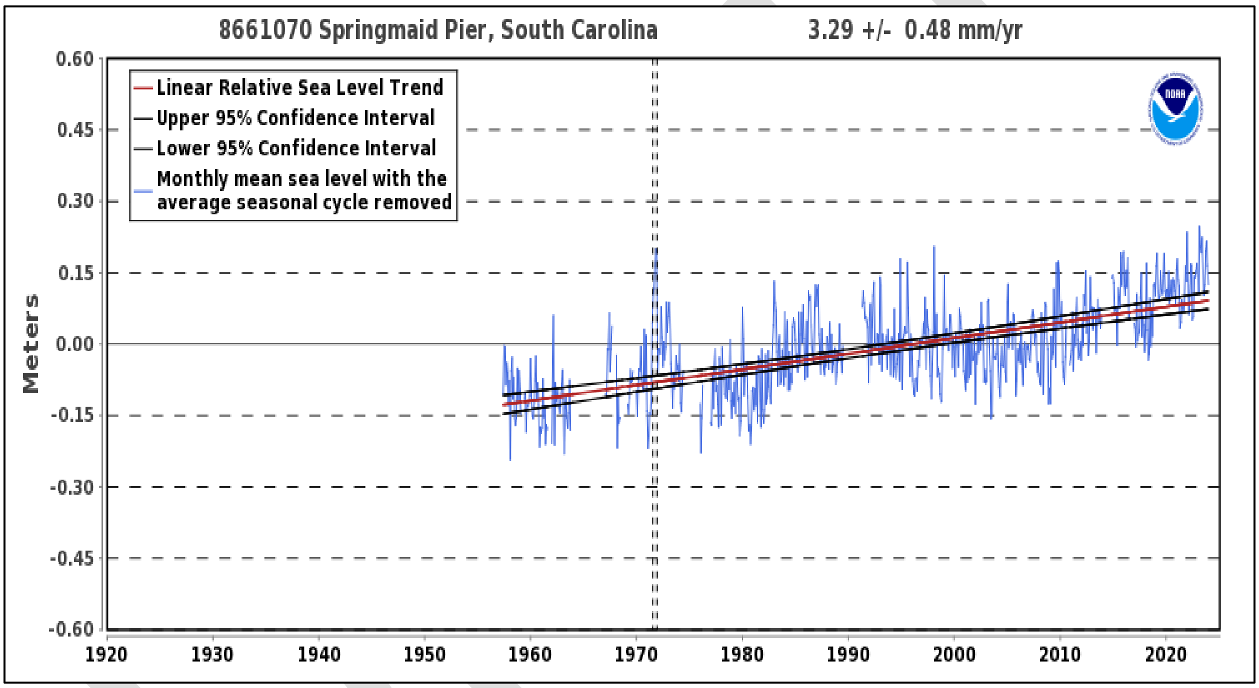


Figure 5-1. Relative Sea Level Trend, NOAA Gauge 8661070

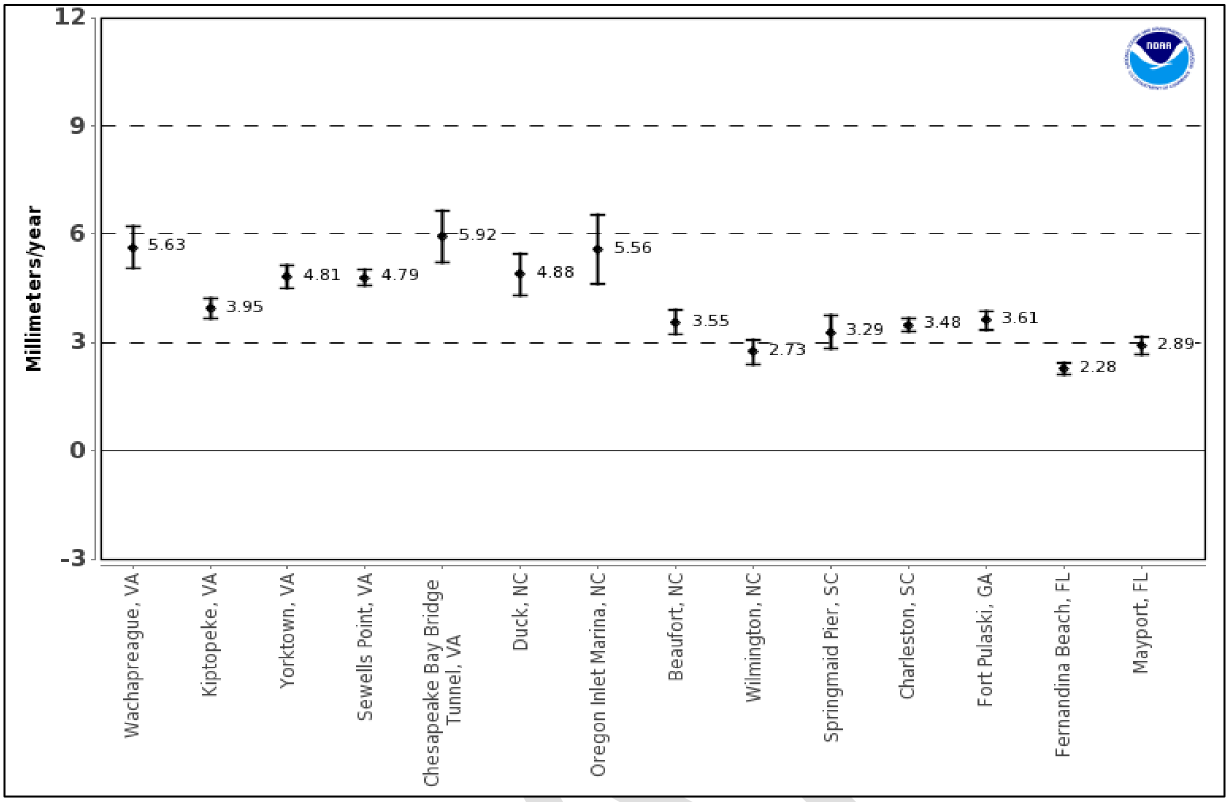


Figure 5-1. Regional Sea Level Trends.

The second step in evaluating SLC was to assess future trends, mainly in determining whether the rate of sea level rise accelerates in the future. Any future increase or decrease in this long-term trend along with land subsidence and glacial rebound needs to be addressed throughout the 50-year period.

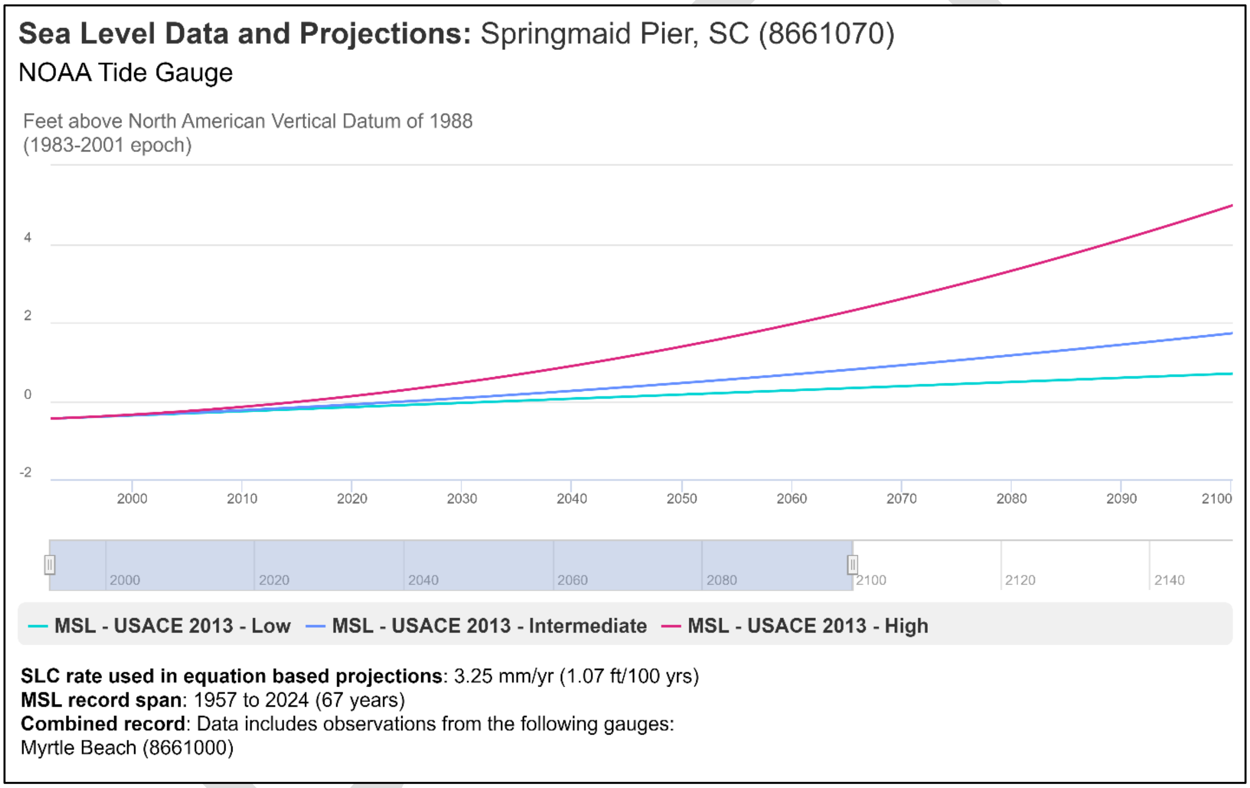
The USACE online tool Sea Level Tracker was used to determine the current rate of SLC observed and the projected future trends in the rate of SLC. A link to the tool is provided below. Extreme water levels (EWL) incorporated into the tool are based on statistical probabilities using recorded historic monthly extreme water level values. The Sea Level Tracker is used to compare actual mean sea level (MSL) values and trends for specific NOAA tide gauges with the USACE SLC scenarios as described in ER 1100-2-8162 and Engineer Technical Letter (ETL) 1100-2-1. The Sea Level Tracker tool calculates the USACE Low, Intermediate, and High sea level change scenarios based on global and local change effects. Historical MSL is represented by either 19-year or 5-year midpoint moving averages. Guidance in using the Sea Level Tracker and technical background is provided in USACE “Sea Level Tracker User Guide,” Version 2.0, December 2022.

https://climate.sec.usace.army.mil/slr_app/

The Sea Level Tracker tool was used to evaluate the NOAA Springmaid Pier gauge data. The regionally corrected rate of 0.0133 ft/yr was used as the rate of SLC and was sourced from Technical Report NOS CO-OPS 065 (Zervas et al., 2013) and accounts for vertical land motion. This regional rate is also the Low USACE estimated SLC rate.

Based on the regional rate only, the sea level increase was 0.665 ft during the 50-year period of 2035 to 2085. Figure 5-2 presents the results of the Tracker tool focused on trends between 1992 to 2024. The light blue line represents the 5-year moving average and the heavy dark pink line represents the 19-year moving average. The 19-year average is useful in that this represents the moon's metonic cycle and the tidal datum epoch. These estimates are referenced to the midpoint of the latest National Tidal Datum epoch, 1992. The reader is referred to ER 1100-2-8162 for a detailed explanation of the procedure, equations employed, and variables included to account for the eustatic change as well as site specific uplift or subsidence to develop corrected rates. The red line is the High SLC prediction, the green is the Intermediate and the blue is the Low rate prediction. From Figure 5-2 it can be noted that the 19-year moving average is below the Low SLC curve and the 5-year moving average is above the Intermediate curve, but both are sloping upward.

Figure 5-2. Springmaid Pier NOAA Gauge #8661070 SLC with 19-Year and 5-Year Moving



Average.

The future USACE sea level predictions for the Waccamaw River project based on the Springmaid Pier gauge are provided in Table 5 1. For the 2035 to 2085 period the predicted Low rate sea level rise (regional rate) is 0.54 ft, the Intermediate SLC increase was 1.14 ft and the High SLC increase was 3.06 ft. The future SLC curves are shown in Figure 5-4. For comparison, the regionalized NOAA estimates (NOAA et al, 2012) are also provided in Table 5-1.

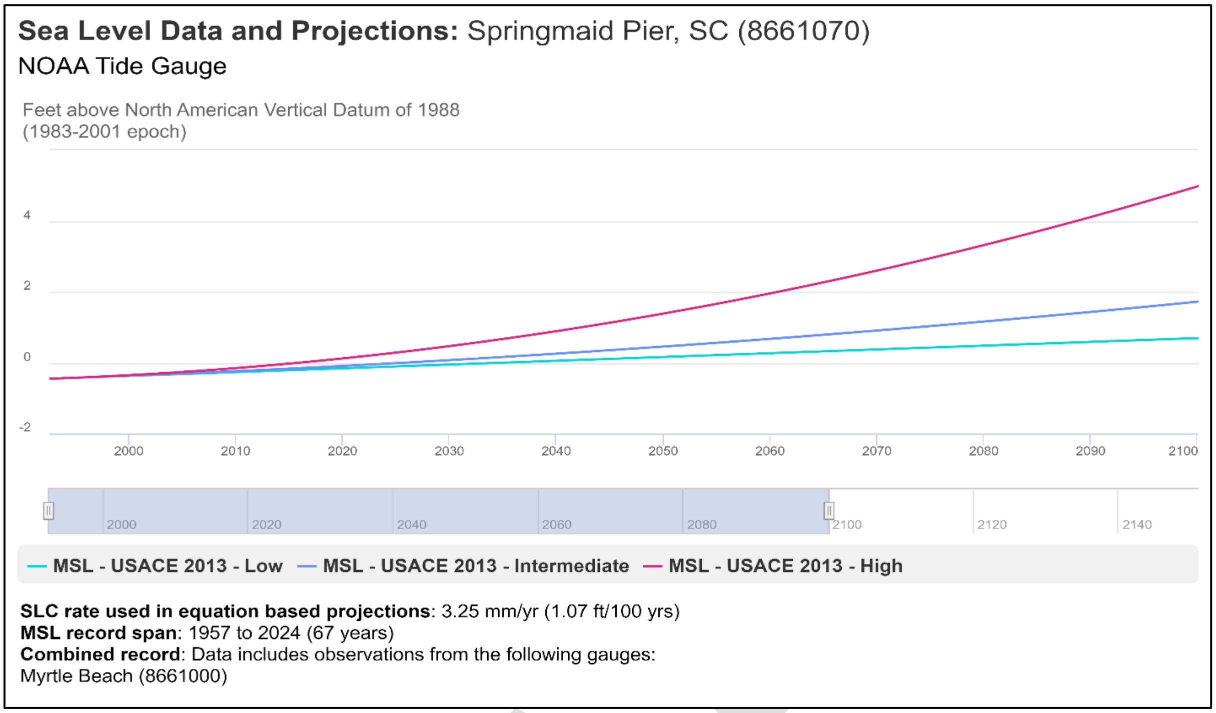


Figure 5-3. Springmaid Pier Gauge USACE Sea Level Change Predictions, 1992 to 2100.

Table 5-1. USACE and NOAA 50-Year and 100-Year Sea Level Change Estimates (ft NAVD88)

Project Year	Year	USACE			NOAA			
		Low	Int	High	Low	Int-Low	Int-High	High
Tidal Epoch	1992	-0.45	-0.45	-0.45	-0.45	-0.45	-0.45	-0.45
Base	2024	-0.106	-0.016	0.28	-0.106	-0.016	0.19	0.426
	2030	-0.04	0.08	0.49	-0.04	0.08	0.37	0.69
Start	2035	0.01	0.175	0.705	0.01	0.175	0.545	0.965
	2040	0.06	0.27	0.92	0.06	0.27	0.72	1.24
	2050	0.17	0.47	1.42	0.17	0.47	1.13	1.89
	2060	0.28	0.69	1.99	0.28	0.69	1.6	2.64
	2070	0.38	0.92	2.64	0.38	0.92	2.12	3.5
	2080	0.49	1.18	3.36	0.49	1.18	2.7	4.45
End	2085	0.55	1.32	3.76	0.55	1.32	3.02	4.98
	2090	0.6	1.45	4.16	0.6	1.45	3.34	5.51
	2100	0.7	1.74	5.03	0.7	1.74	4.03	6.67
	2110	0.81	2.05	5.97	0.81	2.05	4.79	7.93
	2120	0.92	2.37	6.99	0.92	2.37	5.6	9.3
	2130	1.02	2.71	8.08	1.02	2.71	6.46	10.77
	2135	1.08	2.90	8.67	1.08	2.90	6.93	11.56
50-Year Increase =		0.54	1.14	3.06	0.54	1.14	2.48	4.02
100-Year Increase =		1.07	2.72	7.96	1.07	2.72	6.38	10.59

To compare the predicted Springmaid Pier USACE SLC trends with regional NOAA gauges, the tide gauges # 8658120 at Wilmington, NC (Figure 5-5) 80 miles to the north and # 8665520 at Charleston, SC (Figure 5-6) 85 miles to the south were reviewed. The 1992 to 2022 SLC trends with the 19-year and 5-year moving averages are provided in Figures 5-5 and 5-6. Both gauges are active and compliant with over 40-years of data. The Wilmington gauge shows a trend closer to the high rate for the 19-year moving average and above the high rate for the 5-year moving average. For the Charleston gauge the 19-year average is near the intermediate rate while the 5-year moving average is closer to the high rate.

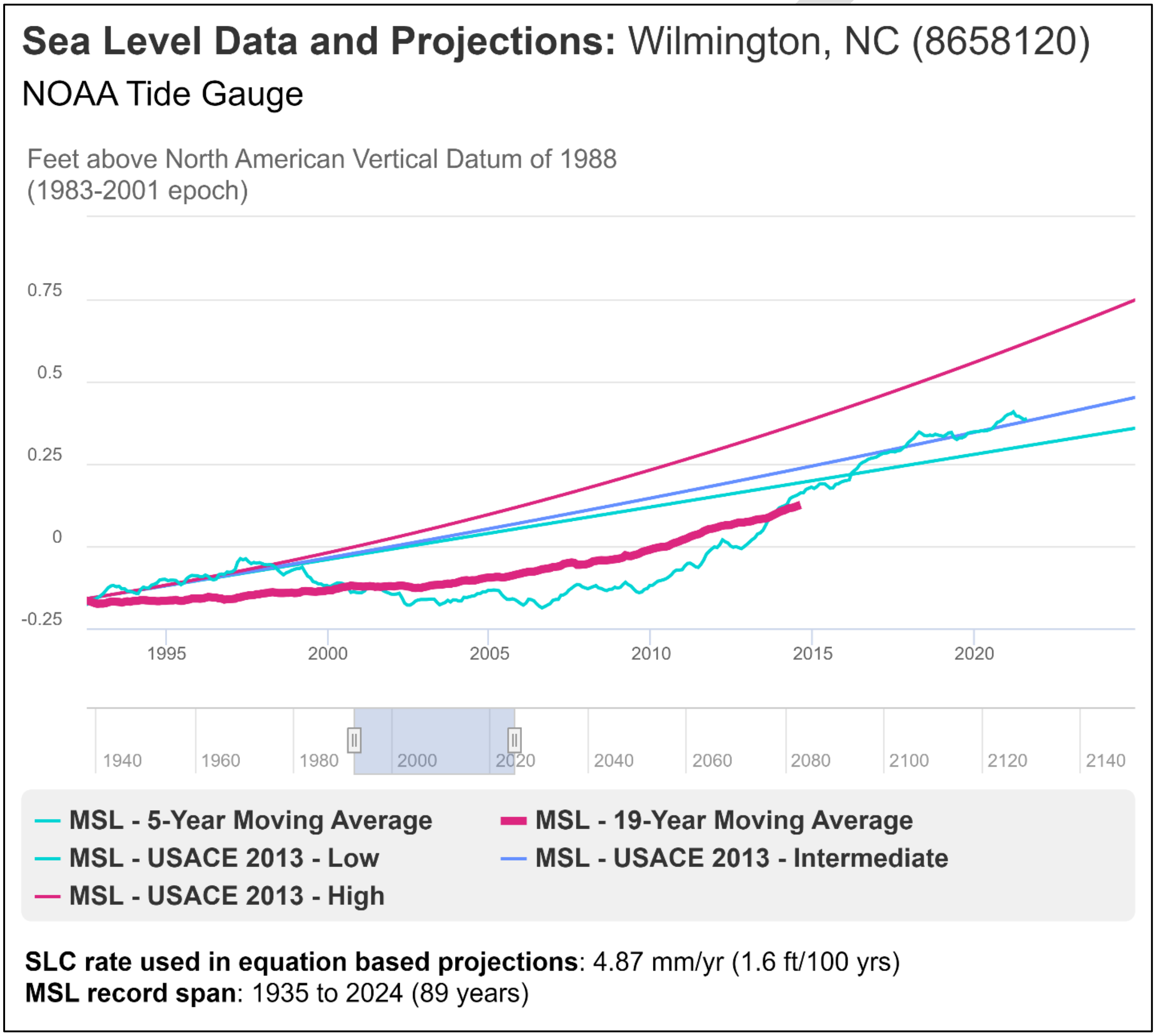


Figure 5-5. Wilmington, NC NOAA Gauge # 8658120 SLC with 19-Year and 5-Year Moving Average

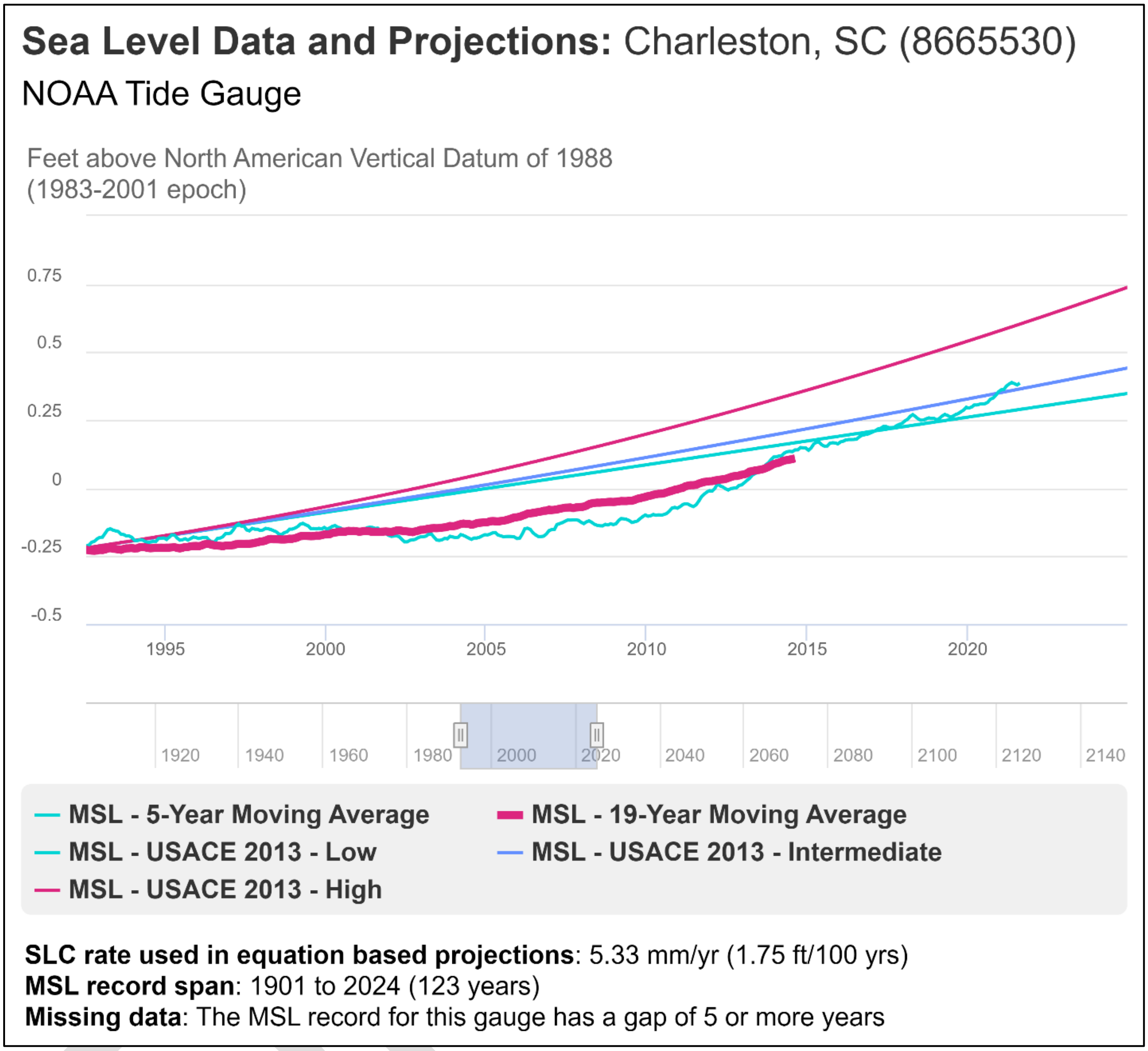


Figure 5-6. Charleston, SC NOAA Gauge # 8665530 SLC with 19-Year and 5-Year Moving Average

The effect of SLC on overall hazard levels for the Waccamaw River project was analyzed using a sensitivity analysis. This approach allows modelers to estimate the correlation between SLC and the increase in water level without having to model different SLC scenarios for each storm, which would significantly multiply the required compute time and lengthen the overall project schedule. Instead, hindcasts were run with and without SLC to estimate the magnitude of impacts to the total water surface elevation. The intermediate curve was chosen for sensitivity analyses because this curve is the closest match to the 5-year moving average and provides a more conservative approach than the low curve, which follows closer to the 19-year moving average. This method was selected in coordination with the USACE Climate Preparedness and Resilience Community of Practice.

Simulations were run with and without SLC at the boundary condition in HEC-RAS. SLC was simulated using the USACE Intermediate value of 1.32 ft NAVD88 for the end year 2085. Each simulation was run using the same 1% AEP upstream boundary conditions, but with varying downstream boundary conditions for no SLC, the Intermediate year 2085 SLC, and Intermediate year 2085 SLC with highest astronomical tides. Cross-section plots of the maximum water surface elevations were created for each location in Figure 5-7 and presented in Appendix 1, Section 5.3.4.6, Figures 79-86.

As expected, the northernmost points feel little to no effect of the higher sea levels. Points further south, closer to the mouth of the river, experienced minimal effect of the higher sea levels, with the addition of tides causing more of an impact than the addition of SLC. At the southernmost point, the maximum water levels at the center of the river were 6.88 ft for no SLC, 7.32 ft for with-SLC, and 8.32 ft for with-SLC and tides.

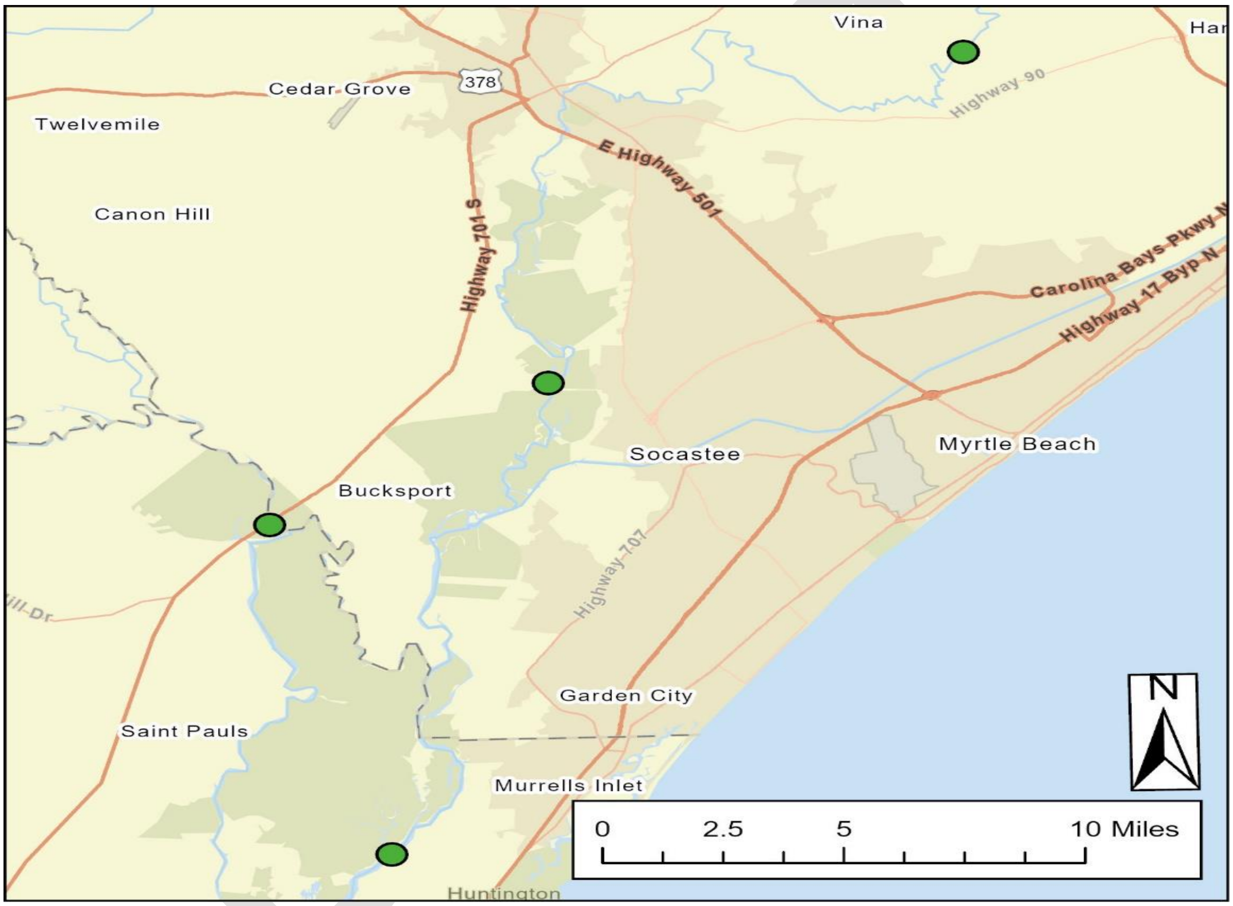


Figure 5-7. Cross-Section Locations.

An analysis was also done for the nonlinearity of SLC related to surge-only annual exceedance frequencies (AEFs). Coastal Hazards System (CHS) wave and water level data for each of the three SLC values modeled in CHS (0 ft, 2.73 ft, and 7.35 ft) were gathered at each location of interest: Longs, Bucksport, Conway, Socastee, and the HEC-RAS model boundary (Figure 5-8). The nonlinear residual, or the water level increase in addition to storm surge and SLC, was calculated as the total water level AEF with surge and SLC, minus the original storm surge water level AEF without SLC, minus the SLC value for each AEF and displayed in Table 5-2 below. It is important to note that these values are for residual from coastal effects only; the order of magnitude is much lower than that of fluvial events in the study location.

Table 5-2 shows that estimating effects of SLC by linear addition will introduce minimal error in the accuracy of SWL AEF estimates compared to overall changes in water levels from flooding. Therefore, the project delivery team concluded that a linear addition of SLC to the total SWL is acceptable in analyzing model results. This allows modelers to estimate the total SWL for scenarios with SLC without having to run the model with each SLC option. Instead, a sensitivity analysis was run in HEC-RAS (Appendix 1, Section 5.3.4.6) to confirm these assumptions.

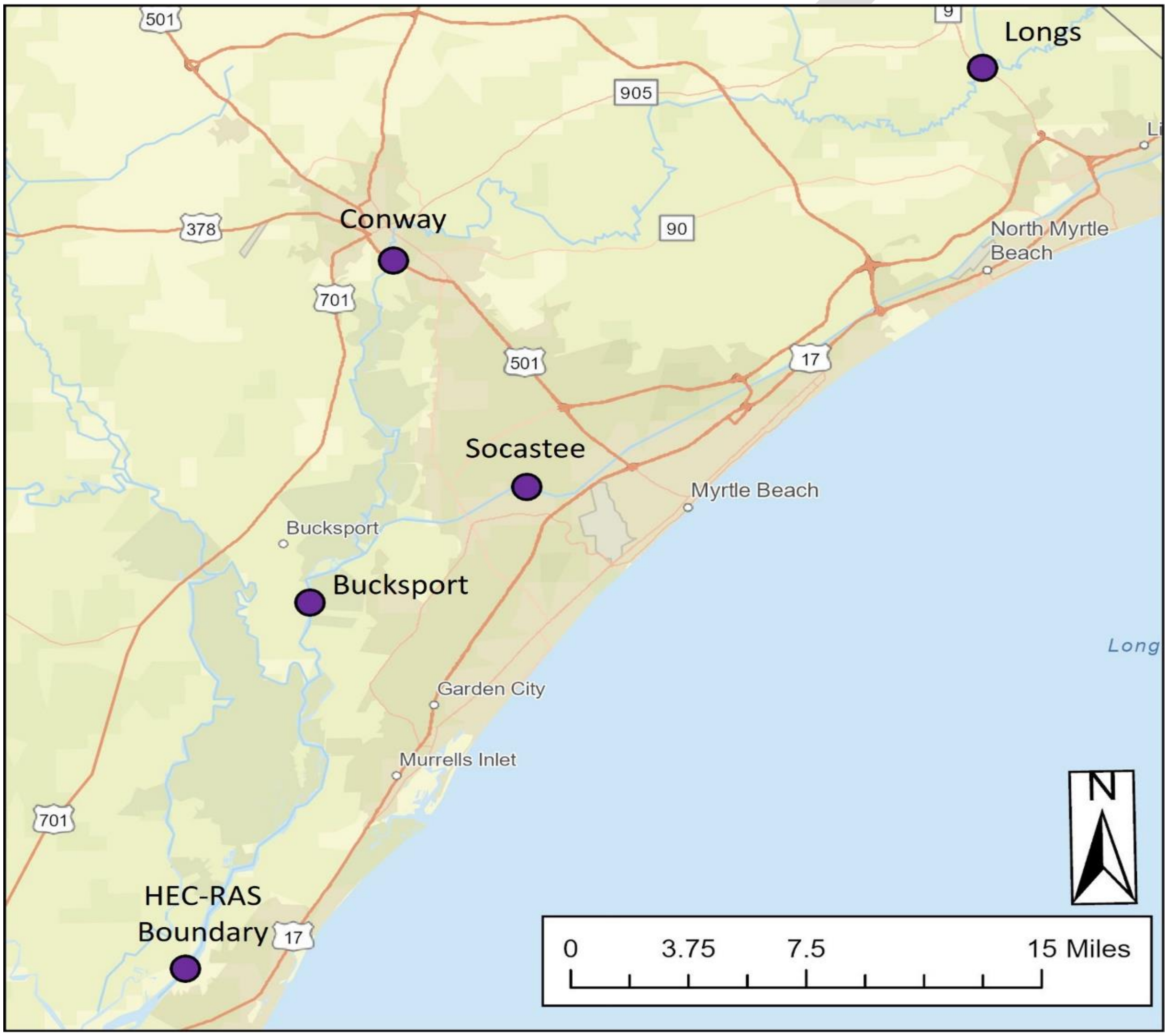


Figure 5-8. CHS Point Locations.

Table 5-2. Nonlinear SWL Residual from Storm Surge.

Location	Annual Exceedance Frequency [Average Nonlinear SWL Residual in feet per foot SLC]						
	50%	20%	10%	5%	2%	1%	0.2%

Longs	0.18	0.13	0.07	0.06	0.06	0.07	0.07
Conway	-0.03	-0.02	-0.01	-0.01	-0.01	0.00	0.00
Socastee	0.09	0.11	0.11	0.11	0.11	0.12	0.16
Bucksport	-0.04	-0.03	-0.02	-0.02	-0.01	-0.01	0.00
HEC-RAS Boundary	-0.01	0.00	0.00	-0.01	-0.02	-0.04	-0.05

The 2015 USACE Climate Change Adaptation Plan references ETL 1100-2-1 for guidance on how to plan and implement adaptation to changing sea level. Because focus areas in this study are far enough inland such that minimal effects of SLC are realized, future sea levels will thus have minimal impact on the adaptation plan.
In addition to increased AEF water levels, SLC will cause land loss throughout topographically low-lying areas. As shown in Figure 5-9, derived from the NOAA Sea Level Rise Viewer (NOAA 2024a), 1 ft of SLC would cause a large portion of the tidal marshlands to drop below MSL. This amount of SLC is less than the 50-year project sea level increase of 1.14 ft, according to the USACE Intermediate curve. Figure 5-9 also shows the MSL footprint with 3 ft of SLC, which roughly corresponds to the 50-year, High curve increase (3.06 ft) or the 100-year, Intermediate curve increase (2.72ft).

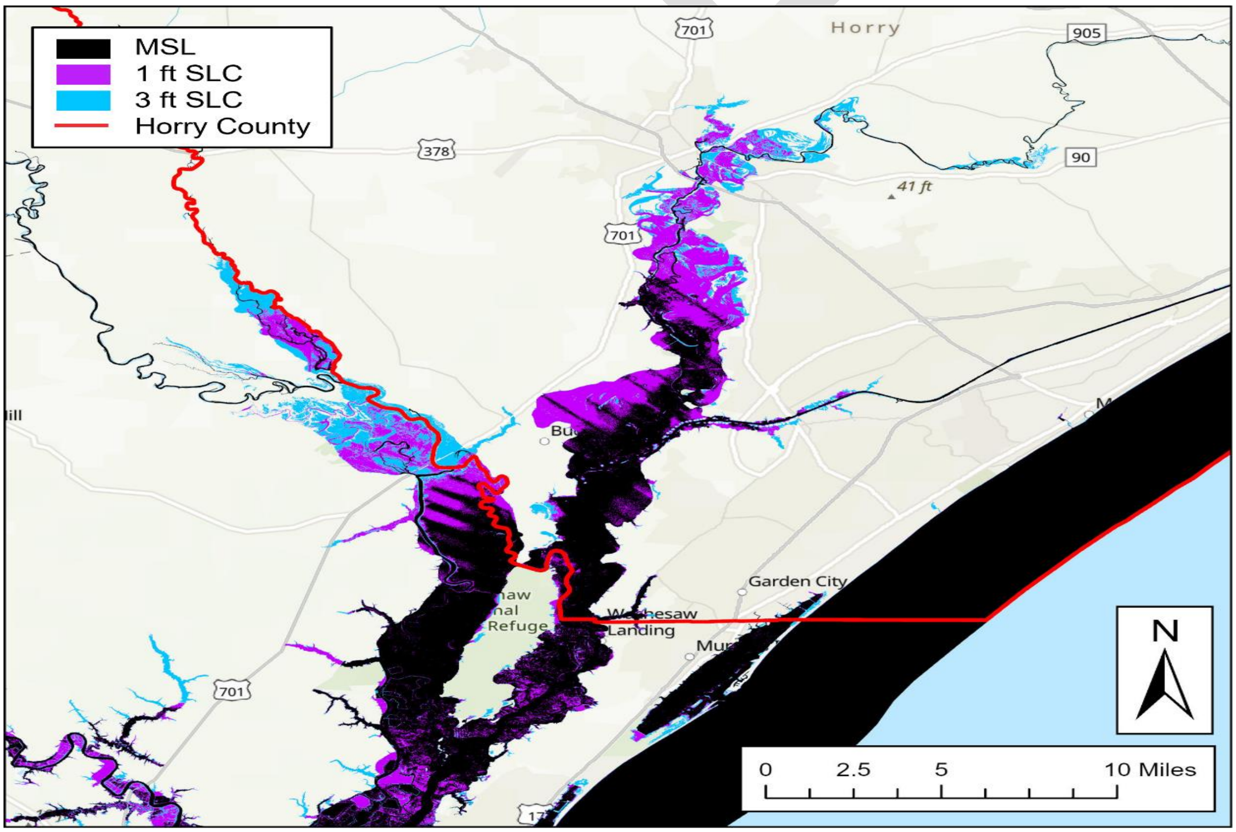


Figure 5-9. NOAA Sea Level Rise Viewer.

6. Projected Climate and Sea Level Change Summary and Conclusion

The literature review projects a strong consensus that air temperatures will rise in the study area and across the country over the next century. Most studies forecast a rise in mean annual air temperature by about 2 to 4 ºC by the second half of the 21st century for the South Atlantic-Gulf Region. However, predictions regarding changes in precipitation are more uncertain, with the studies reviewed showing an even split between anticipating increases and decreases in future annual precipitation. When it comes to streamflow projections, the outcomes are mixed as well, with some models suggesting decreases and others indicating increases for the region, based on the combination of Global Climate Models (GCMs) with macro-scale hydrological models. In summary, flooding in the project area is due to extensive rainfall throughout the year, multi-day rainstorms leading to saturated soils, a warmer Atlantic Ocean contributing to the increased rainfall and an increase in intensity and frequency of hurricanes. The projected changes and impacts to the Waccamaw River Watershed include an increase of rainstorms and extreme rainfall events causing flooding that puts people and infrastructure at risk.

Analysis of the Climate Hydrology Assessment Tool's range of model results shows a distinct upward trend in higher projections, while lower projections remain stable over time. The disparity in model outcomes widens over time, reflecting growing uncertainty the further the projections extend from the starting point. This uncertainty stems from various factors, including the initial conditions set for the GCMs, differences among GCMs themselves, and the choice of Representative Concentration Pathways (RCPs). Further uncertainties arise from the process of climate model downscaling, limited temporal resolution, and the hydrologic models themselves, as evidenced by the broad range of results depicted in Figures 3-3 to 3-4.

The USACE Vulnerability Assessment tool, applied to the project area, did not identify any exceptional vulnerabilities when compared with other Hydrologic Unit Codes (HUCs) nationwide. This watershed is considered relatively vulnerable to climate change impacts for the flood risk reduction business line, being among the 20% most vulnerable watersheds for this business line in the CONUS (202 HUC04s). The assessment pointed out flood risk vulnerabilities related to changes in flood runoff and the extent of urban areas within the 500-year floodplain.

A sensitivity analysis was conducted increasing the intensity of 1% AEP rain event and found that the model was sensitive to the increase. A 14.6% increase in total rainfall for a 96-hour event produced a rise in water surface elevation of more than 2 feet for the Waccamaw River at Conway, SC. Stronger hurricanes coupled with extreme precipitation will destroy or damage public and private buildings and property. Increased inland flooding caused by extreme precipitation events will further increase economic and agricultural losses after an event. Vulnerable populations are most at risk of flooding and may have difficulty evacuating when necessary. These results were not used to choose the recommended plan and are not included in the Economics and Benefits analysis. The results are provided in Appendix A- Hydrology and Hydraulics.

Sea level projections for the Waccamaw River basin, based on the USACE Sea Level

Tracker tool and data from the Springmaid Pier, SC NOAA station, predict varying increases by 2085 and 2135. By 2085, the Low-rate Sea level increase over the life of the project (from 2035 to 2085) was 0.17 m (0.55 ft), the Intermediate sea level increase was 0.40 m (1.32 ft), and the High sea level increase was 1.15 m (3.76 ft). For predicted SLC through year 2135, the Low-rate sea level increase (from the start of the project in 2035 to 2135) was 0.33 m (1.08 ft), the Intermediate sea level increase was 0.83 m (2.72 ft), and the High sea level increase was 2.64 m (8.67 ft) underscoring the broad range of possible future sea level scenarios.

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Appendix A3. Civil Engineering

US Army Corps
of Engineers

September 2024

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1.0 Introduction

The study area covers the Waccamaw River and its tributaries from the South Carolina state line to its confluence with the Pee Dee River. Structural and non-structural measures were developed to reduce risk from flooding. Detailed descriptions of the measures can be found in the main report, as well as the rationale for the selection of the recommended plan.

1.1 Purpose

The purpose of this Appendix is to provide civil site design considerations for the proposed structural and nonstructural measures that were considered in each focus area. Design phase considerations and general construction recommendations are discussed and will be expanded upon during optimization of the tentatively selected plan.

2.0 Structural Measures

2.1 Road Elevation

The elevation of the Pee Dee Highway in Bucksport was proposed as a structural measure. The elevation would begin at the intersection of Highway 701 and end at the intersection of Pauley Swamp Rd. The width of the road was assumed to be 40 ft wide with a 2:1 slope to existing ground on both sides to a max width of 100ft. According to H&H modeling, the road would need to be raised to an elevation of 18.5ft. Portions of the road are already at this elevation or higher. The sections of roadway that would not need to be elevated are from Station 65+00 to 73+00, Station 85+00 to 172+00, and Station 178+00 to 235+00. The total approximate linear feet of road elevation is 20,000 LF. The required amount of structural fill is approximately 111,250 cu yd. Base course and surface would also be required to replace the roadway in the elevated sections. No analysis was performed to determine if the existing roadway drainage would need to be modified. Site specific topographic data was not obtained, so actual fill quantity could vary from this estimate. A utility survey was not performed for this measure, but it is assumed that utilities along the existing roadway may need to be relocated.

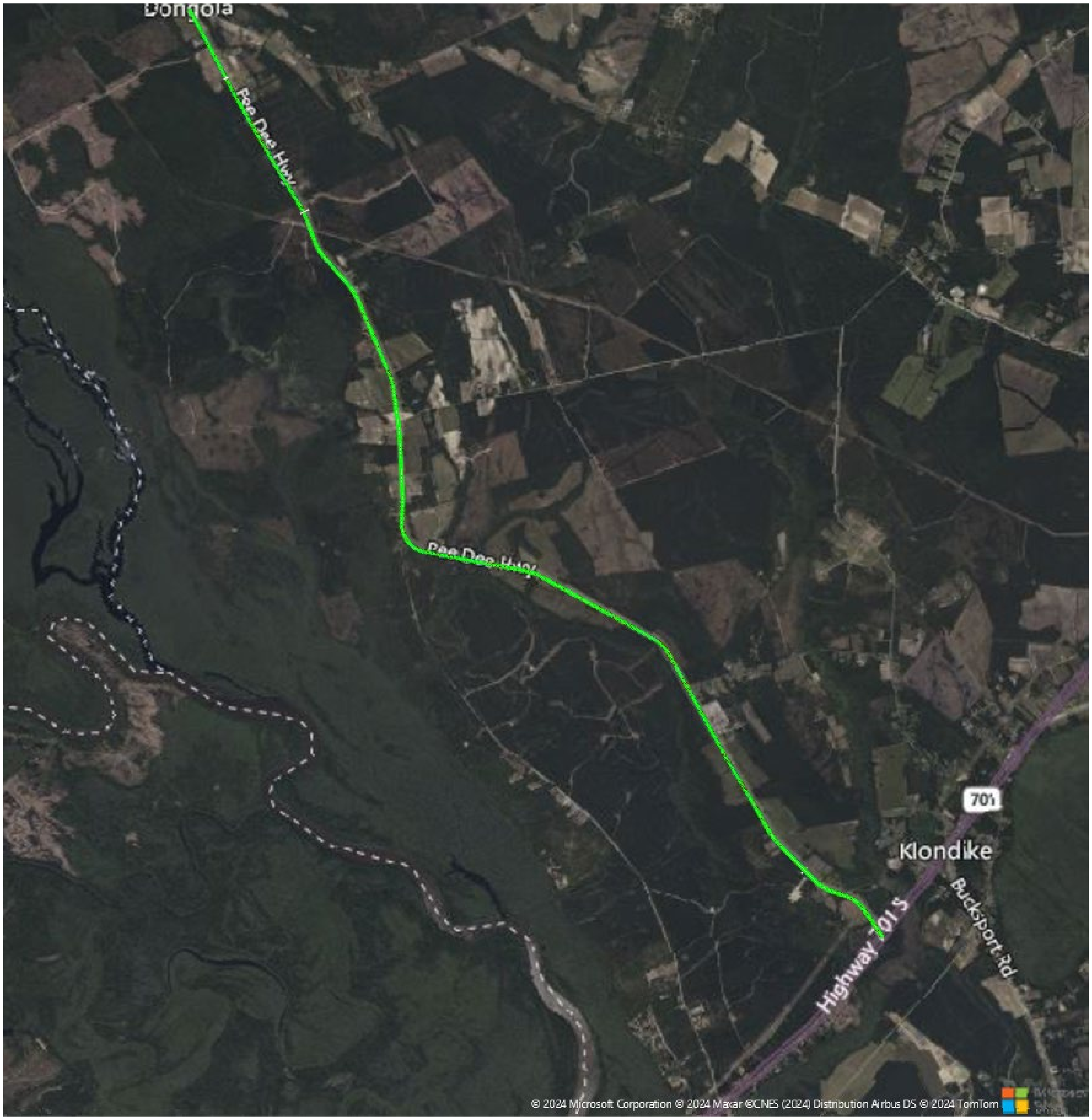


Figure 1: Pee Dee Highway Road Elevation Alignment

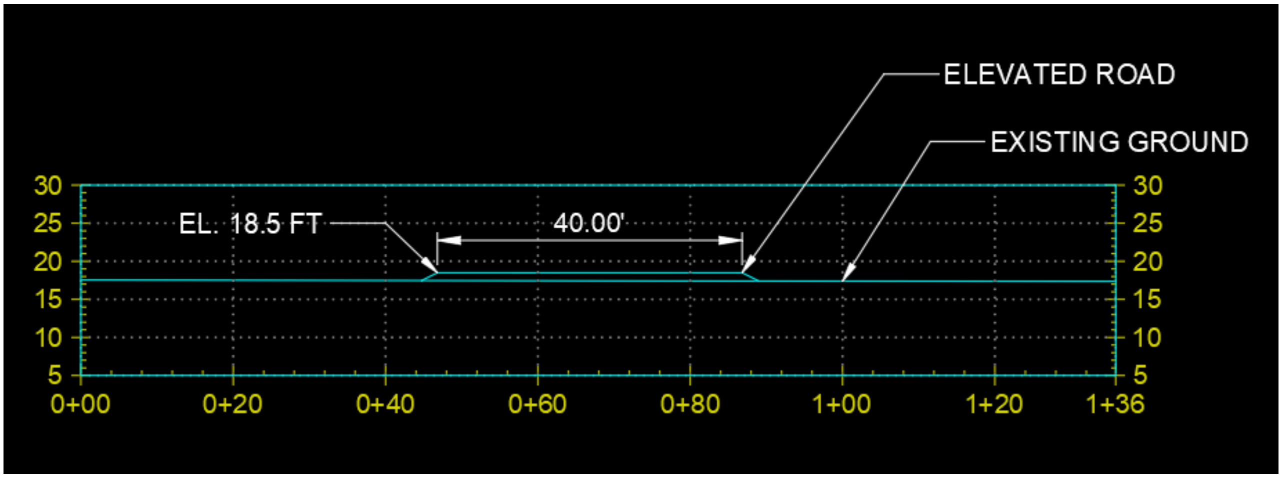
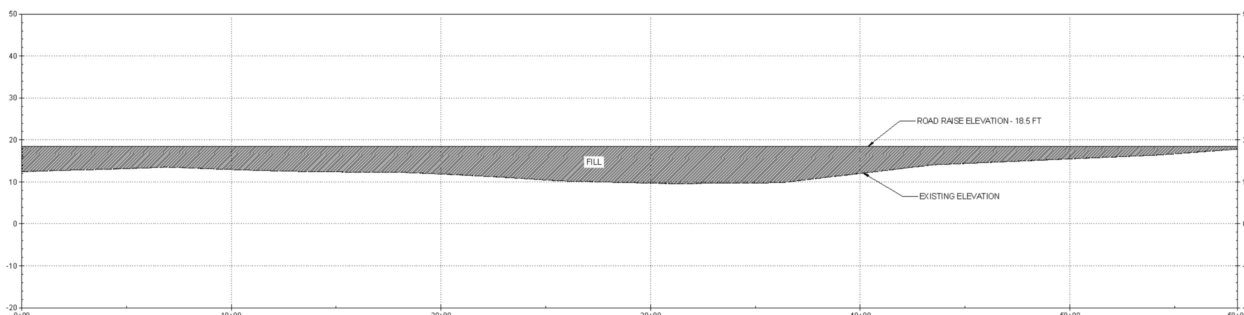
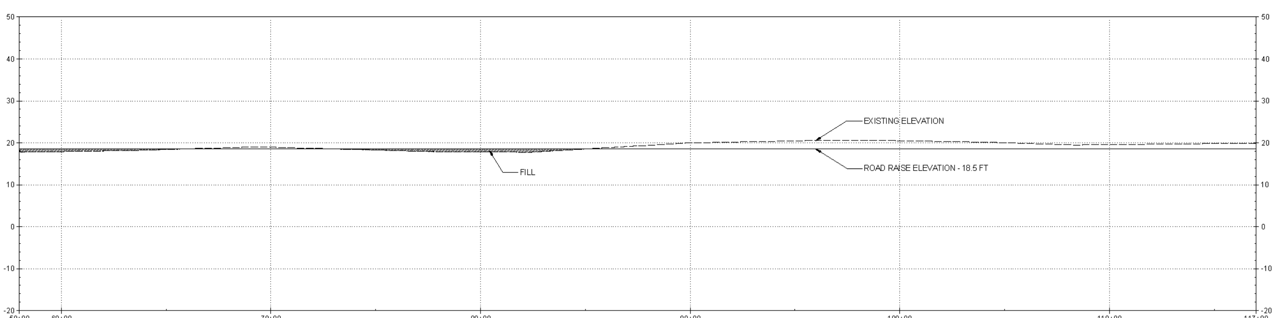


Figure 2: Typical Section of Road Elevation





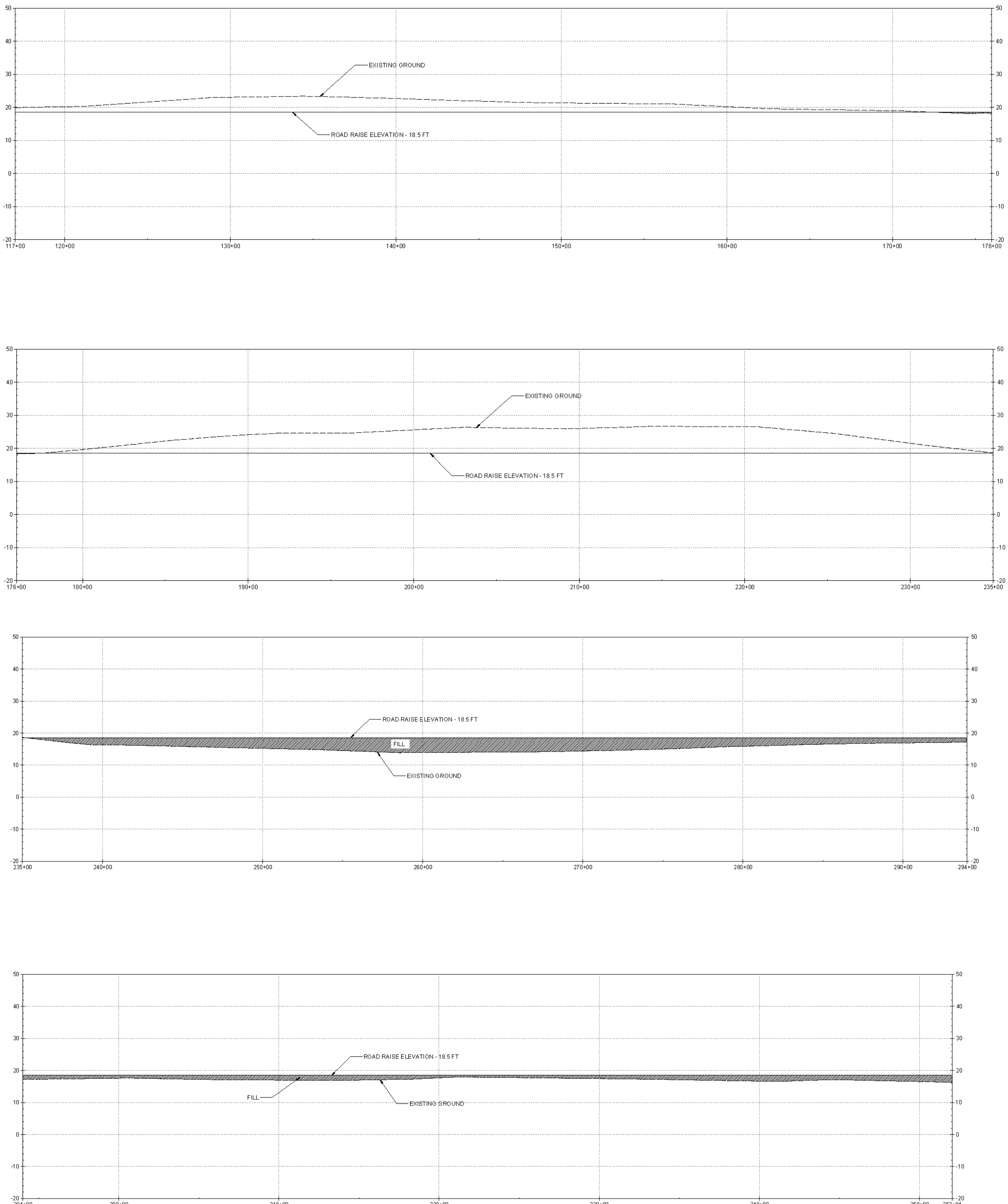


Figure 3: Road Raise Profiles

2.2 Socastee Diversion Canal and Pond

In the Socastee focus area, an excavated diversion canal from Socastee Creek with a detention/retention pond was proposed. The location of the proposed pond is near the intersection of Burcale Rd. and Fantasy Harbour Blvd. The channel would connect the pond to Socastee Creek, following a small natural stream. The channel would require a culvert to be installed under Burcale Rd. Based on H&H modeling, a pond depth of 15ft was assumed with a 3:1 side slope. The channel bottom was estimated to have a width of 20ft, with a 1:1 side slope, and 10ft depth. The estimated quantity of excavation required for the retention/detention pod is 991,870 cu yd and 14,100 cu yd for the channel. No site-specific topographic surveys were performed so the quantity of excavation could vary. No site-specific utility surveys were performed, utility relocations could be required during the construction of this measure.

Considerations during design need to be made about the ability of the in situ soil to retain water in the pond. A Geotech report provided by the non-federal sponsor for an adjacent Fire Station on Burcale Rd. indicated soft to firm fat clays (CH) ranging from 7 to 7.5ft below the surface. Very dense sands were encountered at depths 8-10ft below the surface, and interbedded silts, clays, and sands for the remainder of the estimated pond depth. Bentonite clay may be needed to mix with excavated soil to allow the pond to retain water. Excavated material would need to be hauled away from the site.

Water was not encountered in the hand auger borings at the time of drilling to a depth of 4 feet below the surface. Water levels within the cone soundings were interpreted from pore pressure readings to range from approximately 3 to 4 feet below the existing ground surface. The site is favorable for the development of shallow perched groundwater conditions due to the clayey upper soils. Dewatering would be required during excavation of the channel and pond.

Figure 4: Location of Detention/Retention Pond and Diversion Channel

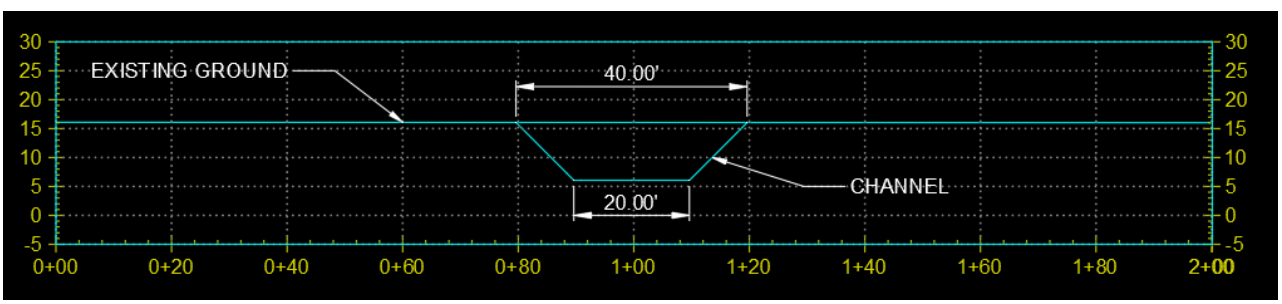


Figure 5: Typical Section of the Diversion Channel

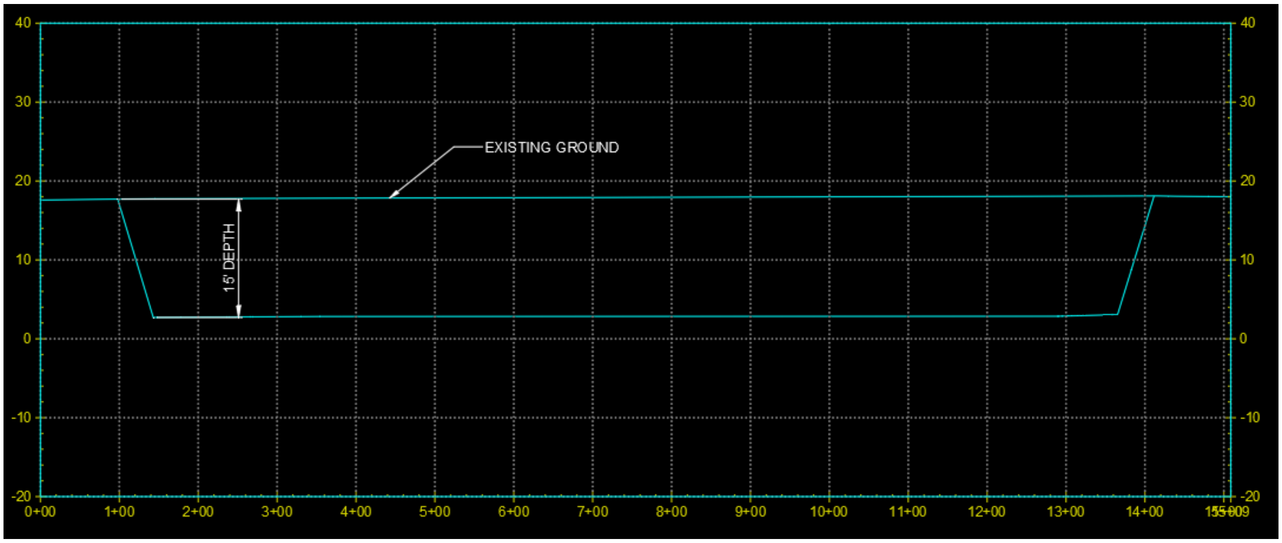


Figure 6: Detention/Retention Pond Section

2.3 Simpson Creek Benching

Benching of Simpson Creek in the Red Bluff focus area was proposed. The existing channel would be benched on the right bank with a 140ft width and 1:1 slope to existing ground, and maximum width of 200ft. The estimated quantity of excavation is 714,380 cu yd. Site-specific topographic surveys were not performed for this measure, including bathymetry data of the existing tributary. Excavation quantities are likely over-estimated due to the existing channel area not being subtracted from the terrain.

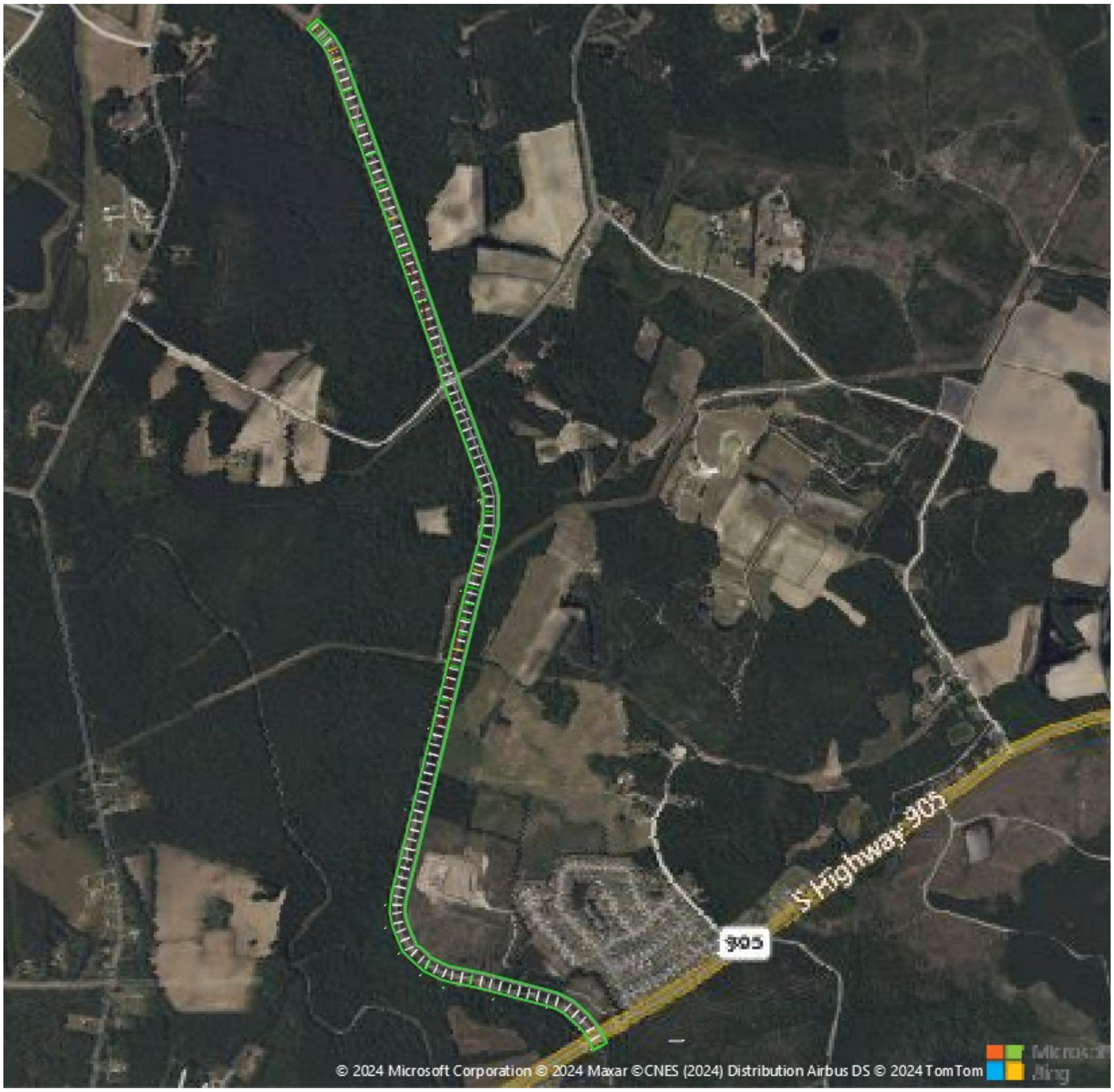


Figure 7: Simpson Creek Benching Alignment

2.4 Flood Walls

The exact alignments of the flood walls proposed for the Longs and Socastee areas have not been determined. Site specific topographic surveys would need to be performed for these measures to ensure they are tying into high ground to achieve the estimated benefits from the H&H modeling. Power line easements cross Socastee Creek, so the flood walls proposed in this focus area may interfere with the existing power lines. Utility surveys would also need to be performed to determine the extent of utility relocations required for these measures.

2.5 Conway Relief Bridges (Cross Drains)

Relief Bridges (Cross Drains) have been proposed in the Conway focus area to connect the floodplain through the roadway embankment on Highway 501, Highway 501 Business, and Highway 905. The approximate locations of the culverts that will collect flow are shown in the figures below, the exact location has not yet been determined. The culverts will be placed a minimum of 500ft away from the bridge abutments to minimize structural impacts. Cross section views of the existing terrain are also shown at each location.

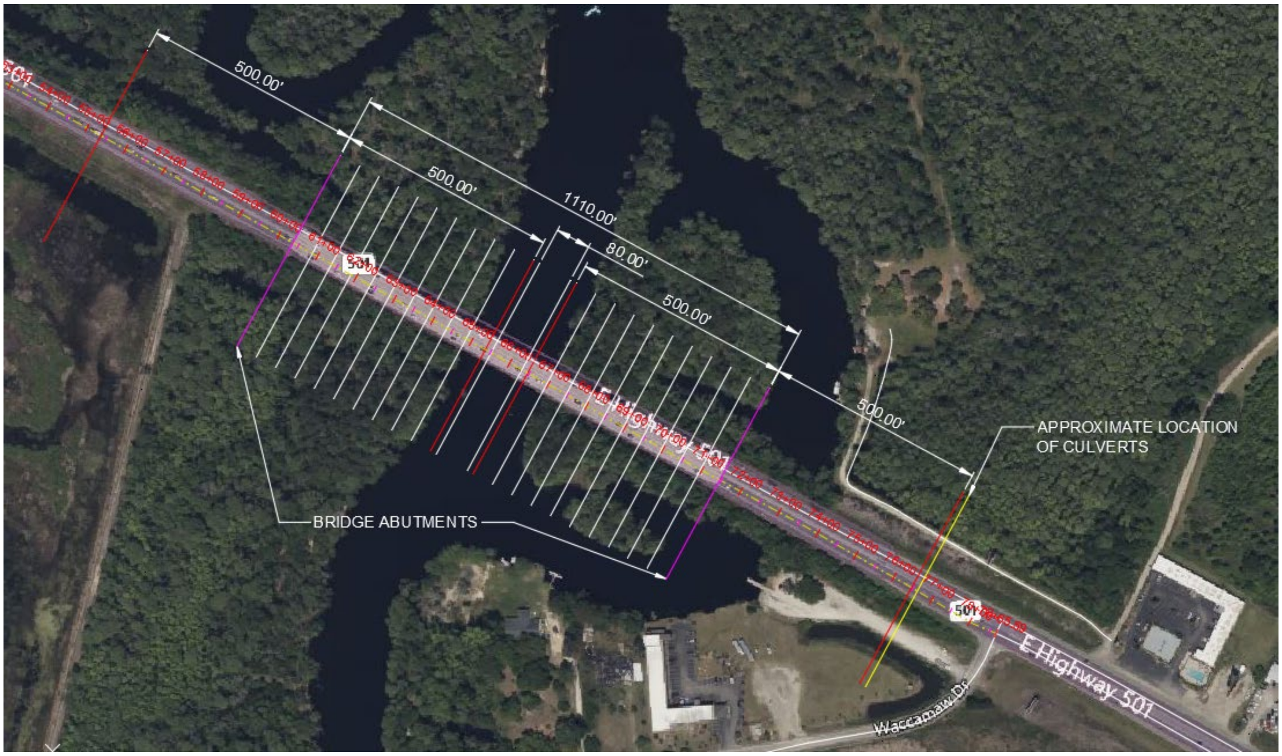


Figure 8: Highway 501 Culvert Location

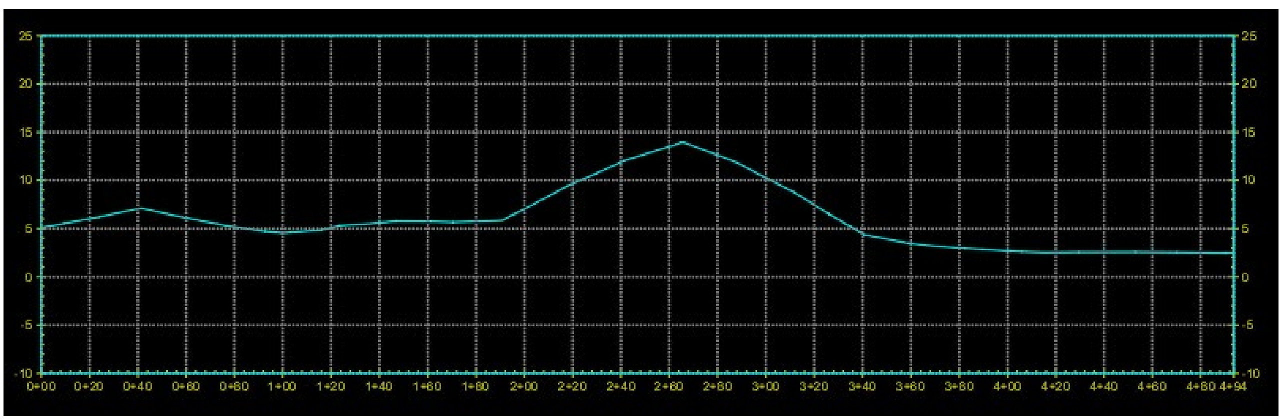


Figure 9: Highway 501 Cross Section

Figure 10: Highway 501 Business Culvert Location

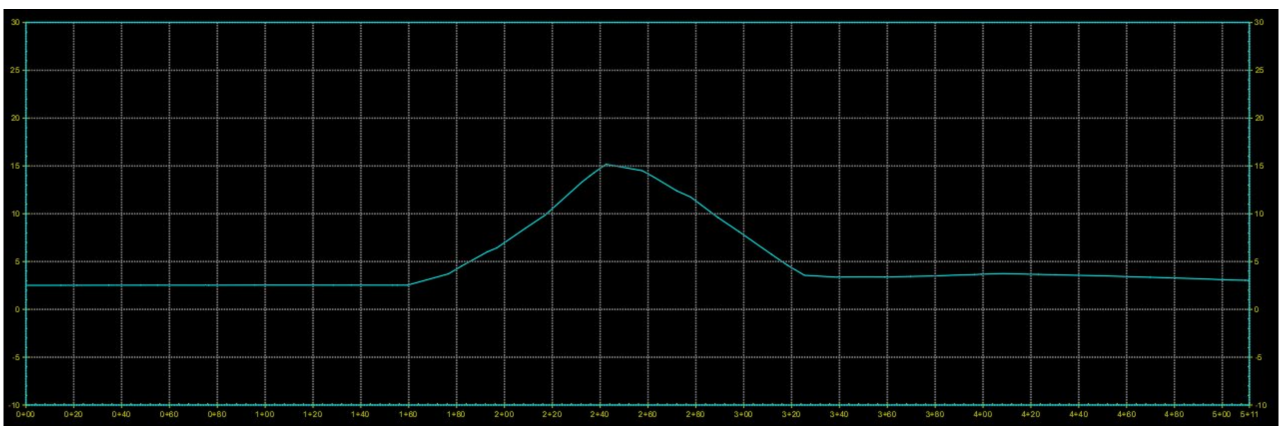


Figure 11: Highway 501 Business Cross Section

Figure 12: Highway 905 Culvert Location

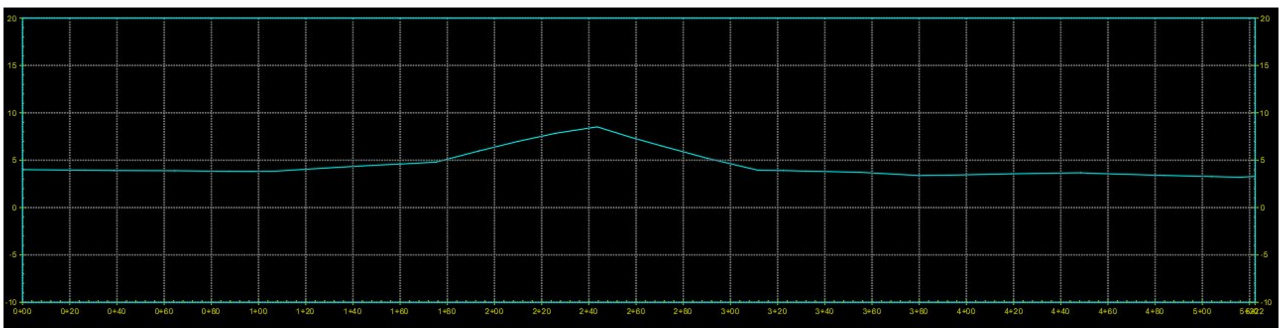


Figure 13: Highway 905 Cross Section

It is assumed that the culverts will be reinforced concrete. Utility surveys will need to be performed to determine the extent of utility relocations required for this alternative. This alternative will consist of multiple 48in. pipes to allow flow to bypass the roadway embankment. The location at Highway 905 may not be able to accommodate 48in. pipes due to the height of the roadway embankment. The design of the culverts will need to be optimized during further analysis possibly using smaller pipe sizes or a rectangular concrete culvert to convey the required amount of flow. An example of what the culverts may look like is shown in the figure below. A riprap apron, headwall, or other type of scour protection will need to be included in the design to prevent erosion of the roadway embankment.

Figure 14: Example of Reinforced Concrete Culverts

The exact depth of the culverts has not yet been determined, however, to minimize impacts to the roadway, a minimum cover depending on the type of pavement at the location is shown in the table below. The culverts are anticipated to be located near the depth of the toe of the roadway embankments.

Table 12.6.6.3-1—Minimum Cover

Type	Condition	Minimum Cover*
Corrugated Metal Pipe	—	$S/8 \geq 12.0$ in.
Spiral Rib Metal Pipe	Steel Conduit	$S/4 \geq 12.0$ in.
	Aluminum Conduit where $S \leq 48.0$ in.	$S/2 \geq 12.0$ in.
	Aluminum Conduit where $S > 48.0$ in.	$S/2.75 \geq 24.0$ in.
Structural Plate Pipe Structures	—	$S/8 \geq 12.0$ in.
Long-Span Structural Plate Pipe Structures	—	Refer to Table 12.8.3.1.1-1
Structural Plate Box Structures	—	1.4 ft. as specified in Article 12.9.1
Deep Corrugated Structural Plate Structures	—	See Article 12.8.9.4
Fiberglass Pipe	—	12.0 in.
Thermoplastic Pipe	Under unpaved areas	$ID/8 \geq 12.0$ in.
	Under paved roads	$ID/2 \geq 24.0$ in.
Steel-Reinforced Thermoplastic Culverts	—	$S/5 \geq 12.0$ in.
* Minimum cover taken from top of rigid pavement or bottom of flexible pavement		
Reinforced Concrete Pipe	Under unpaved areas or top of flexible pavement	$B_c/8$ or $B'_c/8$, whichever is greater, $\geq$ 12.0 in.
Reinforced Concrete Pipe	Under bottom of rigid pavement	9.0 in.
~~* Minimum cover taken from top of rigid pavement or bottom of flexible pavement~~		

Figure 15: AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020, Table 12.6.6.3-1

2.6 Socastee Barrier Removal

Socastee Swamp currently has two weirs constructed as a part of a previous Federal project. Removal of these weirs was proposed to reduce flood risk in the surrounding area. Exact means and methods of the removal of the weirs has not been determined, but it may require dewatering and excavation. Localized bank stabilization may also be required after the weirs have been removed to prevent erosion.

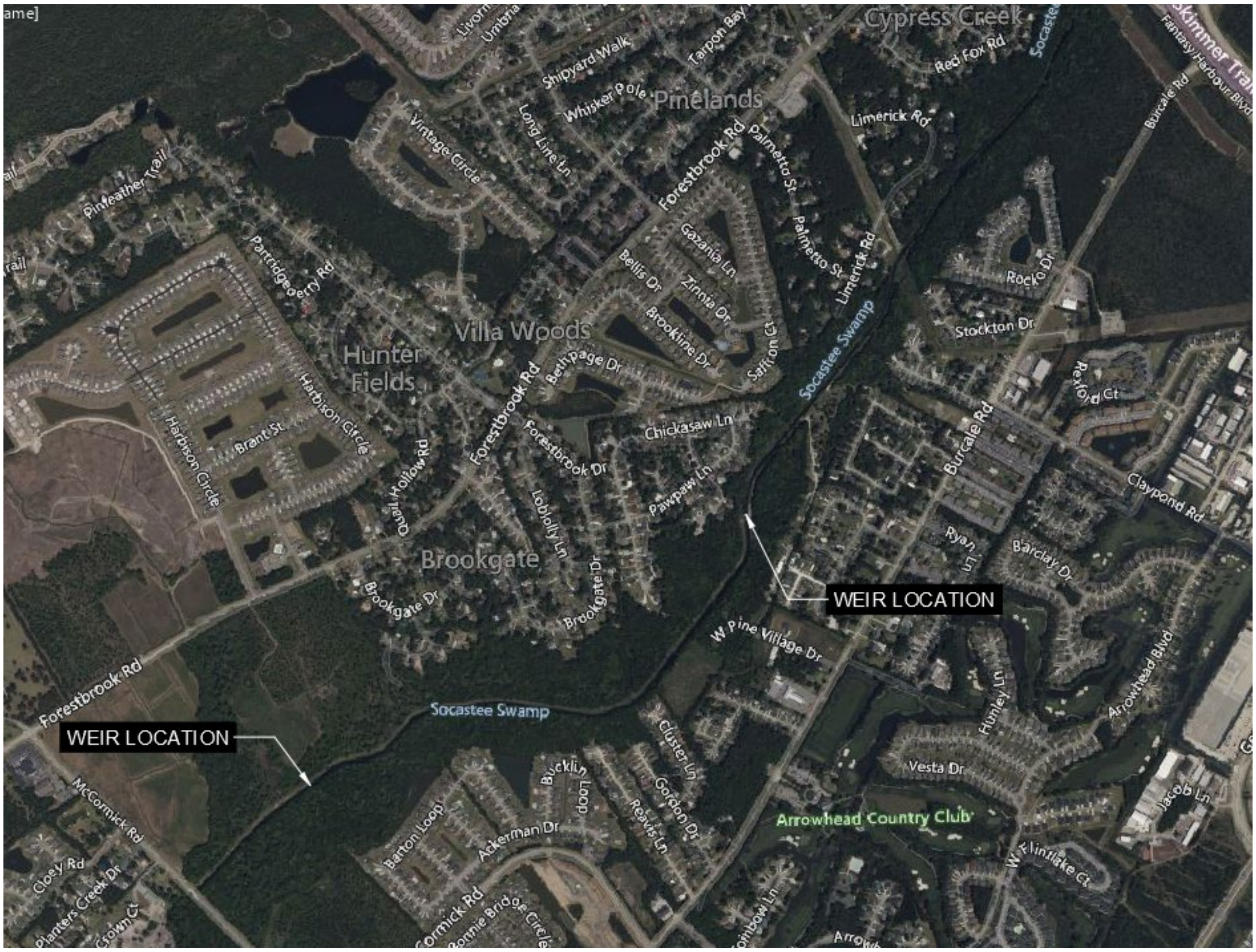


Figure 16: Location of Weirs on Socastee Swamp

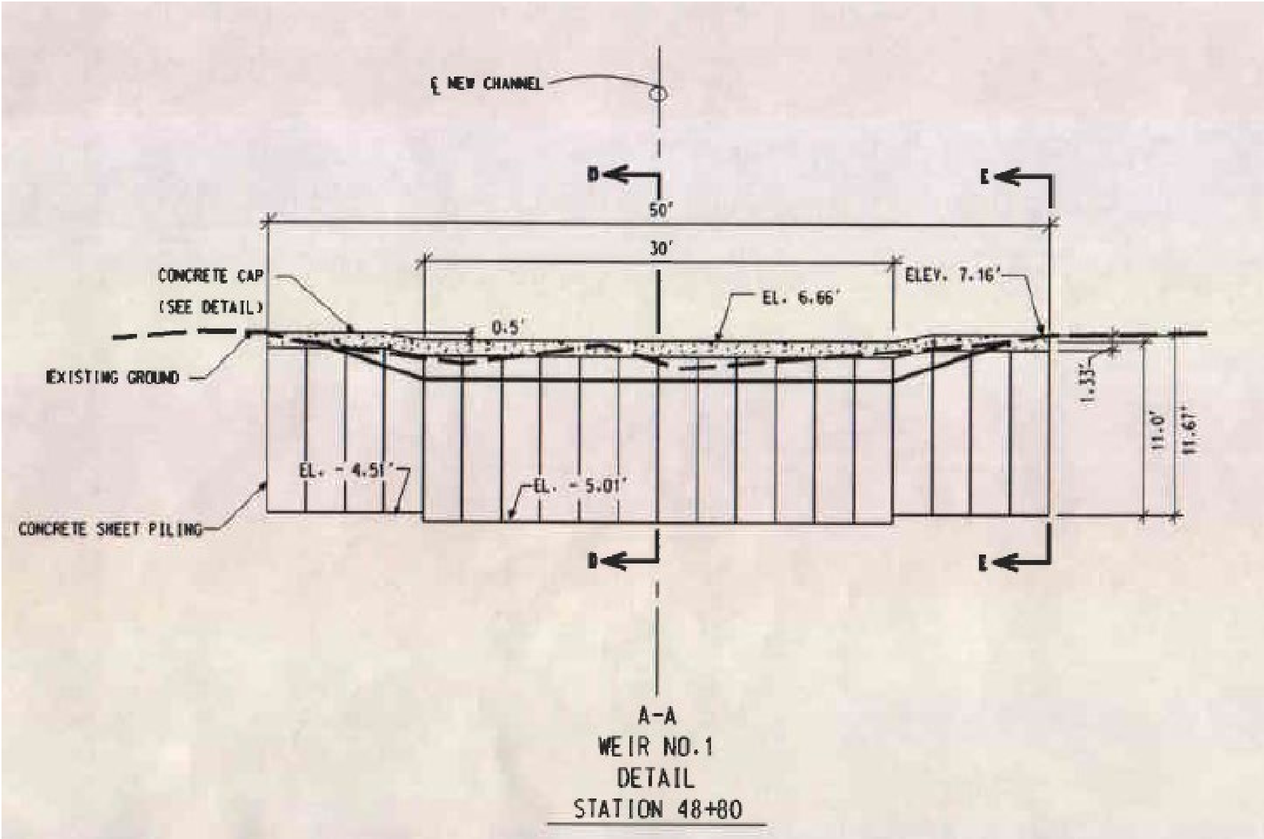


Figure 17: Downstream Weir on Socastee Swamp

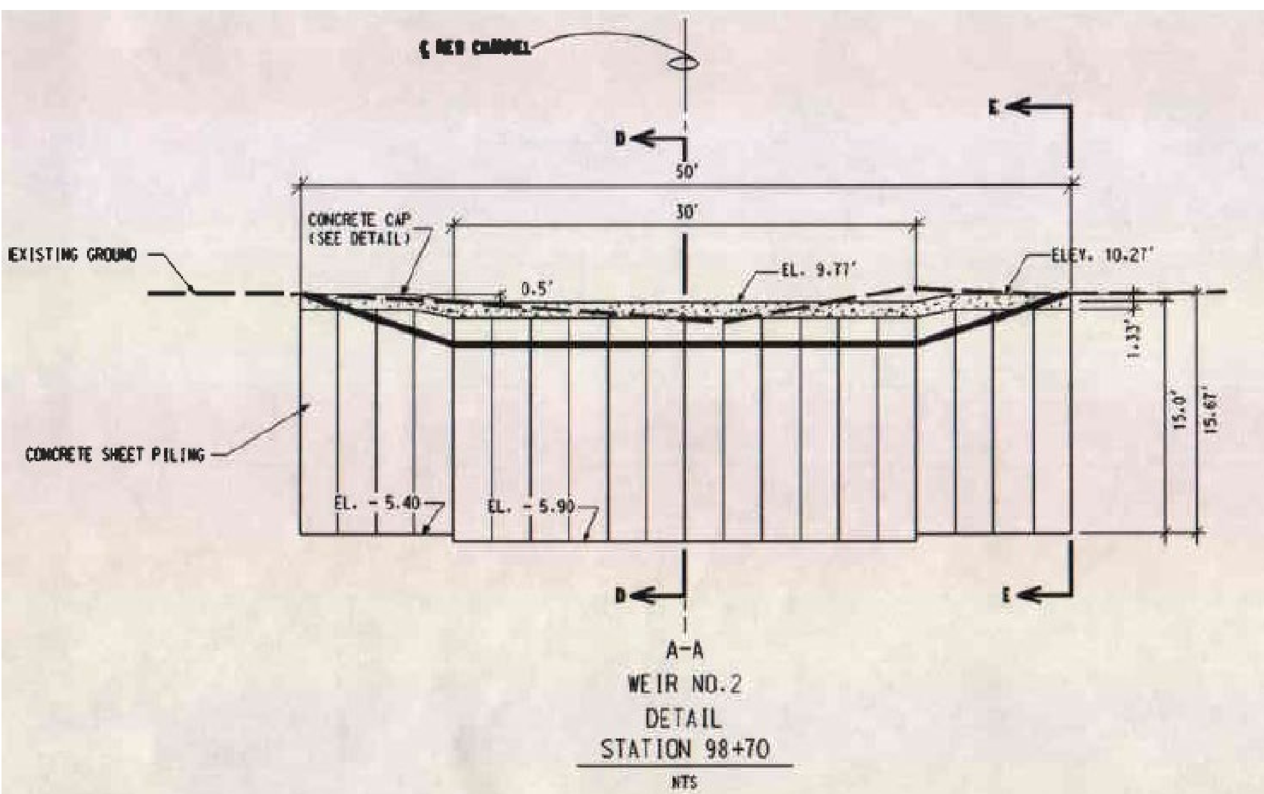


Figure 18: Upstream Weir on Socastee Swamp

2.7 Other Structural Measures

For all other structural measures not addressed above, topographic surveys would need to be performed during design. Existing utilities near the proposed excavation areas should be located prior to construction activities. The number of structures and utilities impacted will be further refined in future planning and design phases.

3.0 Nonstructural Measures

3.1 Elevation

In each of the focus areas, elevation of residential homes is being evaluated. Existing utilities near the proposed excavation areas should be located prior to construction activities. Excavation trenches near the existing structures should be graded such that rainwater does not saturate the soils beneath the existing foundation.

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1.0 Introduction

The study area covers the Waccamaw River and its tributaries from the South Carolina state line to its confluence with the Pee Dee River. Horry County (the non-federal sponsor) is situated within South Carolina's coastal plain and is bordered by North Carolina to the north and the Atlantic Ocean to the east. Detailed descriptions of the measures can be found in the main report, as well as the rationale for the selection of the recommended plan.

1.1 Purpose

The purpose of this Appendix is to provide a geological description in the general vicinity of the structural and nonstructural measures that were considered in each focus area. Design phase considerations and general construction recommendations are discussed and will be expanded upon during optimization of the tentatively selected plan.

2.0 Regional Geology

2.1 Waccamaw River Basin

In South Carolina the Piedmont Unit is separated from the Coastal Plain Unit by a "Fall Line" that begins near the Edgefield-Aiken County line and traverses to the northeast through Lancaster County. The Fall Line is an unconformity that marks the boundary between an upland region (bed rock) and a coastal plain region (sediment). The Waccamaw River Basin lies within the Coastal Plain Unit.

The Coastal Plain is underlain by Mesozoic/Paleozoic basement rock. This wedge of sediment is comprised of numerous geologic formations that range in age from the late Cretaceous Period to Recent. The sedimentary soils of these formations consist of unconsolidated sand, clay, gravel, marl, cemented sands, and limestone that were deposited over the basement rock. The basement rock consists of granite, schist, and gneiss similar to the rocks of the Piedmont Unit. Predominantly, sediments lie in nearly horizontal layers; however, erosional episodes occurring between depositions of successive layers are often expressed by undulations in the contacts between the formations.

The vertical stratigraphic sequence overlying the basement rock consists of unconsolidated Cretaceous, Paleogene, Neogene, and Quaternary sedimentary deposits. The surface deposits of the Lower Coastal Plain were formed during the Quaternary Period that began approximately 1.6 MYA and extends to present day. The Quaternary Period can be further subdivided into the Pleistocene Epoch (1.6 MYA to 10 thousand years ago) and the Holocene Epoch (10 thousand years ago to present day).

The Pleistocene Epoch is marked by the deposition of the surficial soils, the formation of the Carolina Bays and the scarps found throughout the East Coast due to sea level rise and fall. Barrier islands and flood plains along the major rivers were formed during the Holocene Epoch. The sections below show a geologic map for each focus area of the Waccamaw River Basin.

Source: SCDOT Design Manual, January 2019

2.2 Socastee

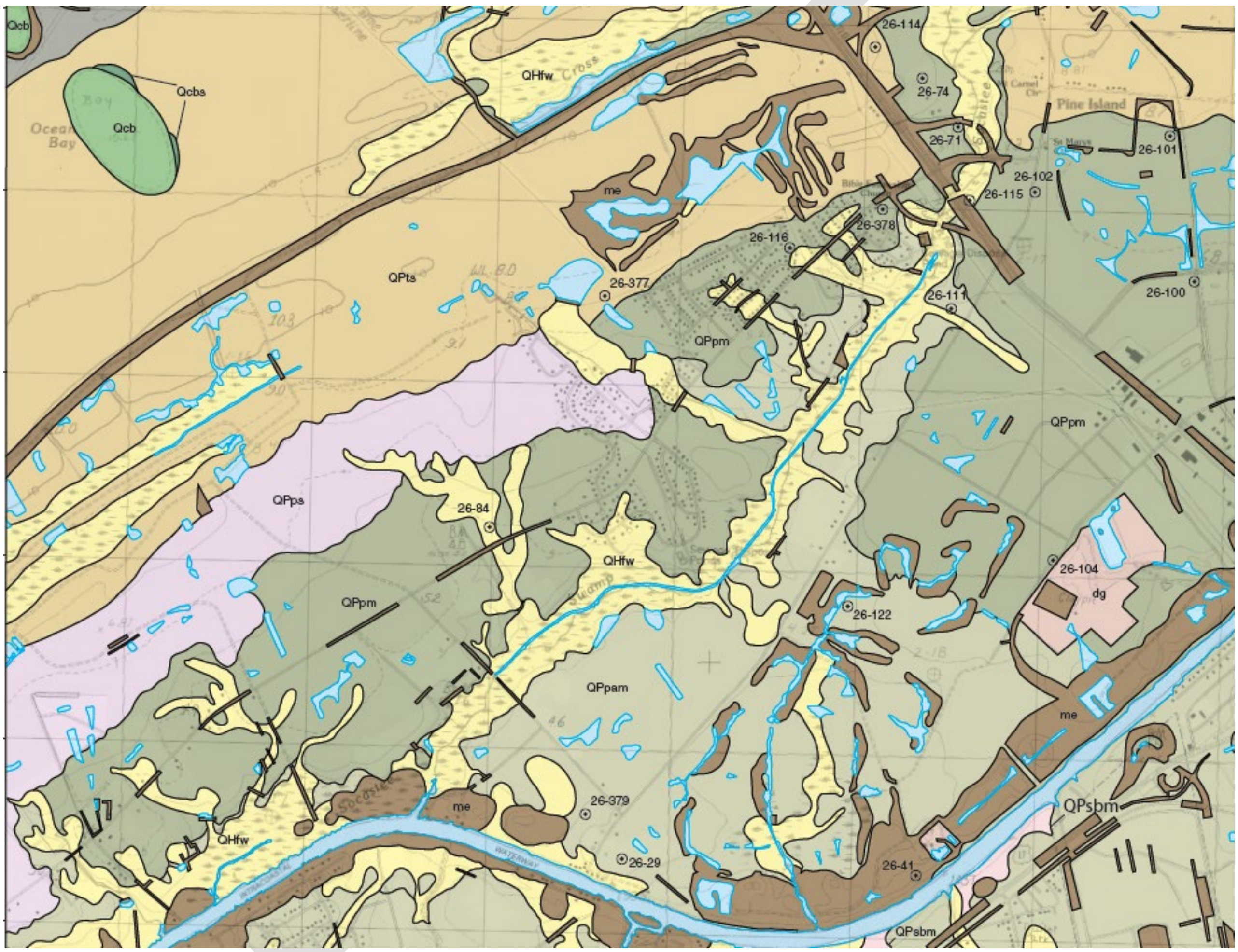
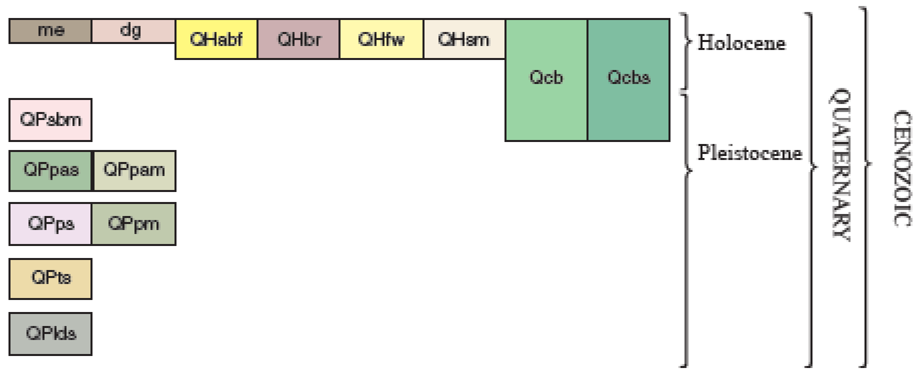


Figure 1: Socastee Focus Area Geologic Map.

CORRELATION OF MAP UNITS



The proposed structural alternatives are in the Socastee Swamp area. Below is the description of main map units for this area.

Bodies of water – Water, fresh, brackish, or salt. Water boundaries are delineated from 2006 digital ortho-quarter quadrangle photos (DOQQs).

QHfw **Freshwater marsh and swamp deposits (Holocene)** – Black (N1), silty clay and peat deposited in stream valleys and areas of locally low elevation. Deposits are identified by the organic material content, sediment type, water salinity, and ecozones. Deposits occur in areas of poor drainage, such as a swale in a dune field or the slow drainage of a stream system. The transition from a freshwater deposit to an estuarine or saltwater deposit can be variable near higher salinity waters. The variability results from changes in rainfall, inflow from groundwater lowering the salinity, and rising tides importing high salinity waters. Thickness 1 to 40 feet.

QPpam **Estuarine deposits** – Medium bluish-gray (5B 5/1), poorly sorted, subrounded to very angular, fine to very coarse quartz sand, with very fine heavy minerals to a medium light gray (N6) to medium bluish gray (5B 5/1) clayey-silty quartz sand with shells.

Source: SCDNR Geological Survey, Geologic Map of the Myrtle Beach Quadrangle, Horry County, South Carolina. W.R. Doar, III. 2014.

2.3 Longs/Red Bluff

The location of the proposed flood walls in Longs are along Buck Creek adjacent to the Aberdeen Country Club. Below is the description of main map units for this area.

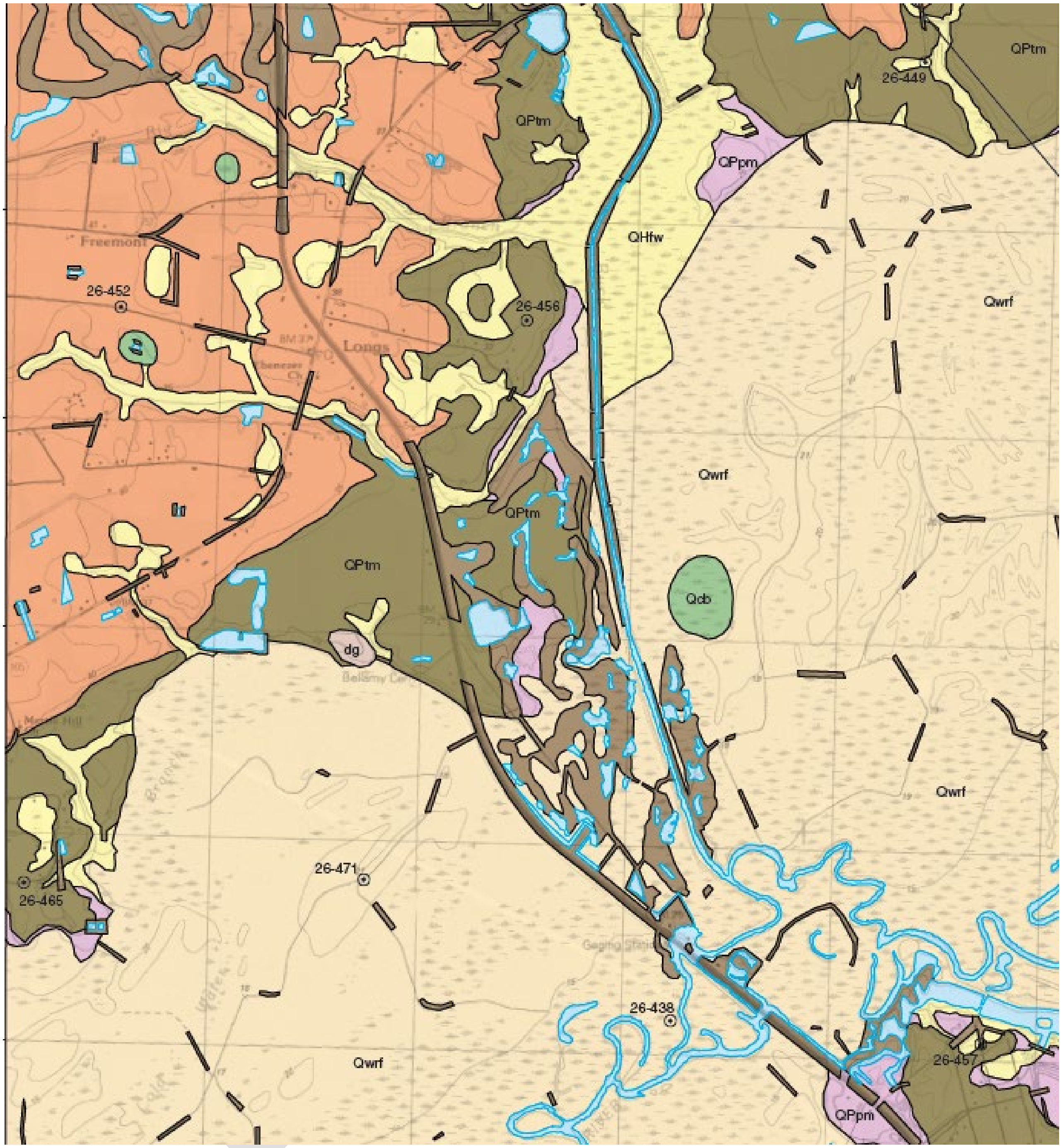
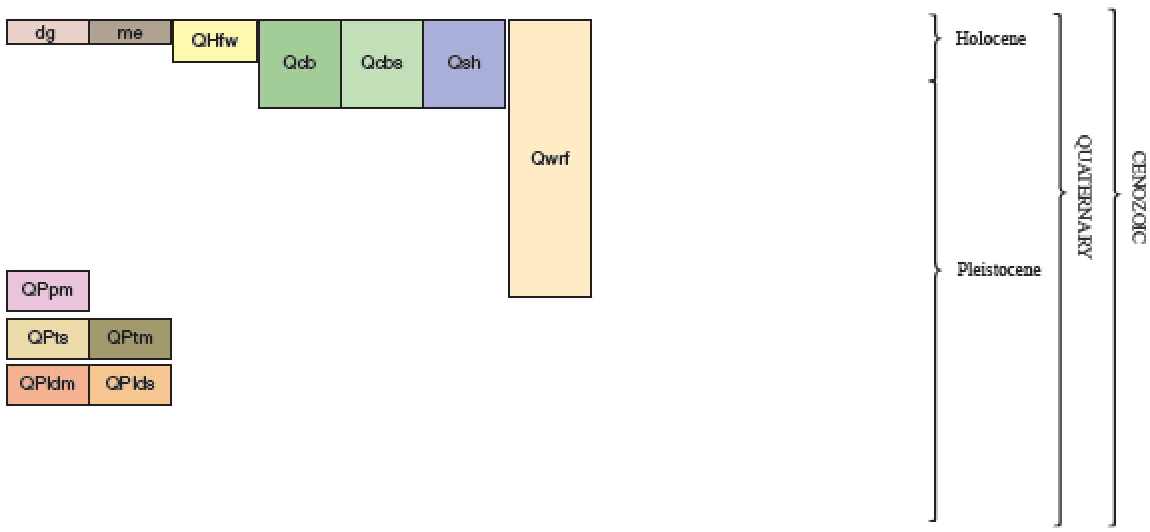


Figure 2: Longs Focus Area Geologic Map

CORRELATION OF MAP UNITS



Bodies of water – Water, fresh, brackish, or salt. Water boundaries are delineated from 2006 digital ortho-quarter quadrangle photos (DOQQs).

Waccamaw River fluvial system (Holocene to Pleistocene)

Qwrf **Waccamaw River floodplain sediments** – Clay to gravel, gray (N4-N9), medium greenish-white (5GY 4/1), brownish-white (5Y 9/1), pale brown (2.5Y 8/2-8/3), brown (5YR 4/6), and yellowish-orange (10YR 6/6-8/6), clay, silt, woody peat, and sand with granules or pebbles. The sand is poorly to very well sorted, very angular to well-rounded with occasional blocky, very fine to very coarse, quartz sand; with minor amounts of coarse blue quartz, medium jasper, iron-stained quartz, very fine- to medium garnet, rose quartz, fine olivine, very fine rutile, and opaque minerals. Comprised of non-marine sediments deposited in the Waccamaw River floodplain. These sediments vary from channel, to bar, to floodplain, to swamp facies deposits in a historically meandering river system. Thickness is 21 to 65 ft.

Pleistocene Sediments

Pleistocene stratigraphic units are interpreted to be alloformations because the surfaces bounding and separating them from other units are unconformities. The North American Commission on Stratigraphic Nomenclature (2005) defines an allostratigraphic unit as "…a mappable body of rock that is defined and identified on the basis of its bounding discontinuities."

Pamlico alloformation (Pleistocene)

Sediments of the Pamlico alloformation are generally above the elevation of 17 feet at their seaward margin where overlapped by sediments of the Princess Anne alloformation. At their landward margin, Pamlico sediments generally are below the elevation of 25 feet where the deposits overlap, overlie, or abut sediments of the Ten Mile Hill alloformation.

QPpm **Estuarine deposits (Pleistocene)** – Silt and clay, medium bluish-gray (5B 5/1), soft, silt and clay; with minor amounts of very fine quartz and phosphate sand. Thickness is 1 to 15 feet.

Ten Mile Hill alloformation (Pleistocene)

Sediments of the Ten Mile Hill alloformation are generally above the elevation of 25 feet at their seaward margin where overlapped by sediments of the Pamlico alloformation. At their landward margin Ten Mile Hill sediments generally are below the elevation of 35 feet where the deposits overlap, overlie, or abut the Ladson alloformation.

QPts **Strand deposits –** Quartz sand, light gray (N7) to dark gray (N3), sub- to well-rounded, moderately sorted, fine- to medium quartz sand; with common fine-grained heavy minerals and shell hash. Forms subdued ridges on this map. Thickness varies from 2 to 40 feet.

QPtm **Estuarine deposits –** Clay to quartz sand, yellowish-orange (10YR 6/6-7/6), pale brown (10YR 8/2), gray (N6-8), black (N-1), brown (10YR 5/4), yellow (10YR 8/4), and medium bluish-gray (5B 5/1), clay; quartz sandy clay; silty clay; silty sand; and sand. The sand is well sorted, sub- to well-rounded, very fine- to medium quartz sand; with minor amounts of coarse blue quartz, fine- to medium amethyst and epidote, and very fine opaque minerals. Forms a gently riverward-sloping plain along the Waccamaw River. Thickness is 12 to 25 ft.

QPldm **Estuarine deposits –** Silty-clayey quartz sand, sandy clay, clay, moderate yellowish brown (10YR 6/6-8/6), pale brown (2.5Y 8/4), medium brown (5YR 6/6), moderate brown (10YR 4/2), pink (10R 8/4), yellow (2.5Y 7/6-8/6), gray (N3-N7), bluish-gray (5B 5/1-7/1), and medium greenish-gray (5G 6/1-7/1), silty clay matrix supported, well sorted, but can be poorly sorted, subangular to subrounded, very fine quartz sand with medium-to-very coarse quartz sand in the poorly sorted layers; with minor very fine opaque minerals and rare fine- to medium amethyst sand; stiff clay; sandy clay. Forms a flat plain with incised creek channels on this map. Thickness is 16-26 ft.

Source: SCDNR Geological Survey, Geologic Map of the Longs Quadrangle, Horry County, South Carolina. W.R. Doar, III. 2016.

The location of the proposed benching and culverts in the Red Bluff focus area are along Simpson Creek. Below is the description of main map units for this area.

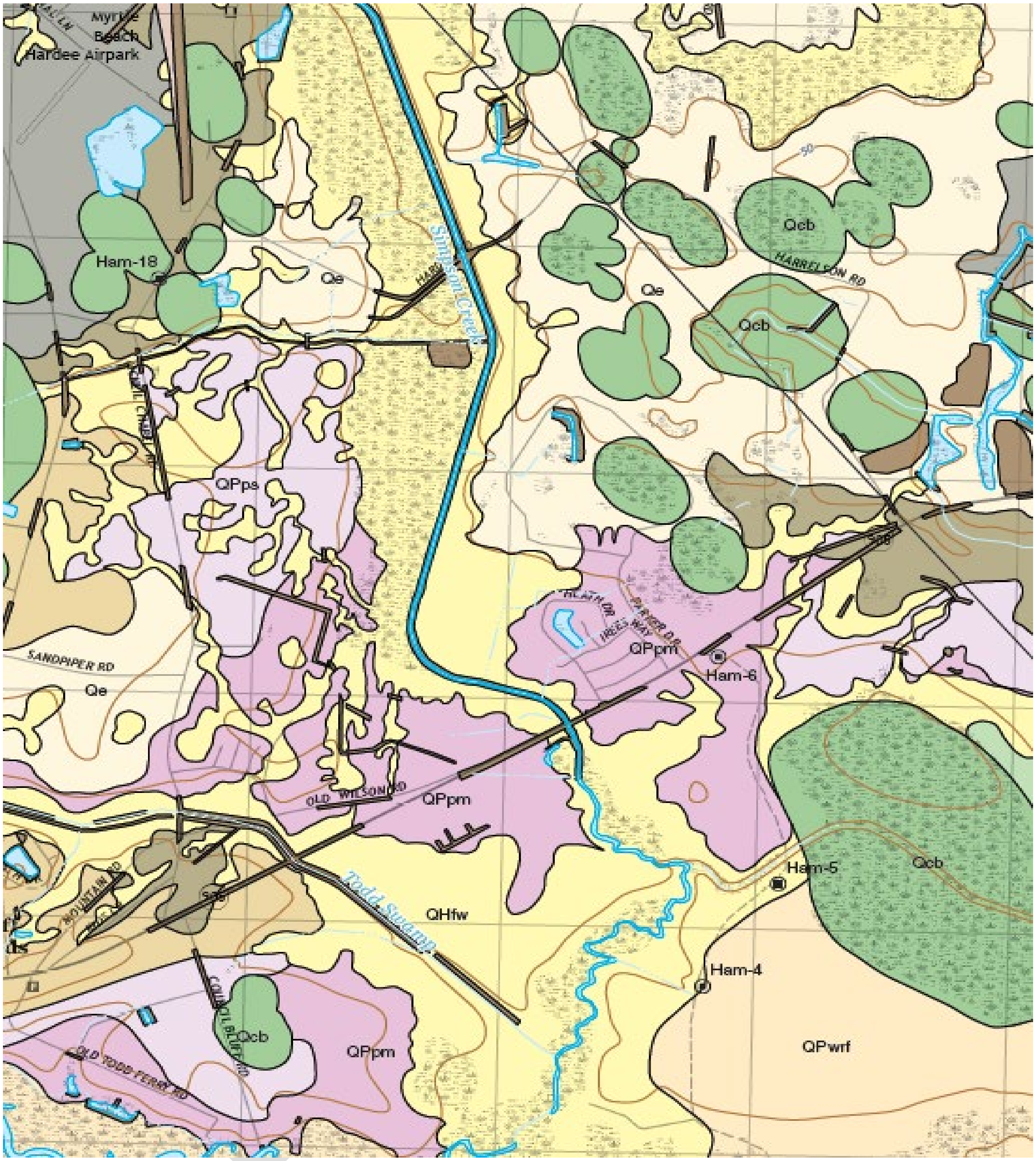
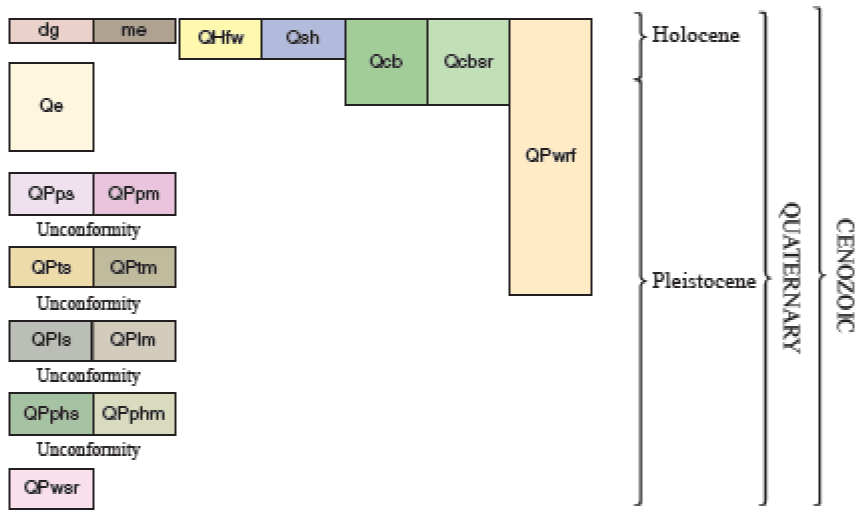


Figure 3: Red Bluff Focus Area Geologic Map

CORRELATION OF MAP UNITS



Water – Water, fresh, brackish, or salt. This designation includes altered shorelines (usually shoreline retreat or stream meanders) or flooded lands (manmade ponds) covered by water after publication of the base map. Water boundaries are delineated from 2006 digital ortho-quarter quadrangle photos (DOQQs).

QHfw **Freshwater marsh and swamp deposits (Holocene)** – Silty clay and peat, black (N1), silty clay and peat deposited in stream valleys and areas of locally low elevation. Deposits are identified by the organic material content, sediment type, and ecozones. Deposits occur in areas of poor drainage, such as a swale in a dune field or the slow drainage of a stream system. Thickness 1 to 12 feet.

Pamlico alloformation (Pleistocene)
Sediments of the Pamlico alloformation are generally above the elevation of 17 feet at their seaward margin where overlapped by sediments of the Princess Anne alloformation. At their landward margin, Pamlico sediments generally are below the elevation of 25 feet where the deposits overlap, overlie, or abut sediments of the Ten Mile Hill alloformation.

QPpm Estuarine deposits – Silt and clay, medium bluish-gray (5B 5/1) to medium gray (N5) to light red (5R 7/6), soft, silt and clay; with minor amounts of very fine quartz and phosphate sand. Thickness is 1 to 15 feet.

Source: SCDNR Geological Survey, Geologic Map of the Hammond Quadrangle, Horry County, South Carolina. W.R. Doar, III. 2017.

2.4 Conway and Bucksport

The SCDNR Geologic Quadrangle maps were not yet available for the Bucksport and Conway focus areas at the time the report was written.

3.0 Structural Measures

3.1 Flood Walls

Preliminary analysis was performed on the proposed flood walls for the Longs and Socastee focus areas to determine what type of wall would be appropriate to estimate construction costs for the TSP milestone. A conceptual analysis was performed on a sheet pile wall using USACE Computer Aided Structural Engineering (CASE) Program CWALSHT. The analysis is not complete, and the results were used for cost estimation purposes only. This measure was not carried forward to TSP.

Geotechnical reports in the vicinity of the proposed flood wall in the Socastee area were obtained from the non-federal sponsor. The boring locations are not in the exact location of the flood wall, but due to the conceptual nature of these measures, they were used to represent the soil conditions of the area. No geotechnical reports were obtained for the Longs area, so the analysis for Socastee was used to estimate costs for the Longs flood wall. While these locations are not geographically located in close proximity to each other, using the SCDNR Geological Survey maps shown above, both flood wall locations are assumed to be in similar Geologic Units, with the Socastee flood wall location having less desirable soil conditions. The Socastee flood wall was assumed to be in the Freshwater marsh and swamp deposits (Holocene) unit, and the Longs flood wall was assumed to be in the Waccamaw River floodplain sediments (Holocene to Pleistocene) unit.

Assumptions for Soils Data:
Using the Geotech Report for the New Forestbrook Fire Station at Burcale Rd:

The results from the Cone Penetration Test (CPT) were used to estimate a tip resistance and friction ratio. An average value from the three soundings for each stratum were then used to estimate a unit weight using the reference from “Estimating soil unit weight from CPT”, P.K. Robertson and K.L. Cabal, Gregg Drilling and Testing Inc., Signal Hill, California, USA. The same unit weight was used for both moist and saturated unit weight in the analysis input. The groundwater table in this area was observed to be around 3ft below the water surface, so majority of the soil is saturated in situ.

Table 1: Sounding ID C-1 Data

Sounding ID: C-1	Layer	Depth	Tip Resistance	Friction Ratio
	1	0 to -7ft	8	7
	2	-7 to -10ft	150	0.5
	3	-10 to -18	25	0.5
	4	-18 to -20	25	7
	5	-20 to -23	25	2
	6	-23 to -25	125	1

Table 2: Sounding ID C-2 Data

Sounding ID C-2	Layer	Depth	Tip Resistance	Friction Ratio
	1	0 to -7ft	8	7
	2	-7 to -12ft	125	0.5
	3	-12 to -16	25	1
	4	-16 to -20	25	4
	5	-20 to -25	25	2
	6	-25 to -26.8	250	2

Table 3: Sounding ID C-3 Data

Sounding ID C-3	Layer	Depth	Tip Resistance	Friction Ratio
	1	0 to -8ft	8	7
	2	-8 to -17ft	50	0.5
	3	-17 to -19ft	25	4
	4	-19 to -22ft	70	1
	5	-22 to -25ft	25	2
	6	-25 to -26	150	1

Table 4: Averaged Vales from CPT Data

Stratum	Depth	Tip Resistance (tsf)	Friction Ratio	γ/γ_w	γ_w	γ
1	0 to -7ft	8	7	1.8	62.4	112.3
2	-7 to -13ft	100	0.5	1.89	62.4	117.9
3	-13 to -25ft	45	2	1.94	62.4	121.1
4	-25 and below	200	1.5	2.15	62.4	134.2

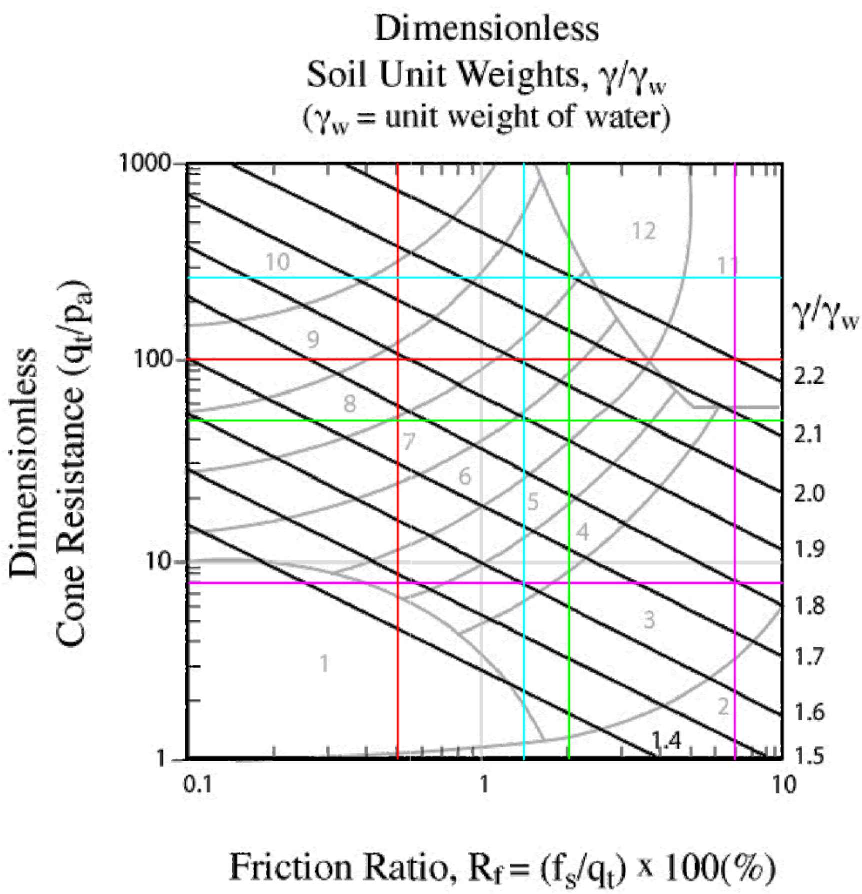


Figure 4: Estimated Unit Weight Ratio, from "Estimating soil unit weight from CPT", P.K. Robertson and K.L. Cabal, Gregg Drilling and Testing Inc., Signal Hill, California, USA.

Stratum I: Upper Soft to Firm Fat Clays
Ground Surface to 7 feet below surface
Geotech Report from US 501 had ϕ values of 18.4-18.8 for Fat Clay (CH) recorded. 18.5 was selected.
Geotech Report from US 501 had cohesion of 180 psf recorded for Fat Clay (CH). This value was selected.

Stratum II: Intermediate Medium Dense to Dense Sands
Depth of 7ft to 13ft below surface
The soils of Stratum II typically exhibited an N60 value of about 5-20, with majority being in the range of 10-30.
UFC 3-220-10 tables 8-3, 8-4 were used to estimate the ϕ at 35 based on M. Dense sand and the N_{60} values.

Stratum III: Interbedded Silts, Clays, and Sands
Depth of 13ft to 25ft below surface
The soils of Stratum III typically exhibited an N60 value of about 5-20, with majority being in the range of 0-5.
UFC 3-220-10 tables 8-3, 8-4 were used to estimate the ϕ at 30 based on loose sand

and the N_{60} values.

Stratum IV: Lower Medium Dense to Very Dense Sands
Depth of 25ft to maximum depth of 26.8 of test soundings.
The soils of Stratum II typically exhibited an N_{60} value of about 5-20, with majority being in the range of 10-30.
UFC 3-220-10 tables 8-3, 8-4 were used to estimate the ϕ at 40 based on dense to very dense sand and the N_{60} values.

Assumptions for Structural Inputs:

An analysis was performed at the tallest wall height at Station 13012.815. The water surface elevation for the 100yr_2075 is 11.281, the terrain elevation is at 2.71ft. Adding two feet of freeboard to the WSE and subtracting the terrain elevation, a max wall height of 10.571ft was calculated with the top elevation at 13.3ft. It was assumed that the ground elevations on either side of the wall were equal. A debris impact load of 500lb/ft at top of the wall was included. The calculations were performed assuming δ′ = 0 and ca = 0. This should be conservative and require greater required sheet pile depth and higher design forces in the sheet pile. A maximum head differential was used for the analysis with the flood side water elevation to top of wall and groundwater elevation at ground surface.

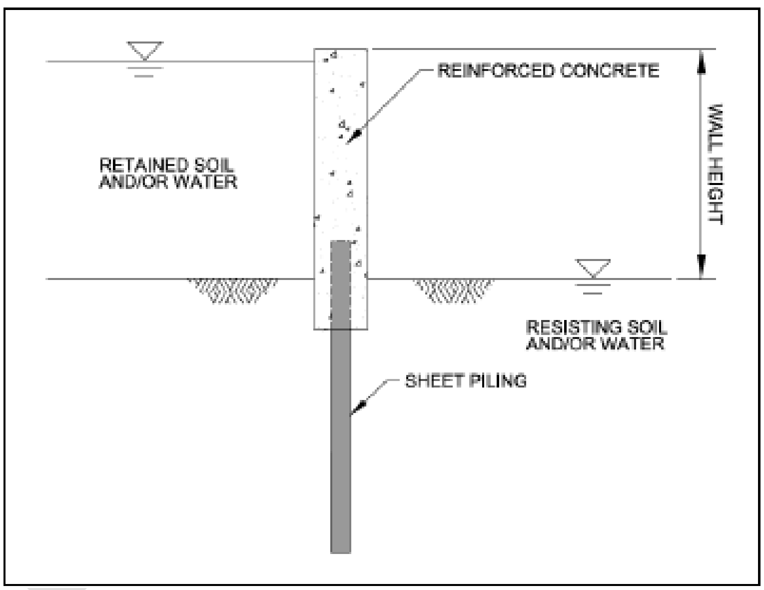


Figure 5: Example Sheet Pile Wall with Concrete Cap

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 2-AUGUST-2024 TIME: 13:57:11
****
* INPUT DATA*
****

I.--HEADING
'SOCASTEE CREEK SHEET PILE WALL DESIGN

II.--CONTROL
CANTILEVER WALL DESIGN
FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.50

III.--WALL DATA
ELEVATION AT TOP OF WALL = 13.30 FT.

IV.--SURFACE POINT DATA

IV.A.--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
50.00	2.71

IV.B.—LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
50.00	2.71

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE = 1.00
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = 1.50

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COHESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADHE-SION (PSF)	BOTTOM ELEV. (FT)	BOTTOM SLOPE (FT/FT)	SAFETY FACTOR ACT.	SAFETY FACTOR PASS.
112	112	18.5	180	0	0	-4.79	0	1	1.5
118	118	35	0	0	0	-10.79	0	1	1.5
121	121	30	0	0	0	-22.79	0	1	1.5
134	134	40	0	0	0			1	1.5

V.B.--LEFTSIDE
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURE = 1.00
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURE = 1.50

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COHESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADHE-SION (PSF)	BOTTOM ELEV. (FT)	BOTTOM SLOPE (FT/FT)	SAFETY FACTOR ACT.	SAFETY FACTOR PASS.
112	112	18.5	180	0	0	-4.79	0	1	1.5
118	118	35	0	0	0	-10.79	0	1	1.5
121	121	30	0	0	0	-22.79	0	1	1.5
134	134	40	0	0	0			1	1.5

VI.--WATER DATA
UNIT WEIGHT = 62.40 (PCF)
RIGHTSIDE ELEVATION = 13.30 (FT)
LEFTSIDE ELEVATION = 2.71 (FT)
SEEPAGE ELEVATION = 2.71 (FT)
SEEPAGE GRADIENT = AUTOMATIC

VII.--VERTICAL SURCHARGE LOADS
NONE

VIII.--HORIZONTAL LOADS

VIII.A.--HORIZONTAL LINE LOADS

ELEVATION (FT)	LINE LOAD (PLF)
13.30	500.00

VIII.B.--HORIZONTAL DISTRIBUTED LOADS
NONE

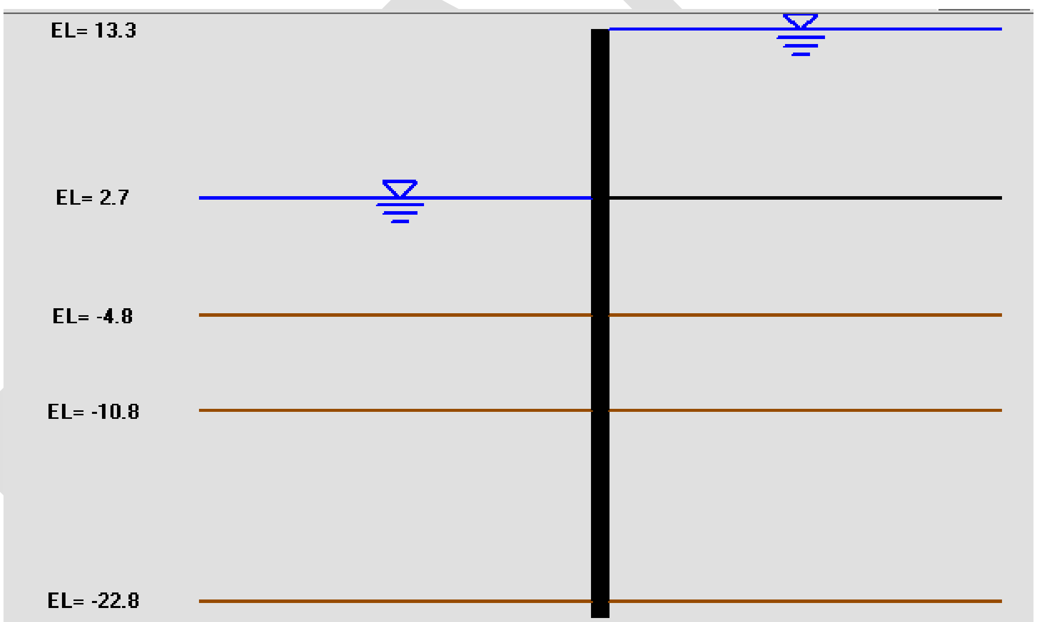


Figure 6: Sheet Pile Wall Input Plot

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 2-AUGUST-2024 TIME: 14:01:01

****
* SOIL PRESSURES FOR *
* CANTILEVER WALL DESIGN *
****

I.--HEADING
'SOCASTEE CREEK SHEET PILE WALL DESIGN

II.--SOIL PRESSURES

RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

SOIL PRESSURES ARE REPORTED FOR A SEEPAGE GRADIENT = 0.0001 AND MAY CHANGE WITH AUTOMATIC ADJUSTMENT OF THE GRADIENT.

ELEV. FT	NET WATER (PSF)	LEFTSIDE PASSIVE (PSF)	LEFTSIDE ACTIVE (PSF)	NET SOIL+ WATER ACTIVE (PSF)	NET SOIL+ WATER PASSIVE (PSF)	RIGHTSIDE ACTIVE (PSF)	RIGHTSIDE PASSIVE (PSF)
0	0	0	0	0	0	0	0
12.3	62.4	0	0	62.4	62.4	0	0
11.3	124.8	0	0	124.8	124.8	0	0
10.3	187.2	0	0	187.2	187.2	0	0
9.3	249.6	0	0	249.6	249.6	0	0
8.3	312	0	0	312	312	0	0
7.3	374.4	0	0	374.4	374.4	0	0
6.3	436.8	0	0	436.8	436.8	0	0
5.3	499.2	0	0	499.2	499.2	0	0
4.3	561.6	0	0	561.6	561.6	0	0
3.3	624	0	0	624	624	0	0
2.7+	660.8	0	0	660.8	660.8	0	0
2.7-	660.8	299.4	0	361.4	960.2	0	299.4
2.3	660.8	331.1	0	329.7	991.9	0	331.1
1.7	660.8	376.6	0	284.2	1037.5	0	376.7
1.3	660.8	408.3	0	252.5	1069.1	0	408.3
0.3	660.8	485.5	0	175.3	1146.3	0	485.5
-0.7	660.8	562.7	0	98.1	1223.5	0	562.7
-1.7	660.8	639.9	0	20.9	1300.7	0	640
-2.0	660.8	660.8	0	0	1321.6	0	660.8
-2.7	660.7	717.1	0	-56.3	1377.9	0	717.2
-3.7	660.7	794.3	0	-133.5	1455.1	0	794.4
-4.7	660.7	871.5	0	-210.7	1532.3	0	871.6
-4.8+	660.7	878.4	0	-186.7	1508.4	0	878.6

-4.8-	660.7	917.3	100.8	-186.7	1508.4	100.8	917.5
-5.7	660.7	1042	114.5	-266.8	1588.5	114.5	1042.3
-6.7	660.7	1179.2	129.6	-388.9	1710.6	129.6	1179.4
-7.7	660.7	1316.3	144.6	-510.9	1832.6	144.7	1316.6
-8.7	660.7	1453.4	159.7	-632.9	1954.7	159.7	1453.7
-9.7	660.7	1590.5	174.8	-755	2076.7	174.8	1590.8
-10.7	660.6	1727.6	189.8	-877	2198.8	189.9	1728
-10.8+	660.6	1739.9	191.2	-744.3	2066.1	191.2	1740.3
-10.8-	660.6	1496.5	235.2	-744.3	2066.1	235.2	1496.9
-11.7	660.6	1609.6	252.9	-696	2017.7	253	1610
-12.7	660.6	1733.9	272.5	-800.7	2122.4	272.5	1734.3
-13.7	660.6	1858.2	292	-905.5	2227.2	292.1	1858.6
-14.7	660.6	1982.5	311.5	-1010.3	2332	311.6	1982.9
-15.7	660.6	2106.8	331.1	-1115	2436.8	331.1	2107.2
-16.7	660.6	2231	350.6	-1219.8	2541.5	350.7	2231.6
-17.7	660.6	2355.3	370.1	-1324.5	2646.3	370.2	2355.9
-18.7	660.5	2479.6	389.7	-1429.3	2751.1	389.8	2480.2
-19.7	660.5	2603.9	409.2	-1534.1	2855.8	409.3	2604.5
-20.7	660.5	2728.2	428.7	-1638.8	2960.6	428.8	2728.8
-21.7	660.5	2852.5	448.3	-1743.6	3065.4	448.4	2853.1
-22.7	660.5	2976.8	467.8	-1848.4	3170.1	467.9	2977.4
-22.8+	660.5	2987.9	469.5	-2493.5	3815.4	469.7	2988.6
-22.8-	660.5	4096.1	306.3	-2493.5	3815.4	306.4	4097
-23.7	660.5	4285.5	320.5	-3304.5	4626.5	320.5	4286.5
-24.7	660.5	4493.7	336	-3497.1	4819.1	336.1	4494.7
-25.7	660.5	4701.9	351.6	-3689.7	5011.8	351.7	4702.9
-26.7	660.4	4910	367.2	-3882.4	5204.4	367.2	4911.1
-27.7	660.4	5118.2	382.7	-4075	5397	382.8	5119.3
-28.7	660.4	5326.4	398.3	-4267.6	5589.7	398.4	5327.6
-29.7	660.4	5534.6	413.9	-4460.2	5782.3	414	5535.8

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 2-AUGUST-2024 TIME: 14:01:11

****
* SUMMARY OF RESULTS FOR *
* CANTILEVER WALL DESIGN *
****

I.--HEADING
'SOCASTEE CREEK SHEET PILE WALL DESIGN

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

WALL BOTTOM ELEV. (FT): -26.55
PENETRATION (FT): 29.26

MAX. BEND. MOMENT (LB-FT): 7.4808E+04
AT ELEVATION (FT): -13.37

MAX. SCALED DEFL. (LB-IN^3): 6.2180E+10
AT ELEVATION (FT): 13.30

SEEPAGE GRADIENT: 0.1808

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN^4 TO OBTAIN DEFLECTION IN INCHES.

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS
DATE: 2-AUGUST-2024 TIME: 14:01:11

****
* COMPLETE OF RESULTS FOR*
* CANTILEVER WALL DESIGN*
****

I.--HEADING
'SOCASTEE CREEK SHEET PILE WALL DESIGN

II.—RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	DEFLECTION (LB-IN^3)	PRESSURE (PSF)
13.3	0	500	62180000000	0
12.3	510.4	531	59603000000	62.4
11.3	1083.2	625	57026000000	124.8
10.3	1780.8	781	54451000000	187.2

9.3	2665.6	999	51879000000	249.6
8.3	3800	1280	49312000000	312
7.3	5246.4	1623	46751000000	374.4
6.3	7067.2	2029	44199000000	436.8
5.3	9324.8	2497	41660000000	499.2
4.3	12082	3027	39137000000	561.6
3.3	15400	3620	36635000000	624
2.71	17647	3999	35171000000	660.82
-2.71	17647	3999	35171000000	361.38
2.3	19316	4140	34160000000	327.68
1.71	21813	4319	32714000000	279.17
1.3	23606	4427	31718000000	245.46
0.3	28142	4631	29316000000	163.25
-0.7	32841	4753	26964000000	81.04
-1.69	37553	4793	24700000000	0
-1.7	37621	4793	24668000000	-1.17
-2.7	42400	4751	22438000000	-83.38
-3.7	47096	4627	20280000000	-165.59
-4.7	51626	4420	18204000000	-247.8
-4.79	52023	4401	18021000000	-174.3
-5.7	55952	4232	16217000000	-196.89
-6.7	60067	3978	14327000000	-310.62
-7.7	63871	3611	12540000000	-424.35
-8.7	67250	3129	10864000000	-538.08
-9.7	70092	2535	9303900000	-651.81
-10.7	72281	1826	7864900000	-765.54
-10.79	72443	1762	7741400000	-653.6
-11.7	73780	1182	6550600000	-622.1
-12.7	74634	510	5363700000	-721.74
-13.7	74766	-262	4305700000	-821.37
-14.7	74077	-1133	3376800000	-921.01
-15.7	72467	-2104	2575800000	-1020.65
-16.7	69836	-3174	1899800000	-1120.28
-17.7	66085	-4344	1344300000	-1219.92
-18.7	61114	-5614	902890000	-1319.56
-19.7	54823	-6984	566860000	-1419.2
-20.7	47113	-8453	325370000	-1518.83
-21.57	39188	-9809	182040000	-1605.37
-21.7	37885	-10008	165060000	-1416.51
-22.7	27408	-10706	70033000	20.03
-22.79	26445	-10699	64077000	149.32

-23.7	16952	-9968	22368000	1456.58
-24.7	7951.3	-7793	4204000	2893.13
-25.7	1844.3	-4182	196950	4329.68
-26.55	0	0	0	5546.21

NOTE: DIVIDE SCALED DEFLECTION MODULUS OF
ELASTICITY IN PSI TIMES PILE MOMENT
OF INERTIA IN IN^4 TO OBTAIN DEFLECTION
IN INCHES.

III.--WATER AND SOIL PRESSURES

ELEVATION (FT)	**WATER PRESSURE (PSF)**	**SOIL PRESSURE LEFTSIDE PASSIVE (PSF)**	**SOIL PRESSURE LEFTSIDE ACTIVE (PSF)**	**SOIL PRESSURE RIGHTSIDE ACTIVE (PSF)**	**SOIL PRESSURE RIGHTSIDE PASSIVE (PSF)**
13.30	0	0	0	0	0
12.30	62	0	0	0	0
11.30	125	0	0	0	0
10.30	187	0	0	0	0
9.30	250	0	0	0	0
8.30	312	0	0	0	0
7.30	374	0	0	0	0
6.30	437	0	0	0	0
5.30	499	0	0	0	0
4.30	562	0	0	0	0
3.30	624	0	0	0	0
2.71+	661	0	0	0	0
2.71-	661	299	0	0	299
2.30	652	324	0	0	338
1.71	638	359	0	0	394
1.30	629	384	0	0	433
0.30	606	443	0	0	528
-0.70	584	503	0	0	623
-1.69	562	562	0	0	716
-1.70	561	562	0	0	717
-2.70	539	622	0	0	812
-3.70	516	682	0	0	907
-4.70	494	741	0	0	1002
-4.79+	492	747	0	0	1010
-4.79-	492	709	78	124	1126
-5.70	471	808	89	140	1276
-6.70	448	917	101	158	1441

-7.70	426	1027	113	176	1606
-8.70	403	1136	125	195	1771
-9.70	381	1245	137	213	1936
-10.70	358	1355	149	231	2101
-10.79+	356	1364	150	232	2116
-10.79-	356	1174	184	286	1820
-11.70	336	1265	199	307	1955
-12.70	313	1365	215	330	2103
-13.70	290	1466	230	354	2251
-14.70	268	1566	246	377	2399
-15.70	245	1666	262	400	2548
-16.70	223	1767	278	424	2696
-17.70	200	1867	293	447	2844
-18.70	178	1967	309	470	2992
-19.70	155	2068	325	494	3141
-20.70	132	2168	341	517	3289
-21.57	113	2255	354	537	3418
-21.70	110	2268	356	540	3437
-22.70	87	2369	372	563	3585
-22.79+	85	2378	374	566	3599
-22.79-	85	3260	244	369	4933
-23.70	65	3419	256	385	5153
-24.70	42	3595	269	403	5394
-25.70	20	3770	282	421	5635
-26.55	0	3922	293	437	5844
-26.70	0	3950	295	439	5871

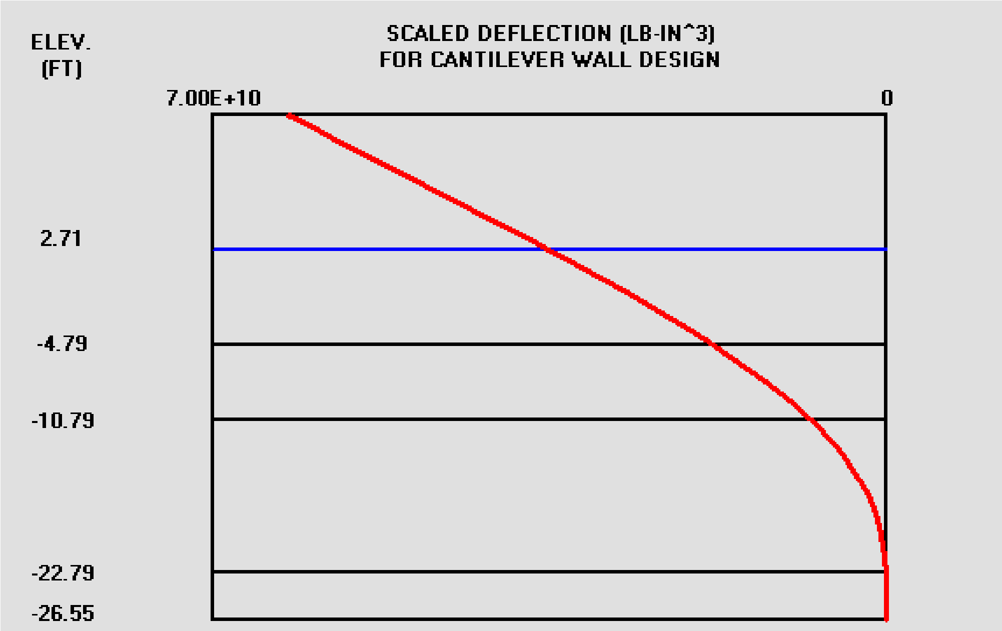


Figure 7: Sheet Pile Wall Deflection Plot

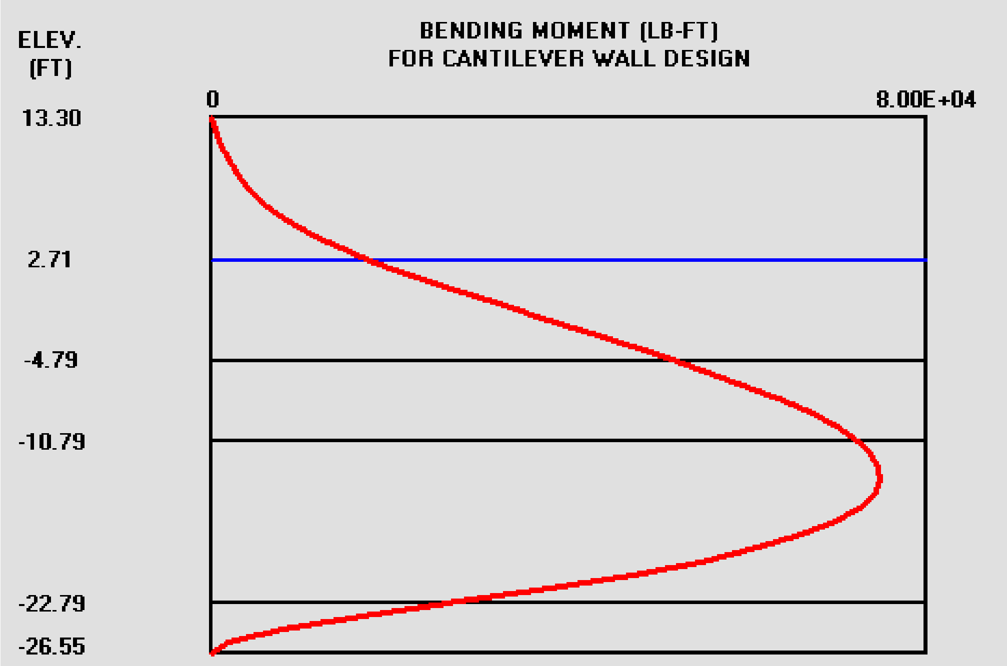


Figure 8: Sheet Pile Wall Bending Moment Diagram

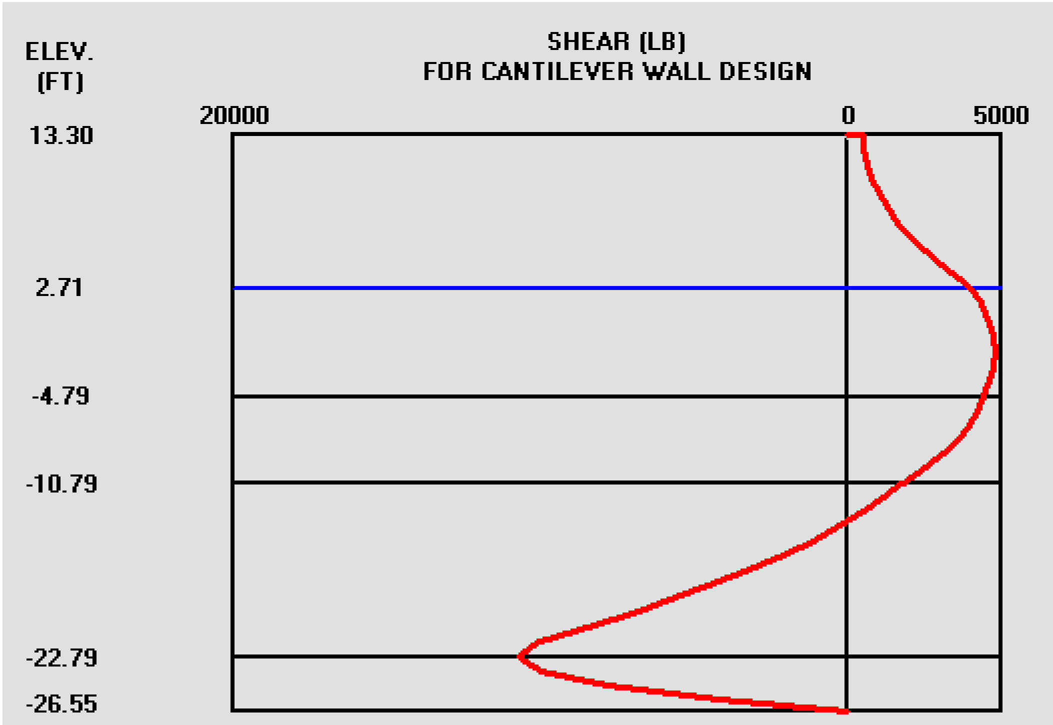


Figure 9: Sheet Pile Wall Shear Diagram

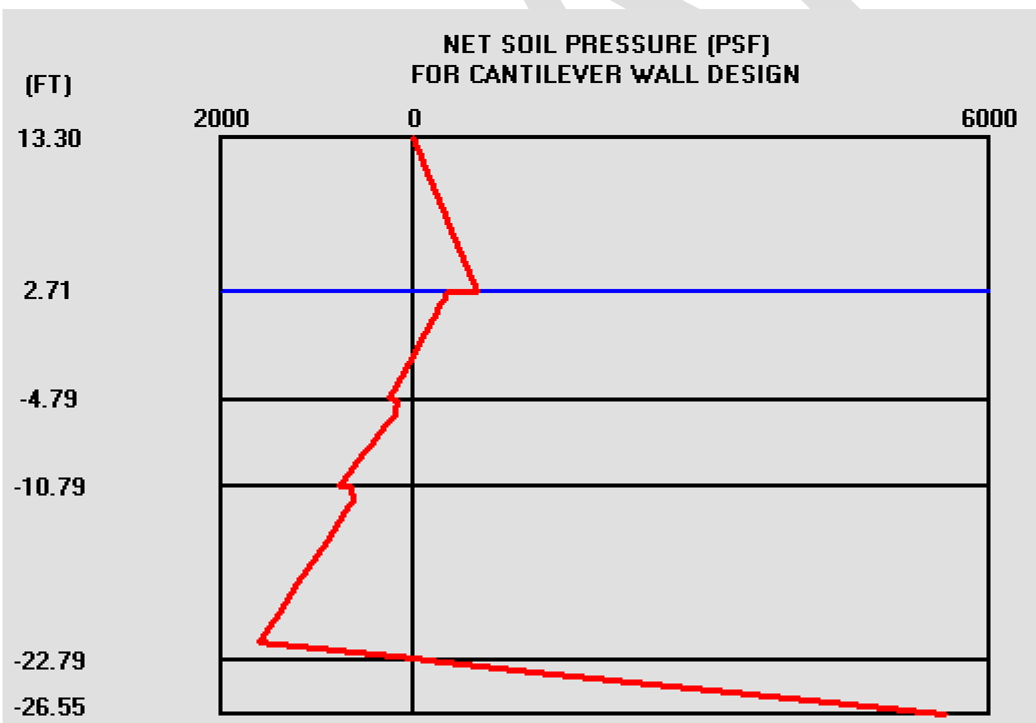


Figure 10: Net Soil Pressure Plot

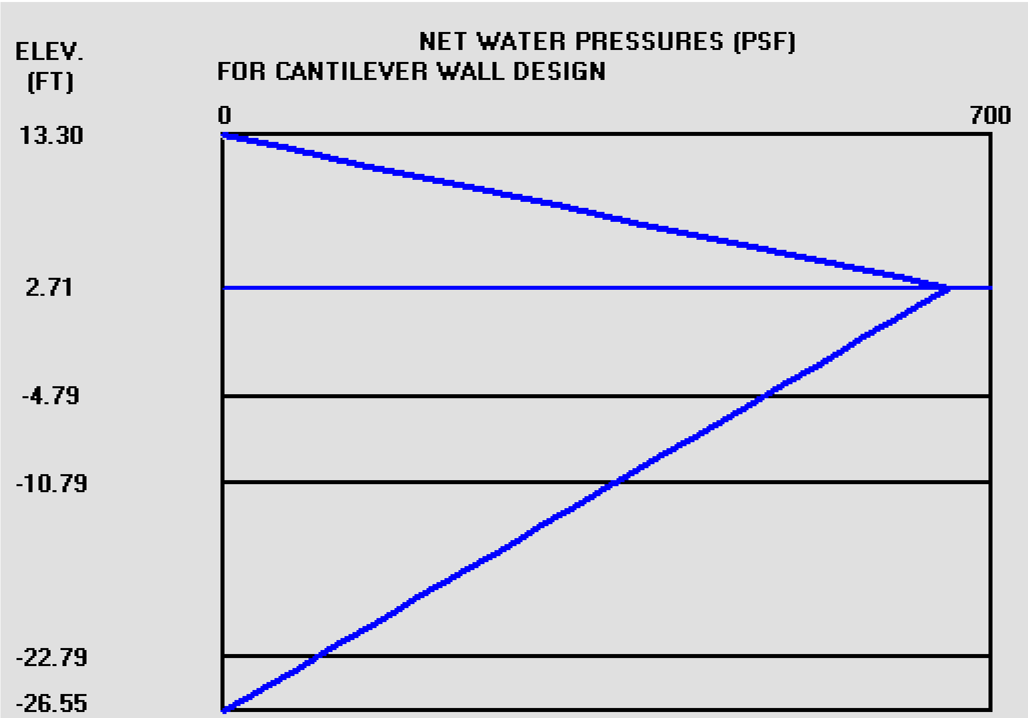


Figure 11: Net Water Pressure Plot

Steel Sheet Pile Design

Maximum Moment = 74.8 kip-ft/ft = 897.6 k-in/ft

Maximum Shear = 10.7 kip/ft

Mu = 1.4(74.8 kip-ft/ft) = 104.72 kip-ft/ft = 1256.6 kip-in/ft

Vu = 1.4 (10.7 kip/ft) = 15.0 kip/ft

$\varphi Mn \geq Mu$, $Mn = Fcr\ Smin$ (from AISC Equation F12.1)

Where:
Fcr – For driven hot rolled sheet pile, the members are restrained against lateral torsional buckling and the pile has sufficient thickness against local buckling; therefore, *Fcr = Fy*.

Smin = Sx

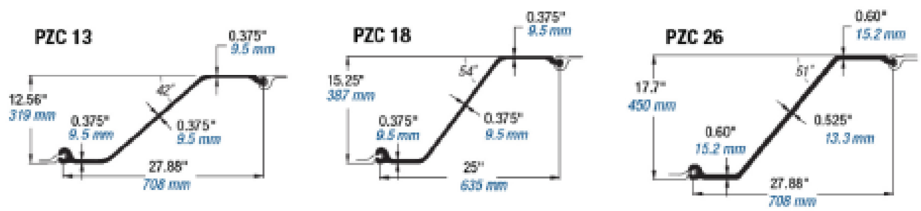
Therefore: *Mn = FySx* where *Fy* is the yield strength and *Sx* is the section modulus of the sheet pile.

$\varphi Fy\ Sx \geq Mu$

Where:
(0.9)(50 ksi)Sx ≥ 1256.6 kip-in/ft

Sx-required ≥ 27.9 in^3/ft

A hot rolled steel sheet pile section PZC17 has a section modulus of 31 in^3/ft, which exceeds the required 27.9 in^3/ft. The shear capacity of the chosen sheet pile section must also be checked.



Minimum Grade 60 Standard					Per Single Section						Per Unit of Wall			
GERDAU	Nominal Width	Wall Depth (Height)	Web Thickness	Flange Thickness	Area	Weight	Moment of Inertia	Section Modulus	Total Surface Area	Nominal Coating Area*	Area	Weight	Moment of Inertia	Section Modulus
Section	in. (mm)	in. (mm)	in. (mm)	in. (mm)	in.2 (cm2)	lbs/ft (kg/m)	in.4 (cm4)	in.3 (cm3)	ft2/ft (m2/m)	ft2/ft (m2/m)	in.2/ft (cm2/m)	lbs/ft2 (kg/m2)	in.4/ft (cm4/m)	in.3/ft (cm3/m)
PZC 12	27.88 706	12.52 318	0.335 8.5	0.335 8.5	13.64 88.0	46.6 69.1	324.5 13,510	51.8 850	6.10 1.86	5.60 1.71	5.87 124.3	20.0 97.6	139.7 19,060	22.3 1,200
PZC 13	27.88 708	12.56 319	0.375 9.5	0.375 9.5	14.82 95.6	50.4 75.1	353.0 14,690	56.2 920	6.10 1.86	5.60 1.71	6.38 135.1	21.7 106.0	152.0 20,760	24.2 1,300
PZC 14	27.88 708	12.60 320	0.420 10.7	0.420 10.7	16.15 104.2	55.0 81.8	381.6 15,890	60.5 990	6.10 1.86	5.60 1.71	6.95 147.2	23.7 115.5	164.3 22,440	26.0 1,400
PZC 17	25.0 635	15.21 386	0.335 8.5	0.335 8.5	13.64 88.0	46.4 69.1	491.8 20,470	64.6 1,060	6.10 1.86	5.60 1.71	6.55 138.6	22.3 108.8	236.1 32,235	31.0 1,670
PZC 18	25.00 635	15.25 387	0.375 9.5	0.375 9.5	14.82 95.6	50.4 75.1	532.2 22,150	69.8 1,145	6.10 1.86	5.60 1.71	7.12 150.6	24.2 118.2	255.5 34,890	33.5 1,800
PZC 19	25.00 635	15.30 388	0.420 10.7	0.420 10.7	16.16 104.2	55.0 81.8	576.3 23,990	75.3 1,235	6.10 1.86	5.60 1.71	7.75 164.1	26.4 128.8	276.6 37,760	36.1 1,945
PZC 25	27.88 708	17.66 449	0.485 12.3	0.560 14.2	20.40 131.6	69.4 103.3	938.7 39,070	106.3 1,740	6.65 2.03	6.15 1.87	8.78 185.9	29.9 145.9	404.1 55,190	45.7 2,455
PZC 26	27.88 708	17.70 450	0.525 13.3	0.600 15.2	21.72 140.1	73.9 110.0	994.3 41,390	112.4 1,840	6.65 2.03	6.15 1.87	9.35 197.9	31.8 155.4	428.1 58,460	48.4 2,600
PZC 28	27.88 708	17.75 451	0.570 14.5	0.645 16.4	23.22 149.8	79.0 117.6	1,057 44,000	119.1 1,950	6.65 2.03	6.15 1.87	10.00 211.6	34.0 166.1	455.1 62,150	51.3 2,755

All dimensions given are nominal. Actual flange and web thicknesses vary due to mill rolling practices; however, permitted variations for such dimensions are not addressed.

Figure 11: PZC Hot Rolled Sheet Pile Data Sheet. Source: http://www.jdfields.com

φ*Vn* ≥ *Vu*, where *Vn* = 0.6*(Fy)(Aw)* (from AISC Equation G2-1)

and *Aw* = *Av* = *(twh)/w* (from Equation 9.4)

Where: φ = 0.9 (from AISC section G1)

Therefore: *(φ)*0.6*(Fy)(Av)* ≥ *Vu*

(0.9)(0.6)(50 ksi)(0.335 in.)(15.21 in.)/(2.08 ft) = 66.14kip/ft

66.14kip/ft ≥ 15.0 kip/ft. Therefore, shear is OK.

Concrete Cap Design

EM 1110-2-2104 requires design according to ACI 318 but with modifications. The design load case is an unusual load case and therefore reinforced concrete design is performed with single load factor of 1.6. This is the principal load factor for maximum hydrostatic loading with a return period in the unusual category, accounting for serviceability requirements, from EM 1110-2-2104.

Design for Full Section. According to paragraph 9.8.5.5 of EM 1110-2-2502, the top of the connection (top of sheet pile) will be designed for both moment (*Ma*) and shear (*Va*). The sheet pile is extended 36 in. into the concrete cap according to paragraph 9.8.5.2. With the bottom of the concrete set at the frost depth of 6 in. below the ground surface, the top of the sheet pile is one foot above the ground surface elevation of 3.71ft. The forces at the top of the sheet pile from the CWALSHT analysis are:

Ma = 15.4 kip-ft/ft
Va = 3.62 kips/ft

Checking bending moment, $\varphi M_n \geq M_u$.
M_u = 1.6 (15.4 kip-ft/ft) = 24.64 kip-ft/ft

Cap Geometry. The concrete cap must provide a minimum of 6 in. (15 cm) of cover over the steel sheet pile but not less than 24 in. (61 cm) in width through the connection.
PZC 17 15.21 wall depth + 6in + 6in = 28in

Minimum cover from EM 1110-2-2104 is 3 in. for this application.

Minimum reinforcement for temperature and shrinkage is 0.0030 of the gross area from EM 1110-2-2104. The required area is 0.0030 (28 ft)(12 ft) = 1.0 in^2 with 0.5 in^2 each face. Try using #7 @ 12 in. with an area (*As*) of 0.6 in^2 per foot.

Calculation of M_n.
$M_n = A_s f_y (d – a/2)$

For this design, f_y = 60 ksi, $f'c$ = 4.0 ksi.
d = 28 in. – 3 in. (cover) – .5 = 24.5 in.

Design for a unit width, b, of 12:
a= $A_s f_y / 0.85\, f'_c\, b$ = 0.6in^2 (60ksi) / 0.85(4.0ksi)(12in) = 0.88 in

M_n = 0.6 in^2 (60ksi) (24.5 in – (0.88/2)) = 866.16 kip-in/ft = 72.2 kip-ft/ft
From ACI 318-19, φ = 0.9 for bending.

φM_n = 0.9(72.2 kip-ft/ft) = 65.0 kip-ft/ft which is greater than M_u = 24.64 kip-ft/ft

Check of reinforcing ratio (ρ) according to EM 1110-2-2104.

$P_{provided} = A_s / bd = 0.6\ in^2/ft / 12in\ (24.5in) = 0.002$

Check minimum reinforcing requirements. From EM 1110-2-2104 the minimum requirements are:

$$\rho > \frac{3.0\sqrt{f'_c}}{F_y} = \frac{3.0\sqrt{4{,}000\ psi}}{60{,}000\ psi} = 0.0032$$

$$\rho > \frac{200}{F_y} = \frac{200}{60{,}000\ psi} = 0.0033$$

Or that ρ provided is greater than 4/3 of ρ required.

Check that ρ is less than $0.25\rho_b$ as required by EM 1110-2-2104.
$\rho_b = 0.85f'_c / f_y * \beta_1\ [87{,}000/(87{,}000 + 60{,}000psi)] = 0.0285$
$0.25\ \rho_b = 0.25(0.0285) = 0.0071 > 0.002$ OK

3.2 Flood Gate

A vertical lift gate structure was proposed in the Bucksport focus area along the Old Pee Dee Road Cowford Swamp Bridge. The exact geometry of the structure is unknown at this stage of the study, including the span of the gate structure. Structural loads have not yet been calculated for the gate structure, so the foundation required to support this structure is conceptual. No site-specific geotechnical data for this structure was obtained. Based on structural drawings of the adjacent Cowford Swamp bridge, it is assumed that a prestressed concrete pile deep foundation will be required for the gate structure. This measure was not carried forward to TSP.

3.3 Relief Bridges (Culverts)

Culverts under existing bridges in the Conway focus area were proposed to help connect the floodplain and improve conveyance by reducing bottle necking. The bridge locations are at the 905 Bridge, the 501 Business Bridge, and the 501 bridge. Approximate locations and the terrain elevations are shown in the figures below. The culverts are estimated to be 48in diameter concrete pipes. Each bridge location will have 4 adjacent culverts to improve conveyance. No site-specific geotechnical data for these structures were obtained. As the study progresses, geotechnical sampling and testing will be obtained with available funds. SCDOT subsurface investigation guidelines will be followed for culverts/pipes that cross an alignment in a transverse direction, a current Average Daily Traffic greater than 5,000 vehicles per day, having a diameter greater than or equal to 48in, and will be founded at or below the original grade. Number of samples from these guidelines will be reduced to an appropriate amount for a feasibility study. The subsurface investigation will attempt to characterize possible unsuitable soil conditions for the culvert foundation. If unsuitable soil is encountered soil remediation or deep pile foundation may be required for stability and to mitigate settlement. Internal erosion features for the culverts will also be considered.

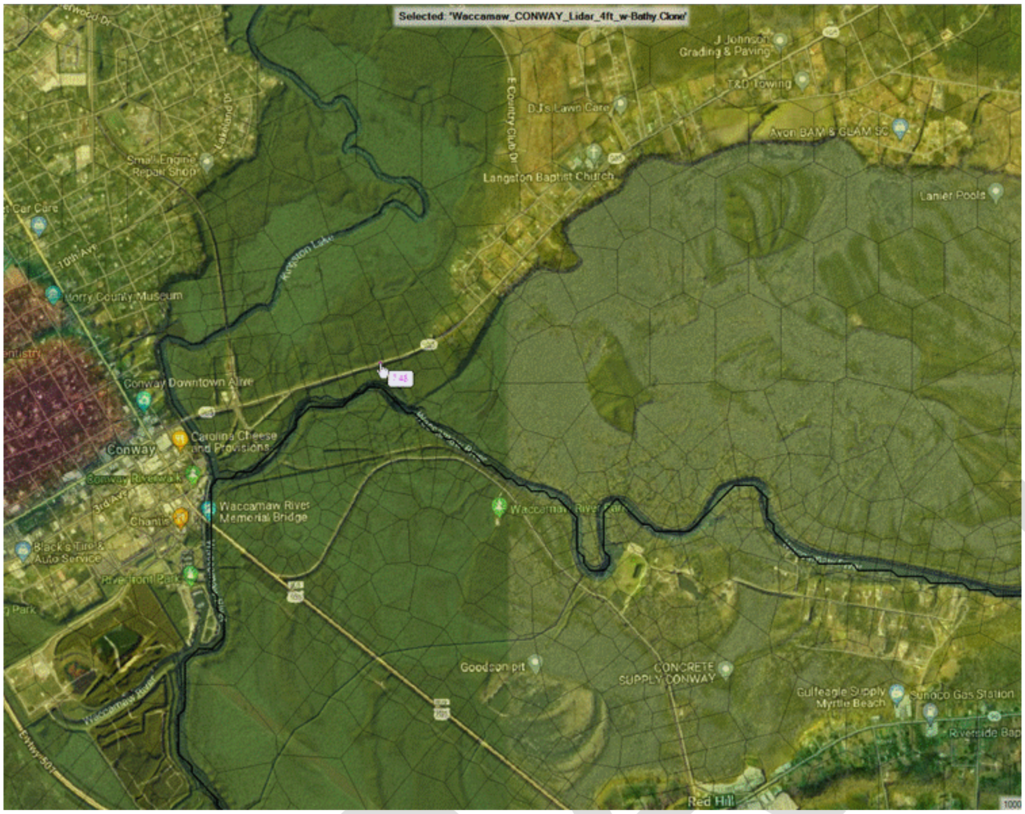


Figure 12: Highway 905 Bridge Location

Figure 13: Highway 501 Business Bridge Location

Figure 14: Highway 501 Bridge Location

3.4 Weir Removal

Removal of the existing weirs on Socastee Creek in the Socastee focus area was proposed to improve conveyance. The demolition of these weirs will require sediment control BMPs to mitigate sediment transport downstream. The side slopes where the existing weirs are located will need to be permanently stabilized to mitigate erosion post-demolition. No site-specific geotechnical data for these structures were obtained and further geotechnical considerations for these measures will be developed as the study progresses.

3.5 Excavation

Structural measures that would require excavation include benching and the installation of floodplain connection culverts. Based on limited information of the regional geology, difficult excavation due to rock is unlikely. However, debris and other unsuitable material may be encountered during the excavation operations. Due to the nature of the location of these measures, it is assumed that the soils will be mostly saturated. Temporary unwatering measures, by sump pumps, drainage ditches, or other methods as determined by the contractor, may be needed to control surface water during excavation operations. Site specific information has not been obtained for these measures and further geotechnical considerations for excavation for weir removal and floodplain connection culverts will be developed as the study progresses.

4.0 Nonstructural Measures

4.1 Elevation

Elevation of residential structures has been proposed in all focus areas. Structure elevation would likely include a deep pile foundation. Based on limited information of the regional geology, difficult excavation due to rock is unlikely. However, debris and other unsuitable material may be encountered during the excavation operations.

Excavation trenches near the existing structures should be graded such that rainwater does not saturate the soils beneath the existing foundation. Temporary unwatering measures, by sump pumps, drainage ditches, or other methods as determined by the contractor, may be needed to control surface water during excavation operations.

5.0 Report Limitations

The geological information provided in this report is based on general data obtained for the SC coastal plain area, SCDNR Geologic Quadrangle Maps, and limited geotechnical reports from adjacent construction in the area. This report does not account for human placed materials, existing organic materials, and/or surficial deposits that may overlay the geological formation. Site specific groundwater information is not available at the time of this report. Collection of groundwater data, such as the installation of piezometers and monitoring wells, will not be included in this study. Groundwater can vary based on site topography, seasons, rainfall, and other factors. Impermeable to semi-impermeable surfaces, such as concrete, rock, clay, debris, etc., can cause perched groundwater conditions. Site specific investigations can help the engineers and contractors have a better understanding of the subsurface conditions at the proposed work sites.

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References

U.S. Army Corps of Engineers, Engineering Manual (EM) 1110-2-2502 (Floodwalls and Other Hydraulic Retaining Walls). Issued 2022

UFC 3-220-10. Soil Mechanics. Unified Facilities Criteria (UFC), 2022.

Appendix A4. Geotechnical Engineering Attachments

Report of Geotechnical Exploration
New Forestbrook Fire Station at Burcale Road
Myrtle Beach, South Carolina
S&ME Project No. 212687

PREPARED FOR:

Horry County Maintenance Department
307 Smith Street
Conway, South Carolina 29526

PREPARED BY:

S&ME, Inc.
1330 Highway 501 Business
Conway, SC 29526

August 30, 2021

August 30, 2021

Horry County Maintenance Department
307 Smith Street
Conway, South Carolina 29526

Attention: John Barnhill; Director

Reference: **Report of Geotechnical Exploration**
New Forestbrook Fire Station at Burcale Road
Myrtle Beach, South Carolina
S&ME Project No. 212687

Dear Mr. Barnhill:

S&ME, Inc. has completed the subsurface exploration for the referenced project. Our exploration was conducted in general accordance with our Proposal No. 212687, dated March 17, 2021, authorized by you through the issuance of Horry County Purchase Order No. 21002085 dated April 19, 2021.

The purpose of this study was to characterize the surface and subsurface soils on the proposed site, and to provide recommendations for site preparation and earthwork, foundation types and seismic design values, on-site soil suitability, pavement subgrade preparation, and pavement section thickness recommendations. This report presents the findings of our exploration along with our conclusions and recommendations.

S&ME, Inc. appreciates this opportunity to be of service to you. Please contact us if you have questions concerning this report or any of our services.

Sincerely,

S&ME, Inc.

Jeremy Lighthall, E.I.T.
Staff Professional

SOUTH CAROLINA CERTIFICATE OF AUTHORIZATION
S&ME, INC.
No. C00473

Ronald P. Forest, Jr., P.E.
Senior Engineer
Registration No. 21248

SOUTH CAROLINA REGISTERED PROFESSIONAL ENGINEER
No. 21248
RON FOREST JR.
8/30/21

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Appendices

♦ Executive Summary

For your convenience, this report is summarized in outline form below. This brief summary should not be used for design or construction purposes without reviewing the more detailed information presented in the remainder of this report.

1. **Soil Conditions:** Topsoil was observed to range from approximately 2 inches to 8 inches, and averaged approximately 6 inches in thickness across the site. Our hand auger borings and test soundings encountered a layer of soft to firm clays (Stratum I) to a depth of about 7 to 7 ½ feet. A few organics were encountered within this stratum in the hand auger borings to a depth of 4 feet. Underlying Stratum I, an intermediate layer of medium dense to dense sands (Stratum II) was encountered to depths ranging from 13 feet to 13 ½ feet below the surface. Below these sands, a zone of interbedded very soft to firm silts and clays, and loose to dense sands (Stratum III) was encountered to a depth of approximately 23 ½ feet to 25 feet below the existing ground surface. Beneath Stratum III, a zone of medium dense to very dense sand (Stratum IV) was encountered to the maximum exploration depth of 26.8 feet.

    - **Warning:** The contractor should anticipate potentially soft, clayey surface conditions once the topsoil is removed from this site. Stripping and grubbing should not be performed while the site is excessively wet, or else this may cause the upper clay surface to deteriorate significantly. Install drainage measures as soon as possible, preferably prior to stripping and grubbing operations.

2. **Subsurface Water:** Water was not encountered in the hand auger borings at the time of drilling to a depth of 4 feet below the surface. Water levels within the cone soundings were interpreted from pore pressure readings to range from approximately 3 to 4 feet below the existing ground surface. This site is favorable for the development of shallow perched groundwater conditions due to the clayey upper soils.
3. **Seismic Site Class and Liquefaction:** Test data indicates that this site is best described as IBC 2018 (Code) seismic ***Site Class E*** due to the generally soft clayey soil profile. Liquefaction of the soil profile during seismic shaking was determined not to be a significant concern at this site, considering the anticipated ground accelerations associated with the design magnitude earthquake, so site class F does not apply.
4. **Seismic Design Parameters:** Based on the average shear wave velocities that we measured, and the extrapolated value of about 590 fps to a depth of 100 feet, Seismic Site Class E parameters appear to be appropriate for design of the new fire station. The following seismic design parameters apply to the 2018 Code: S_{DS} = 0.46g, S_{D1} = 0.32g, and Peak Ground Acceleration (PGA_M) = 0.34g. For a structure having a Risk Category of IV, the S_{DS} and S_{D1} values obtained are consistent with ***Seismic Design Category D*** as defined in section 1613.5 of the IBC, 2018 edition.
5. **Shallow Foundations:** Shallow foundations may be used to support the building assuming that the structure can be designed to tolerate the predicted static settlements associated with the building loads. Considering the assumed structural loads, we recommend an allowable bearing capacity of 2,000 pounds per square foot (psf) for design of isolated shallow spread footings. The estimated total static settlement under the assumed loads is approximately 1 inch or less. The estimated column-to-column differential static settlement under the assumed loads is approximately 1/2 inch or less.

6. **Grade Slabs:** Grade slabs may be soil-supported if the site is prepared as recommended herein, and a modulus of subgrade reaction (k) of 150 pci may be used for slab reinforcing design. Within finished spaces, we recommend at least a 4-inch-thick layer of granular material be placed immediately beneath the slabs to act as a capillary break. Granular materials used may consist of a crushed, well-graded gravel blend such as SCDOT Graded Aggregate Base Course (GABC), or an open-graded, manufactured washed gravel such as SCDOT No. 57 or No. 67 stone.
7. **Pavements:** Flexible (asphalt) pavements are not recommended for use in areas that will be traveled by fire trucks. Only rigid Portland cement concrete pavements should be used in those areas. Flexible asphalt pavements may be used in employee parking lot (car traffic) areas only.

   - For heavy-duty rigid (concrete) pavement in fire/rescue vehicle travel areas, we recommend 4,000 psi compressive strength Portland cement concrete with a thickness of 8 inches that is continuously steel-reinforced, overlying a compacted graded aggregate base course (GABC) thickness of 6 inches, overlying a drainage layer consisting of 6 inches of open-graded, manufactured, granitic gravel meeting the gradation of SCDOT No. 57 or No. 67 stone. Non-woven geotextile filter fabric (TenCate Mirafi 140N) is recommended to be placed between the GABC layer and the drainage layer to prevent migration of fines, and woven geotextile (TenCate Mirafi HP-370) is recommended to be placed between the drainage layer and the soil subgrade to provide both separation and strength. Perimeter underdrains are also recommended. See Figure 5 in Appendix I for a typical pavement section detail.

   - We have been involved in several fire station pavement repair projects with Horry County Maintenance Department over the years, and most of the pavement deterioration has been attributed to poor subsurface drainage. The gravel drainage layer plus underdrain approach has been implemented in these pavement repair projects with success. While adding some initial cost, the long term savings of using this approach are expected to be quite significant over the service life span of the facility.

   - For light-duty flexible (asphalt) pavement areas in employee car parking areas only, we recommend 2 inches of hot mix asphalt (HMA) surface course type C over 6 inches of compacted SCDOT graded aggregate base course over at least 18 inches of sandy imported select fill separating the native clay subgrade from the bottom of the base course layer. We anticipate that the clayey subgrade that is exposed at cut grade elevation may not be stable enough to support the fill material without additional stabilization support; therefore, we recommend the inclusion of a layer of TenCate Mirafi HP-370 on top of the clay subgrade in the employee parking lot. Perimeter underdrains are also recommended for the employee parking lot.

1.0 Introduction

The purpose of this exploration was to obtain subsurface information to allow us to characterize the subsurface conditions at the site and to develop recommendations concerning earthwork, foundations, pavements, and other related construction issues. This report describes our understanding of the project, presents the results of the field exploration and laboratory testing, and discusses our conclusions and recommendations.

A site plan showing the approximate exploration location is included in Appendix I. The sounding logs, hand auger logs, discussion of the field exploration procedures, and legends of soil classification and symbols are included in Appendix II. Appendix III contains the results of the laboratory testing and our laboratory test procedures.

1.1 Site and Project Description

Project information was originally provided in an email from Mr. John Barnhill (Horry County Maintenance Dept.) to Ron Forest, Jr. (S&ME, Inc.) on March 10, 2021. The email contained an aerial map of a Pine Island Tract, located just southwest of the intersection of Burcale Road and Fantasy Harbour Boulevard in Myrtle Beach, South Carolina. On August 2, 2021, another email was sent from John Barnhill to Ron Forest Jr. that contained a site layout plan dated July 30, 2021. The existing site consists of a wooded area neighboring a powerline easement. We understand that the new fire station will be constructed in the currently wooded area. A site vicinity map is attached in Appendix I as Figure 1.

1.2 Project Description

The proposed new fire station will consist of a four bay, drive through truck building, associated concrete driveways and truck aprons, office/living area, employee parking lot, and a detention pond. We anticipate that the structure will be supported on shallow foundations and may include cold-formed metal framing and/or structural masonry walls and a soil-supported slab on grade. We were not provided structural load information. We assume based on our previous experience with similar projects that column and wall loads will not exceed 75 kips and 5 kips per foot, respectively, and that a maximum uniform area load of 250 pounds per square foot, including the slab. We also anticipate that up to 2 to 3 feet of fill may be needed on this site to achieve the design grades.

2.0 Exploration Procedures

2.1 Field Exploration

On August 5, 2021 and August 13, 2021, representatives of S&ME, Inc. visited the site. Using the information provided, we performed the following tasks:

- We performed a site walkover, observing features of topography, existing structures, ground cover, and surface soils at the project site.
- We established one seismic cone penetration test (SCPT) sounding location and two cone penetration test (CPT) sounding locations, labeled C-1 through C-3.

- One SCPT sounding (C-2), was advanced to a depth where no further advancement could be made under the maximum force of the rig, defined as "refusal". We advanced this sounding within the approximate center of the future building footprint to a depth of 26.8 feet.
- Two CPT soundings (C-1 and C-3) were advanced within the approximate future building footprint to target depths of 25 feet and 26 feet, respectively.
- Within the SCPT sounding, downhole shear wave velocity measurements were obtained at approximate 1 meter depth intervals until the sounding was terminated. In the SCPT/CPT soundings, an electronically instrumented cone penetrometer was hydraulically pushed through the soil to measure tip point stress, pore water pressure, and sleeve friction. The data was then used to determine soil stratigraphy and to estimate soil strength parameters.
- We advanced a hand auger boring at each of the SCPT/CPT sounding locations to observe the near surface soils (C-1 through C-3). These hand auger borings were advanced to a depth of 4 feet each.
- We advanced five additional hand auger borings (P-1 through P-5) with dynamic cone penetrometer (DCP) testing within the proposed parking and driving areas. These hand auger borings were each advanced to a depth of 4 feet below the surface. In conjunction with these hand auger borings, DCP testing was performed at approximate one-foot intervals in each boring in general accordance with ASTM STP 399, "Dynamic Cone for Shallow In-Situ Penetration Testing" to provide us with an index for estimating soil strength parameters and relative consistency of the near-surface soils encountered.
- The subsurface water levels at test locations were measured in the field at the time of our field work or were interpreted from CPT pore pressure readings.
- A test location sketch showing the approximate locations of the soundings and hand auger borings is attached in Appendix I as Figure 2.

A brief description of the field exploration procedures performed, as well as the sounding and hand auger boring logs, is attached in Appendix II.

2.2 Laboratory Testing

After the recovered soil samples were brought to our laboratory, a geotechnical professional examined and/or tested each sample to estimate its distribution of grain sizes, plasticity, moisture condition, color, presence of lenses and seams, and apparent geologic origin in general accordance with ASTM D 2488, "Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)".

The resulting classifications are presented on the hand auger boring logs, included in Appendix II. Similar soils were grouped into representative strata on the logs. The strata contact lines represent approximate boundaries between soil types. The actual transitions between soil types in the field are likely more gradual in both the vertical and horizontal directions than those which are indicated on the logs.

We performed the following quantitative ASTM-standardized laboratory tests to help classify the soils and formulate our conclusions and recommendations. The laboratory tests performed included the following:

- One bulk sample tested in general accordance with ASTM D 2216, "Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass", to measure the in situ moisture content of the soil.

- One bulk sample tested in general accordance with ASTM D 6913, "Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis", to measure the distribution of particle sizes greater than 75 µm.
- One bulk sample tested in general accordance with ASTM D 1140, "Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75-µm) Sieve", to measure the percent clay and silt fraction.
- One bulk sample tested in general accordance with ASTM D 4318, "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils", to measure the plasticity of the soil.
- One bulk sample tested in general accordance with ASTM D 698, "Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 lbf/ft^3)", to measure the moisture-density relationship of the soil.
- One bulk sample recompacted and tested in general accordance with ASTM D 1883, "Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils", to evaluate soil support characteristics for pavements.

The laboratory test results and procedures for the above listed tests are attached to this report in Appendix III.

3.0 Site and Surface Conditions

This section of the report describes the general site and surface conditions observed at the time of our exploration.

3.1 Existing Ground Cover

The site is currently wooded with an adjacent cleared powerline easement. The existing trees ranged from a few feet in height to over 50 feet in height. The site is densely vegetated with small trees, large trees, and shrubs. There is a ditch running parallel to Burcale Road, which limits access to the site to the existing powerline easement.

3.2 Topography

The site appears to be relatively level, with less than a few feet of elevation change across the site excluding the ditches; however, a topographic site plan was not provided to us and it was outside the scope of our work to survey the site. As a result, the existing ground surface elevation was set to zero for the purposes of this exploration and this is reflected on the sounding logs and the interpreted cross-sectional subsurface soil profile.

3.3 Topsoil

Topsoil was encountered in each of our hand auger borings. Within the hand auger borings, topsoil was measured to range in thickness from 2 inches to 8 inches, averaging approximately 6 inches across the site. Topsoil thickness may be greater in unexplored areas. Root mass may extend significantly deeper.

3.4 Local Geology

The site is located in the Coastal Plain Physiographic Region of South Carolina. This area is dominated topographically by a series of relic beach terraces, which progressively increase in surface altitude as they proceed inland. These terraces have been extensively mapped and correlated over wide areas. Surface soils penetrated by our borings and soundings have been interpreted to be a part of the Socastee Formation, consisting of relatively recent marine deposits laid down approximately 200,000 years ago.

4.0 Subsurface Conditions

The generalized subsurface conditions at the site are described below. For more detailed descriptions and stratifications at test locations, the respective sounding and hand auger boring logs should be reviewed in Appendix II.

4.1 Interpreted Subsurface Profile

An interpreted subsurface cross-sectional profile of the site soils is attached as Figure 3 in Appendix I to illustrate a general representation of the subsurface conditions within the proposed construction area. The cross-section orientation in plan view is shown on Figure 2. Profile A-A' (Figure 3) depicts the subsurface conditions across the site, looking in a westerly direction.

The strata indicated in the profile are characterized in the following section. Note that the profile is not to scale and was prepared for illustrative purposes only. Subsurface stratifications may be more gradual than indicated, and conditions may vary between test locations. Soil classifications are based on the soil behavior type (SBT)[1] as tabulated in the CPT data within each sounding.

Soils presented on the profile were grouped into several general strata based on estimated physical properties derived from the borings and the recovered samples. The strata encountered are labeled I through IV on the soil profile to allow their properties to be systematically described.

4.2 Description of Subsurface Soils

This section describes subsurface soil conditions observed at the site as illustrated on the profile.

4.2.1 Stratum I: Upper Soft to Firm Fat Clays

Underlying the topsoil, a stratum of soft to firm fat clay (USCS Classification "CH") was encountered within each of our hand auger borings to their termination depths of 4 feet below the existing ground surface. The CPT soundings encountered similar soils to depths ranging from 7 feet to 7 ½ feet below the surface. Within the CPT

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[1] Soil Behavior Type (SBT) is calculated based on empirical correlations with tip resistance, sleeve friction, and pore pressure. A CPT may define a soil based on its behavior as one type while its grain size and plasticity, the traditional basis for soil classification, may define it as a different type.

soundings, this stratum exhibited SBTs of clay, very stiff fine-grained soils, and silt mixtures. The soils of Stratum I typically exhibited tip resistances ranging from about 8 tons per square foot (tsf) to about 15 tsf, indicating typically a typically soft to firm consistency. A layer of stiff clayey soils with tip resistances ranging from about 20 tsf to 30 tsf was encountered in soundings C-2 and C-3 at depths of 6 to 7 ½ feet; however, this thin stiff zone was not observed in sounding C-1. DCP values within this stratum ranged from 4 blows per increment (bpi) to 7 bpi. Generally, DCP values ranged from 5 bpi to 7 bpi, indicating a firm consistency with occasional soft zones.

A composite bulk sample was collected from the upper portion of Stratum I and subjected to natural moisture content, grain size distribution, plasticity, Proctor, and California Bearing Ratio (CBR) testing. The soil was collected from the proposed pavement area at approximately 1 to 4 feet below grade in hand auger borings P-1 through P-5. The sample was classified as fat clay (CH) with a fines content of 90.7 percent by weight passing the No. 200 sieve, a liquid limit of 56 percent, a plastic limit of 22 percent, and a plasticity index of 31 percent. The natural moisture content was measured to be 26.4 percent. The standard Proctor maximum dry density was 116.0 pounds per cubic foot (pcf) at an optimum moisture content of 12.6 percent, indicating that the soil in place is about 13.8 percent wet of its optimum moisture content. The CBR value of a remolded sample of this soil was measured to be 1.2 percent at 94.9 percent compaction (ASTM D 698), indicating poor capacity for direct pavement section support.

4.2.2 Stratum II: Intermediate Medium Dense to Dense Sands

Underlying Stratum I, beginning at a depth of about 7 to 7 ½ feet below the surface, an intermediate stratum of sands (Stratum II) was encountered which continued to a depth of about 12 ½ feet to 13 feet within the three soundings. The soils of this stratum exhibited tip resistances ranging from about 20 tsf to about 250 tsf, indicating a loose to very dense relative density, but typically ranged from about 40 tsf to 200 tsf, indicating typically medium dense to dense conditions within most of the stratum. Typically, the shallower sands of this stratum exhibited higher tip resistances, and therefore were considered to have a denser relative density. Very dense seams of sand were observed in soundings C-1 and C-2 between depths of about 8 to 10 feet.

4.2.3 Stratum III: Interbedded Silts, Clays, and Sands

Beneath the sands of Stratum II, a layer of interbedded silts, clays, and sands (Stratum III) was encountered to depths ranging from 23 ½ feet to 25 feet below the surface. Within sounding C-1, a layer of sensitive fine-grained soils was encountered between depths of approximately 13 to 14 feet below ground surface. The tip resistances within the sands of this stratum were measured to typically range from 20 tsf to 70 tsf, with a seam of sand in sounding C-3 exhibiting a tip resistance of up to 160 tsf. This indicates typically loose to medium dense relative density, with a seam of dense sand at C-3 between depths of approximately 19 ½ feet to 21 ½ feet. The clays and silt mixtures of this stratum exhibited tip resistances ranging from 3 tsf to 15 tsf, indicating a very soft to firm consistency.

4.2.4 Stratum IV: Lower Medium Dense to Very Dense Sands

Below the interbedded silts, clays, and sands of Stratum III, Stratum IV consisted of sandy soils which extended to the maximum exploration depth of each of the test soundings, at a maximum depth of 26.8 feet below the surface at test location C-1. Tip resistances within the sands typically ranged from 45 tsf to over 500 tsf at refusal. These tip resistances indicate a medium dense to very dense relative density.

4.2.5 Subsurface Water

Subsurface water was not significantly encountered within our hand auger borings at the time of drilling, although some of the observed soils were wet. Water levels within the cone soundings were interpreted from pore pressure readings to be approximately 3 feet to 4 feet below ground surface (bgs) at soundings C-1 through C-3. The near-surface soils at this site are prone to the potential for development of shallow perched groundwater conditions due to their clayey consistency. Subsurface water levels may also fluctuate seasonally at the site, being influenced by rainfall variations and other factors, such as construction practices.

5.0 Seismic Site Class and Design Parameters

Seismic-induced ground shaking at the foundation is the effect taken into account by seismic-resistant design provisions of the International Building Code (IBC). Other effects, including landslides and soil liquefaction, must also be considered.

5.1 Building Code Seismic Provisions

As of January 1, 2020, the 2018 edition of the International Building Code (IBC) has been adopted for use in South Carolina. We classified the site as one of the Site Classes listed in the IBC Section 1613.3, using the procedures described in Chapter 20 of ASCE 7-16.

5.1.1 Evaluation of the Potential for Site Class F Conditions

The initial step in site class definition is to check for the four conditions described for Site Class F, which would require a site specific evaluation to determine site coefficients F_A and F_V. Soils vulnerable to potential failure include the following: 1) quick and highly sensitive clays or collapsible weakly cemented soils, 2) peats and highly organic clays, 3) very high plasticity clays, and 4) very thick soft/medium stiff clays. These soils were not evident in the borings or soundings.

One other determining characteristic, liquefaction potential under seismic conditions, was assessed. Soils were assessed qualitatively for liquefaction susceptibility based on their age, stratum, mode of deposition, degree of cementation, and size composition. This assessment considered observed liquefaction behavior in various soils in areas of previous seismic activity.

Liquefaction of saturated, loose, cohesionless soils occurs when they are subjected to earthquake loading that causes the pore pressures to increase and the effective overburden stresses to decrease, to the point where large soil deformation or even transformation from a solid to a liquid state results. Earthquake- induced ground surface acceleration at the site was assumed from the building code design peak ground acceleration of 0.34g according to the 2018 IBC.

Our analysis, which is more fully described in Section 5.1.2 below, indicates that some thin, potentially liquefiable layers were identified during our analysis; however, these soils underlie dense sands and thick clay layers, which will mitigate surface settlement. Significant and widespread liquefaction of the subsoils appears unlikely to occur at this site in the event of the design magnitude earthquake specified by the 2018 code (ASCE 7-16); therefore, Site Class F conditions do not reasonably apply to this site.

5.1.2 *Liquefaction Potential Index (LPI)*

To evaluate liquefaction potential, we performed analyses using the data obtained in the borings, considering the characteristics of the soil and water levels observed in the boring. The liquefaction analysis was performed based on the design earthquake prescribed by the 2018 edition of the International Building Code, the "simplified procedure" as presented in Youd et al. (2001), and recent research concerning the liquefaction resistance of aged sands (Hayati & Andrus, 2008; Andrus et al. 2009; Hayati & Andrus, 2009).

To help evaluate the consequences of liquefaction, we have computed the Liquefaction Potential Index (LPI), which is an empirical tool used to evaluate the potential for liquefaction to cause damage. The LPI considers the factor of safety against liquefaction, the depth to the liquefiable soils, and the thickness of the liquefiable soils to compute an index that ranges from 0 to 100. An LPI of 0 means there is no risk of liquefaction; an LPI of 100 means the entire profile is expected to liquefy. The level of risk is generally defined below.

- ***LPI < 5*** – surface manifestation and liquefaction-induced damage not expected.
- ***5 ≤ LPI ≤ 15*** – moderate liquefaction with some surface manifestation possible.
- ***LPI > 15*** – severe liquefaction and foundation damage is likely.

The LPI for this site under the 2018 Code is less than 5, which indicates that the risk of surface damage due to liquefaction is relatively low across the site. Therefore, we consider that Site Class F conditions do not apply, and based upon the shear wave velocity tests that we performed, the soil conditions within this site are determined to be Site Class E.

5.1.3 *Shear Wave Velocity Test Results*

Based upon the measured and extrapolated shear wave velocity, this site is determined to be **Site Class E**. This recommendation is provided based on the shear wave velocity measured at test sounding C-2 to a depth of 26.8 feet, where maximum reaction force occurred, and then extrapolated to a depth of 100 feet. The average weighted shear wave velocity was measured to be 556 feet per second (fps) in the upper 26.8 feet. When extrapolated to a depth of 100 feet, an average shear wave velocity of 590 fps is estimated, which is less than the 600 fps that is required for consideration of Site Class D parameters. See Figure 4 in Appendix I for the shear velocity graph.

Note: Because the extrapolated average shear wave velocity of 590 fps is close to the minimum value of 600 fps that is needed for Site Class D, it may be possible to improve the seismic site class from E to D by using more rigorous alternate test methods, such as measuring the shear wave velocity to greater depths using Multi-channel Analysis of Surface Waves (MASW) and Micro-tremor Array Methods (MAM). If desired, this additional testing can be performed for an additional fee; please contact us for more information.

5.2 Seismic Design Coefficients for Site Class E

Selection of the base shear values for structural design for earthquake loading is the responsibility of the structural engineer. However, for the purpose of evaluating seismic hazards at this site, S&ME has evaluated the spectral response parameters for the site using the general procedures outlined under the 2018 International Building Code.

Table 5-1: Seismic Design Coefficients

Criteria	Seismic Site Class	S_S	S_1	S_{DS}	S_{D1}	PGA_M	Seismic Design Category (Risk Cat. IV)
2018 IBC	E	0.311	0.114	0.462	0.319	0.343	D

5.2.1 *Seismic Design Category*

For a structure having a Risk Category classification of IV under the 2018 Code, the S_{DS} and S_{D1} values obtained are consistent with "Seismic Design Category D" as defined in section 1613.3.5 of the IBC.

Note: As mentioned in Section 5.1 above, by using more rigorous alternate test methods such as measuring the shear wave velocity to greater depths using Multi-channel Analysis of Surface Waves (MASW) and Micro-tremor Array Methods (MAM), it may be possible to improve the seismic site class from E to D. However, due to this being a Risk Category IV structure, even if the site class is improved from E to D, it would not change the seismic design category, which would remain "D".

6.0 Conclusions and Recommendations

The conclusions and recommendations included in this section are based on the project information outlined previously and the data obtained during our exploration. If the construction scope is altered, the proposed building location is changed, or if conditions are encountered during construction that differ from those encountered by the borings or soundings, then S&ME, Inc. should be retained to review the following recommendations based upon the new information and make any necessary changes.

6.1 Site Preparation

The following recommendations are provided regarding site preparation and earthwork:

1. While subsurface water was not observed within the hand auger borings in the upper 4 feet at the time of our exploration, we observed standing water to be ponded in equipment tracks on-site, and excess pore water pressures were measured near the surface in several of the CPT soundings. Therefore, this site is susceptible to perched water and it is prudent to implement and maintain temporary drainage measures during construction to drain the site and to divert water away from the construction area. Surface and

subsurface water conditions that occur during construction will determine the need for and extent of these temporary drainage measures. (Note: some permanent groundwater control measures such as underdrains in both the light-duty and heavy-duty pavement areas are recommended later in this report.)

2. Strip surface vegetation, topsoil, and rootmat, and dispose of outside the building and pavement footprints. Soils containing more than about 5 percent organics should be removed from the proposed construction areas. Although the organic topsoil thickness that we measured only averaged about 6 inches, we recommend an allowance of at least 12 inches for stripping, due to the soft consistency of the underlying soils which may become intermingled with the topsoil during the stripping operations.

    - **Warning:** The contractor should anticipate potentially soft, clayey surface conditions once the topsoil is removed from this site. Stripping and grubbing should not be performed while the site is excessively wet, or else this may cause the upper clay surface to deteriorate significantly. Install drainage measures as soon as possible, preferably prior to stripping and grubbing operations.

3. Fat clays (CH) were encountered in the upper soil profile at the site. These soils may pump, rut and become unstable under construction equipment when they are wet, and may be difficult to dry out once they become wet. Be prepared that these unfavorable conditions will be exacerbated during periods of wet weather.
4. After the surface has been stripped, the existing subgrade surface should be densified in-place with a heavy sheepsfoot roller operating in static (non-vibratory) mode prior to placement of any new fill. The densification of the surface should be performed under the observation of an S&ME representative. After surface densification but prior to placement of any new fill, have a representative of the Geotechnical Engineer observe the prepared surface for stability. This may consist of a visual observation of a proofroll, performed by the contractor, in all areas to receive fill to confirm stability prior to fill placement.

    A. Where stable conditions cannot be achieved by traditional means (drying, etc.), a soil-reinforcing woven geotextile such as TenCate Mirafi model HP-370 may be required to be placed on the subgrade in order to stabilize the surface sufficiently to allow the first lift of fill material to be placed and compacted. Based upon the results of the proofroll, it may also become necessary to perform undercutting and replacement of unstable soils. This should be a decision made at the time of construction in consultation with the Geotechnical Engineer based upon the conditions observed.

    B. The earthwork should be observed by a representative of the Geotechnical Engineer, so that recommendations regarding the undercut depth and the use of geotextiles in excavation bottoms can be made at the time of construction.

5. Pavement areas should also be proofrolled at soil subgrade elevation under the observation of a representative of the Geotechnical Engineer (S&ME). If any areas of instability are observed during the proofroll, further stabilization should be performed, as determined by the Geotechnical Engineer.

6.2 Fill Placement and Compaction

Where new fill soils are to be placed, the following recommendations apply:

1. Prior to fill placement, sample and test each proposed fill material to determine suitability for use, maximum dry density, optimum moisture content, and natural moisture content.

A. It is recommended that the fill soils used to build up the pad for the structure and pavements meet the following minimum requirements: plasticity index of 6 percent or less (ASTM D4318); clay/silt fines content of not greater than 15 percent (ASTM D1140); moisture content within 3 percent of the optimum moisture content for compaction (ASTM D1557). Typically, this would include USCS soil classifications SW, SP, SW-SC, SW-SM, SP-SC, and SP-SM.
B. Based upon our laboratory test data, the soils observed within the hand auger borings do not appear likely to meet these criteria due to wetness, excess clay content, and excess plasticity, so it should be anticipated that near-surface on-site soils that are excavated during construction are likely to be unsuitable to re-use as structural backfill. Therefore, the contractor should plan to import all fill soil to be used for the site development.
C. The proposed pond area was not explored; however, it is likely that the soils within the proposed pond are similar to the rest of the site and the near surface soils will not be suitable for use as fill.

2. Where fill soil is required, the first lift of fill placed over the native clay subgrade should be compacted to at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557). The remainder of the structural fill (other than the first lift) should be compacted throughout to **at least 98 percent** of the modified Proctor maximum dry density (ASTM D 1557).

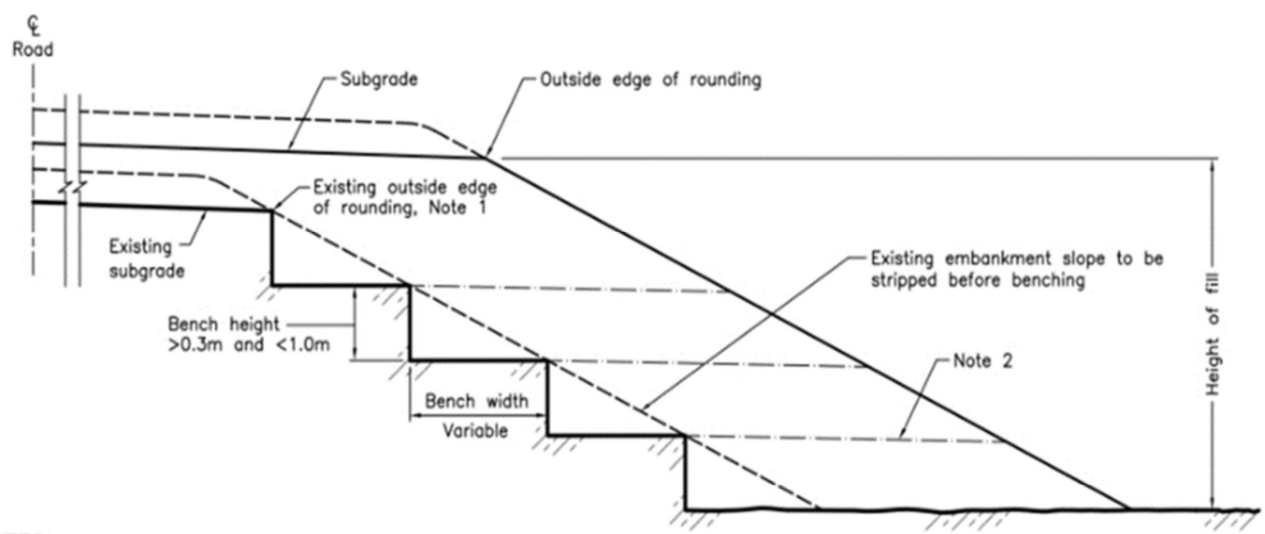
   A. This is a higher degree of compaction than is normally specified for commercial projects, but is appropriate in this case due to the very heavy loads that are transferred to the subgrade by the fire trucks that are anticipated to use this facility.
   B. Compacted soils should not exhibit pumping or rutting under equipment traffic.
   C. The first lift of structural fill placed over the native clay subgrade may be placed 12 inches thick. Loose lifts of fill after the first lift should be no more than 10 inches thick prior to compaction; reduce the maximum lift thickness to 6 inches if using small, portable compaction equipment such as walk-behind vibrating plate tamps or reciprocating tamps ("jumping jacks").
   D. Structural fill should extend at least 5 feet from the edge of structures and pavements before being allowed to exhibit a lower level of compaction.

3. Where present, the subsurface water level should be maintained at least 2 feet below any surface to be densified prior to beginning compaction. This is to reduce the risk of the compaction operations drawing water up to the surface and deteriorating it.
4. All fill placement should be witnessed by an experienced S&ME soils technician working under the guidance of the Geotechnical Engineer. In general, at least one field density test for every 2,500 square feet should be conducted for each lift of soil in large area fills, with a minimum of 2 tests per lift. At least one field density test should be conducted for each 150 cubic feet of fill placed in confined areas such as isolated undercuts and in trenches, with a minimum of 1 test per lift.

6.2.1 Ditch Filling

The ditch that runs parallel to Burcale Road will need to be mucked of all soft sediments prior to fill placement for the proposed pavement in this area. The side slopes of any ditches must also be properly benched to accommodate the placement of new fill in horizontal lifts. Fill placed within these areas should be notched into the embankment using a benching procedure as shown in Figure 6-1 below, and the fill lifts shall be placed horizontally into the benches or notches. It is not recommended to place the fill in diagonal lifts parallel to the embankment slope, because this method decreases the stability of the fill and could create a slip plane. Once prepared, have a representative of the Geotechnical Engineer observe all pond and ditch excavations prior to backfilling, to confirm that they are in a suitable condition to receive new fill.

Figure 6-1: Example Benching Diagram for Slopes <3H:1V



NOTES:

1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.

2 Benches are to be excavated one level at a time and the fill placed and compacted before the next bench is excavated.

6.3 Shallow Foundations

The following recommendations are provided for the design and construction of shallow foundations at this site for the proposed structure.

1. The proposed building may be supported on shallow foundations using isolated footings and slab-on-grade construction as planned. A net available bearing pressure of up to 2,000 psf should be used for design of individual spread footings and wall footings that are extended to bear within structural fill compacted as recommended in Section 6.2 of this report.
2. It should be anticipated that where footings bear directly on fill, the previously placed fill soils exposed in the bottom of the footings may need to be tamped to increase their density prior to the placement of foundation concrete. Also, foundations which are extended to bear within or near the soft clays of Stratum I are likely to require over-excavation and replacement with No. 57 or No. 67 stone of the upper few feet of clays to provide proper bearing support to the structure. This should be a decision made at

the time of construction in consultation with the Geotechnical Engineer based on the results of DCP testing performed by a soils technician in each footing excavation.

3. Even if smaller dimensions are theoretically allowable from a bearing pressure consideration, the minimum wall footing width should be at least 18 inches, and the minimum column footing width should be 30 inches, to avoid punching shear. Footings should be embedded to a minimum depth of at least 12 inches, or the depth specified on the drawings, whichever is greater.
4. Have a representative of the Geotechnical Engineer (S&ME) observe and test each cleaned footing excavation prior to concrete placement to measure that the required level of soil compaction and bearing capacity is present at the foundation bearing surface. Also, have a representative of the Geotechnical Engineer observe any undercut areas in footings prior to backfilling, in order to confirm that poor soils have been removed and that the exposed subgrade is suitable for support of footings or backfill.
5. For the purposes of settlement estimation, we assumed the structures will be constructed near existing grade elevations, with a maximum grade elevation increase above existing grades of 3 feet. If grades will increase by more than 3 feet in elevation above existing grade, then additional settlement due to the dead weight of the fill embankment may occur, and this needs to be considered because it could cause the total settlements to exceed 1 inch:

   A. Considering a 75 kip column load, a 250 psf uniform area load, including the slab, and a 2,000 psf spread footing bearing pressure, the estimated post-construction static settlement of a typical column footing will likely be on the order of 1 inch or less.
   B. Considering a 5 kip per linear foot wall load and a 250 psf uniform area load including the slab, and a 2,000 psf spread footing bearing pressure, the estimated post construction static settlement of a typical wall strip footing will likely be on the order of 1 inch or less.
   C. Differential settlements between individual walls and columns are typically on the order of 50 percent of the maximum total settlement value under static loading, or in this case, 1/2 inch or less.

6.4 Grade Slab Support and Construction

The following recommendations are given for the support and construction of soil-supported grade slabs:

1. Soils similar to those recommended for use as imported structural fill in Section 6.2 of this report are anticipated to provide adequate support to proposed soil-supported grade slabs, assuming preparation and compaction of the subgrade as recommended above. A modulus of subgrade reaction (k) of 150 lbs/in^3 (pci) is recommended for use for reinforcing design.
2. A plastic vapor barrier should be placed over the subgrade prior to placing concrete to limit moisture infiltration into finished spaces.
3. Place a layer of at least 4 inches of compacted granular materials below the interior floor slab. Granular materials used may consist of a crushed, well-graded gravel blend such as SCDOT Graded Aggregate Base Course (GABC), or an open-graded, manufactured washed gravel such as SCDOT No. 57 or No. 67 stone.

   A. If washed (No. 57/67) gravel is used as the underslab layer, then the contractor should plan on using a pump truck to place the floor slab concrete since these materials are cohesionless and are difficult to drive vehicles on.

   B. If GABC is used, then either a pump truck or direct discharge from concrete batch trucks may be appropriate depending upon the circumstances.
   C. If GABC is used, this underslab layer should be compacted to at least 98 percent of the modified Proctor maximum dry density (ASTM D 1557), and tested for density by a representative of S&ME.

4. Have a representative of the Geotechnical Engineer observe a proofroll of all slab subgrades prior to concrete placement. Softened soils may need to be undercut or stabilized before concrete placement.

6.5 Pavement Section Design and Construction

Flexible (asphalt) pavements are not recommended for use in areas that will be traveled by the fire trucks and rescue vehicles. Only rigid Portland cement concrete pavements should be used in those areas; see also Figure 5 in Appendix I. Flexible pavements may only be used in the employee parking lot.

We assume that new pavement subgrades will be constructed atop compacted structural fill soils compacted to **at least 98 percent** of the modified Proctor maximum dry density. This is a higher degree of compaction than is normally specified for commercial projects, but is appropriate in this case due to the very heavy loads that are transferred to the subgrade by the fire trucks that are anticipated to use this facility. We have performed our evaluations assuming that a CBR value of at least 10 percent will be available from structural fill soils compacted to 98 percent, which is typical of well compacted sandy soils. If soils exhibiting a CBR value of less than 10 percent at 98 percent compaction are to be used on this project, these recommendations may require revision.

Traffic volumes for the proposed development were not provided to us in preparation for our exploration and pavement section analysis. Based upon our previous experience on similar fire station projects, we have assumed traffic load information. A required capacity of about 2,000,000 Equivalent Single Axle Loads (ESALs) was estimated for the rigid (concrete) pavements subjected to fire truck/rescue traffic. The volumes for light-duty asphalt pavements are based on an assumption of 60 passenger vehicle or light truck trips per day. Both sections assume a design life of 20 years. The resulting recommended pavement section components are provided in Table 6-1 below.

For flexible pavements, the pavement thickness computations were made using the AASHTO method, assuming an initial serviceability of 4.2 and a terminal serviceability index of 2.0, and a reliability factor of 95 percent. Assuming that only SCDOT approved source materials will be used in flexible pavement section construction, we used a structural layer coefficient of 0.44 for the HMA layers and a coefficient of 0.18 for the graded aggregate base course (GABC).

Rigid pavement design assumes an initial serviceability of 4.5 and a terminal serviceability index of 2.5, and a reliability factor of 90 percent. Assuming that the rigid pavement will be continuously reinforced, we used an average load transfer coefficient of 3.2. We also assumed a minimum 28-day design compressive strength of at least 4,000 psi for the PCC. A sub-base drainage factor of 1.0 was assigned, based upon the assumption that the sub-base soils will consist of granular soils.

If reinforced joint design with appropriate load transfer devices (such as steel dowels) is not provided at all expansion and construction joints, then the rigid pavement section thickness design needs to be reconsidered using a higher load transfer coefficient, which may result in an increase in the pavement section thickness to maintain a similar ESAL capacity.

Table 6-1: Recommended Minimum Pavement Sections[a]

Pavement Area	Theoretical Applied Traffic Load 20 years (ESALs)	HMA Surface Course Type C (inches)	4,000 psi Continuously Reinforced Portland Cement Concrete (inches)	SCDOT Graded Aggregate Base Course [GABC] (inches)	No. 57/67 Gravel Drainage Layer (inches) over Mirafi HP-370 Geotextile	Sandy Subbase Fill (inches) over Mirafi HP-370 Geotextile
Heavy-Duty Rigid (Concrete)	2,000,000	---	8.0	6.0	6.0	---
Light-Duty Flexible (Asphalt)	51,000	2.0	---	6.0	---	≥18

(a) Single-stage construction and soil compaction as recommended is assumed; S&ME, Inc. must observe pavement subgrade preparation and pavement installation operations.

We anticipate that the clayey subgrade that is exposed at cut grade elevation in the light-duty employee parking lot may not be stable enough to support the sandy subbase fill material without additional stabilization support; therefore, we recommend the inclusion of a layer of TenCate Mirafi HP-370 on top of the clay subgrade in the employee parking lot.

6.5.1 Pavement Drainage Systems

The site civil engineer should determine the specific layout of the drainage system for the project based on these recommendations.

1. Within the rigid concrete pavement areas, a gravel drainage blanket layer 6 inches in thickness should be constructed along with the proper base course and pavement section. See also Figure 5 in Appendix I for a typical pavement section detail showing the drainage layer.

    A. The drainage layer, located between the soil subgrade and the graded aggregate base course, should consist of a washed, open graded, manufactured granitic gravel meeting the gradation of SCDOT No.57 or No. 67 stone. Do not substitute marine limestone gravel.
    B. Non-woven geotextile filter fabric (TenCate Mirafi 140N) is recommended to be placed between the GABC layer and the drainage layer, to provide separation and filtration;
    C. Woven geotextile (TenCate Mirafi HP-370) is recommended to be placed between the drainage layer and the subgrade to provide separation and tensile reinforcement.
    D. The gravel drainage layer should be at least 6 inches in thickness.

2. Perimeter underdrains should also be considered by the civil engineer for inclusion in the pavement area design of <u>both the heavy-duty and light-duty parking lot areas</u> due to the presence of the shallow clayey soils that may promote the development of near-surface perched water conditions.

   A. The site civil engineer should be consulted regarding the type and location of the perimeter underdrains. Our experience is that two types of underdrain systems are commonly used in this locality, depending upon the traffic application and the preferences of the civil engineer. One commonly used system is a gravel-filled, fabric-wrapped trench, or "French drain" containing an embedded perforated plastic HDPE pipe. This type of underdrain is shown as a typical detail on Figure 5 attached in Appendix I. Another type of system that we often see used is an edge drain product such as AdvanEdge by ADS, Inc. This is a fabric-wrapped, perforated HDPE slot style drain. Some engineers have used a combination of these two systems.
   B. If the civil engineer incorporates perimeter French drains into the subsurface drainage system design, then the French drains should be constructed using the same No. 57 or No. 67 stone, and should be wrapped in a non-woven geotextile, such as Mirafi's 140N Series fabric. French drains should tie into the nearest storm sewer catch basin, or other discharge points as directed by the site civil engineer.

3. Do not fill landscaped islands with clayey or silty (impermeable) spoils that may impede the movement of water into the underdrains.

6.5.2 *General Pavement Section Construction*

The following general recommendations are provided regarding pavement construction:

1. Fill placed in pavement areas should be compacted to at least 98 percent of the modified Proctor (ASTM D 1557) maximum dry density as recommended previously in section 6.2 of this report. Prior to pavement section installation, all exposed pavement area subgrades should be methodically proofrolled at final subgrade elevation under the observation of S&ME, Inc., and any identified unstable areas should be repaired as directed.
2. The stone base course underlying pavements should consist of a graded aggregate base course (GABC) as specified by the SCDOT 2007 Standard Specifications for Highway Construction, Section 305. Proposed materials for use should be provided by a SCDOT-approved source.

   - **Do not substitute "commercial grade" base course for SCDOT-approved base course material.**

3. As stated in the SCDOT Section 305 specification, all new base course should be compacted to at least 100 percent of the modified Proctor maximum dry density (SC T-140). Base courses should not exhibit pumping or rutting under equipment traffic. Heavy compaction equipment is likely to be required in order to achieve the required base course compaction, and the moisture content of the material will likely need to be maintained very near the optimum moisture content in order to facilitate proper compaction. S&ME, Inc. should be contacted to perform field density and thickness testing of the base course prior to paving.
4. Experience indicates that for flexible pavements a thin surface overlay of asphalt pavement may be required in about 10 years due to normal wear and weathering of the surface. Such wear is typically visible in several forms of pavement distress, such as aggregate exposure and polishing, aggregate

stripping, asphalt bleeding, and various types of cracking. There are means to methodically estimate the remaining pavement life based on a systematic statistical evaluation of pavement distress density and mode of failure. We recommend the pavement be evaluated in about 7 years to assess the pavement condition and remaining life.

5. Construct the HMA surface course in accordance with the specifications of Section 403 of the South Carolina Department of Transportation Standard Specifications for Highway Construction (2007 edition). Construct HMA intermediate courses in accordance with the specifications of Section 402 of this same specification.
6. It is important that the asphaltic concrete be properly compacted, as specified in Section 401 of the SCDOT specification. Asphaltic concrete that is insufficiently compacted will show wear much more rapidly than if it were properly compacted. Sufficient testing should be performed during flexible pavement installation to confirm that the required thickness, density, and quality requirements of the pavement specifications are followed.
7. For rigid pavements, we recommend air-entrained ASTM C 94 continuously reinforced Portland cement concrete that will achieve a minimum compressive strength of at least 4,000 psi at 28 days after placement, as determined by ASTM C 39. We also recommend that the pavement concrete be constructed in a manner which at least meets the minimum standards recommended by the American Concrete Institute (ACI).
8. We recommend that at least 1 set of 5 cylinder specimens be cast by S&ME per every 50 cubic yards of concrete placed or at least once per placement event in order to measure achievement of the design compressive strength. We also recommend that S&ME be present on site to observe concrete placement.

7.0 Limitations of Report

This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations in this report are based on the applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, express or implied, is made.

The analyses and recommendations submitted herein are based, in part, upon the data obtained from the subsurface exploration. The nature and extent of variations of the soils at the site to those encountered at our boring and sounding locations may not become evident until construction. If variations appear evident, then we should be provided a reasonable opportunity to re-evaluate the recommendations of this report.

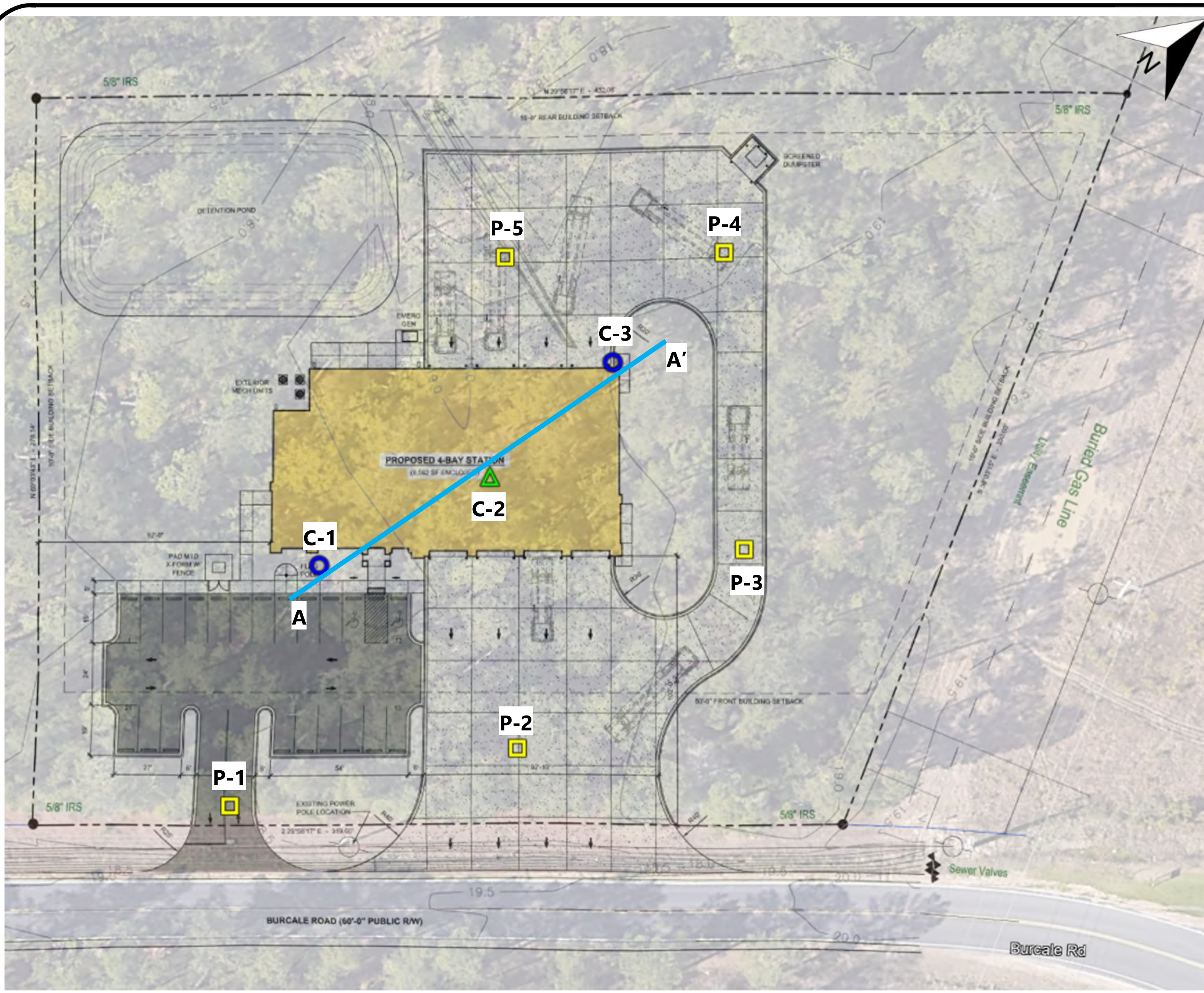
In the event that any changes in the nature, design, or location of the structure are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and conclusions modified or verified in writing by the submitting engineers.

Assessment of site environmental conditions; sampling of soils, ground water or other materials for environmental contaminants; identification of jurisdictional wetlands, rare or endangered species, geological hazards or potential air quality and noise impacts were beyond the scope of this geotechnical exploration.

Appendices

Appendix I – Figures

	Site Vicinity Map	SCALE: AS SHOWN	FIGURE NO. 1
	New Forestbrook Fire Station at Burcale Road Myrtle Beach, South Carolina	DATE: 8-20-2021	
		PROJECT NO. 212687	



LEGEND

Hand Auger Boring and CPT Sounding to Refusal With Seismic Testing

Hand Auger Boring and CPT Sounding to 25′ Without Seismic Testing

Hand Auger Boring With DCP Testing

Subsurface Soil Profile

Test Location Sketch

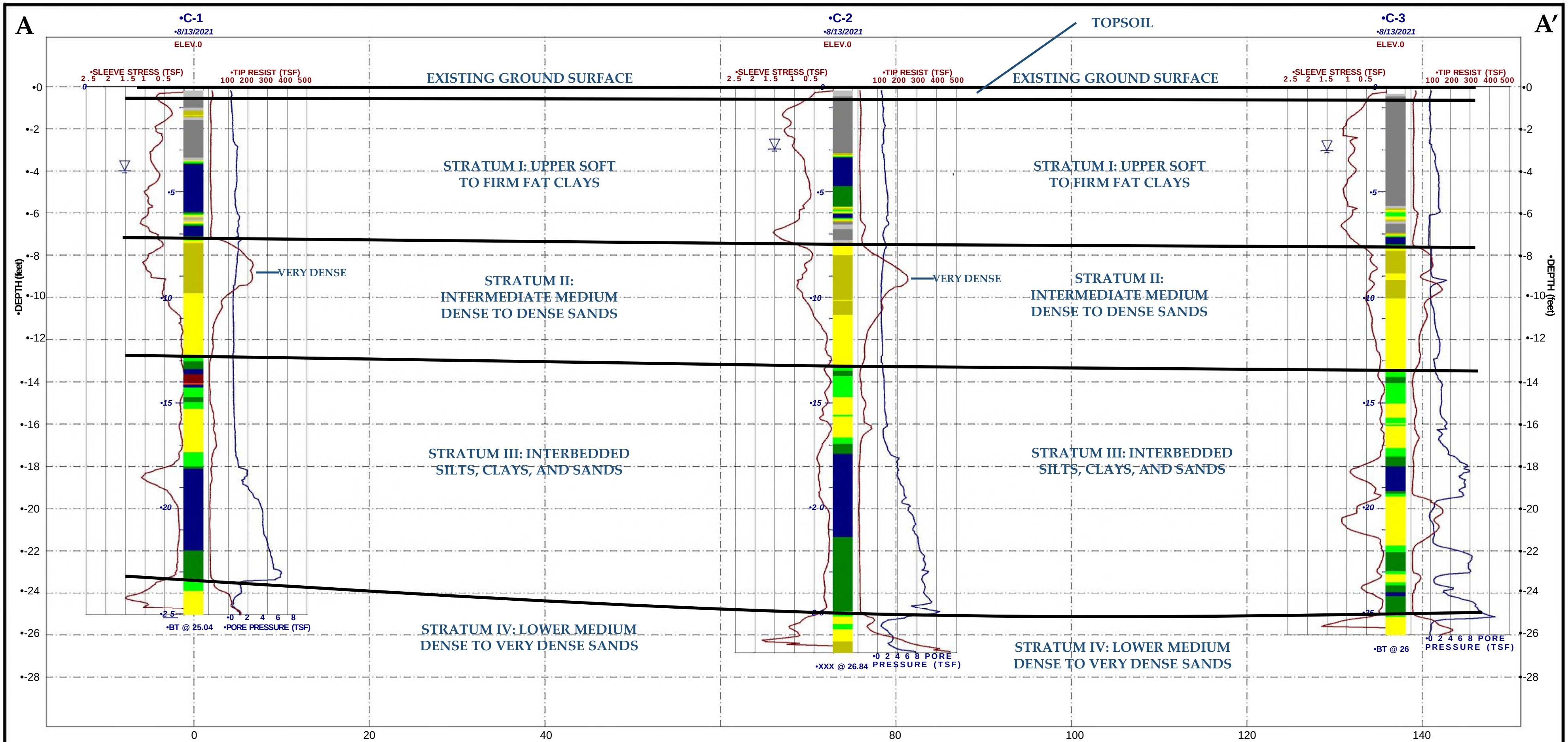
New Forestbrook Fire Station at Burcale Road
Myrtle Beach, South Carolina

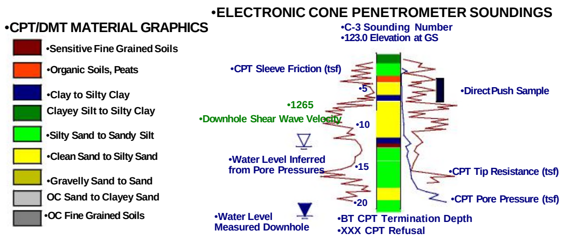
SCALE: AS SHOWN

DATE: 8-20-2021

PROJECT NO. 212687

FIGURE NO. 2





•The depicted stratigraphy is shown for illustrative purposes only and is not warranted. Separations between different
•strata may be gradual and likely vary considerably from those shown. Profiles between nearby borings have been
•estimated using reasonable engineering care and judgment. The actual subsurface conditions will vary between boring locations.

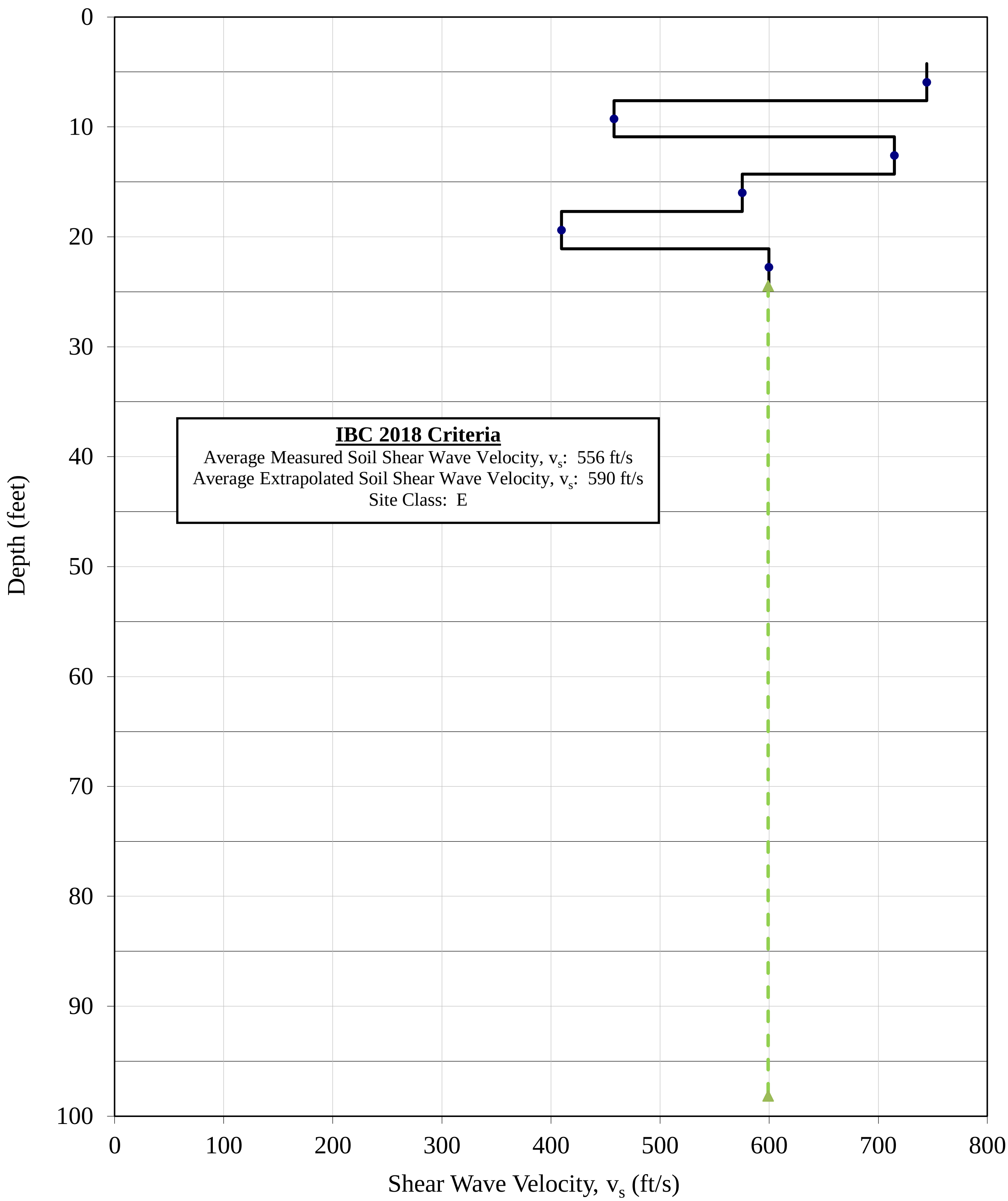
•SUBSURFACE PROFILE	•JOB NO: •212687
•PROJECT: New Forestbrook Fire Station at Burcale Road	
•FIGURE: 3	•DATE: •8/20/21
•LOCATION: Myrtle Beach, SC	

Figure 4: Shear Wave Velocity Calculations

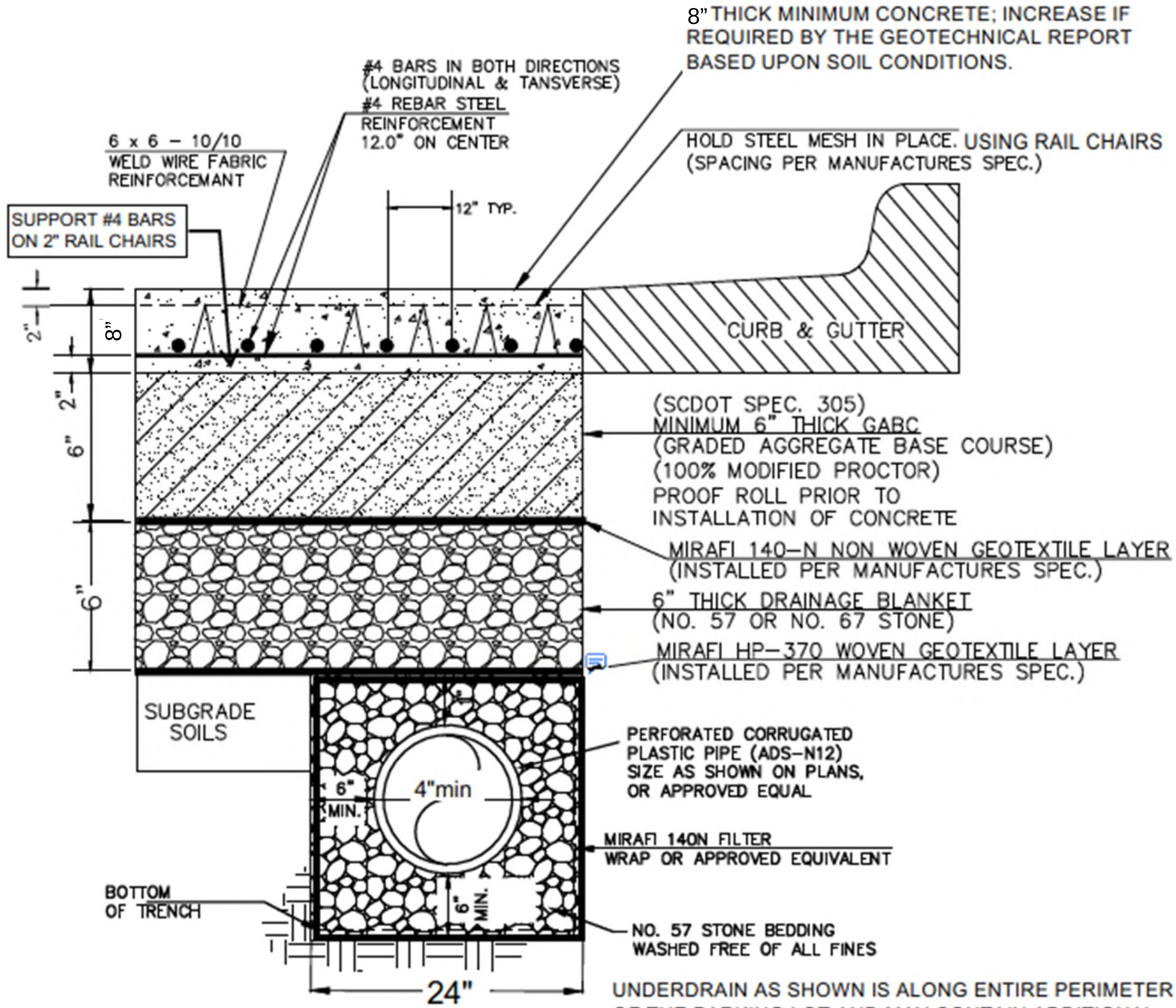
New Forestbrook Fire Station at Burcale Road
Myrtle Beach, SC

Sounding ID: **C-2**
Date: 08/13/21

Project Number: **212687**



* Site Class based on 2018 International Building Code - *Table 1613.5.2 - SITE CLASS DEFINITIONS*



NOTES

1. ALL MATERIALS SHALL MEET APPLICABLE STANDARDS SET FORTH IN THE SCDOT CONSTRUCTION STANDARDS, LATEST EDITION.

2. SUBGRADE SHALL BE FREE OF ALL ROCKS AND DEBRIS LARGER THAN 1" DIAMETER. CERTIFIED COMPACTION TESTS MUST BE SUBMITTED TO ENGINEER PRIOR TO APPLICATION OF BASE MATERIAL.

3. SUBGRADE SHALL BE GRADED AND SLOPED TOWARD DRAINAGE STRUCTURES AND DITCHES TO PREVENT THE ENTRAPMENT OF WATER WITHIN BASE MATERIAL. SUBGRADE SHALL BE INSPECTED AND APPROVED BY OWNERS REPRESENTATIVE PRIOR TO THE INSTALLATION OF BASE.

4. CONCRETE SHALL HAVE THE FOLLOWING 28 DAY MINIMUM COMPRESSIVE STRENGTHS AND SHALL HAVE A MAXIMUM AGGREGATE SIZE OF 3/4".
FOR RENOVATION PROJECTS: USE 5,000 PSI
FOR NEW CONSTRUCTION PROJECTS: USE 4,000 PSI
HIGH-RANGE WATER REDUCER IS RECOMMENDED DUE TO THE STEEL AND WIRE.

CONCRETE PAVEMENT SECTION

NOT TO SCALE

	Pavement Detail	SCALE: AS SHOWN	FIGURE NO.
		DATE: 8/23/21	5
	New Forestbrook Fire Station at Burcale Road Myrtle Beach, South Carolina	PROJECT NUMBER 212687	

Appendix II – Exploration Data

♦ Summary of Exploration Procedures

The American Society for Testing and Materials (ASTM) publishes standard methods to explore soil, rock and ground water conditions in Practice D-420-18, "*Standard Guide for Site Characterization for Engineering Design and Construction Purposes*." The boring and sampling plan must consider the geologic or topographic setting. It must consider the proposed construction. It must also allow for the background, training, and experience of the geotechnical engineer. While the scope and extent of the exploration may vary with the objectives of the client, each exploration includes the following key tasks:

- Reconnaissance of the Project Area
- Preparation of Exploration Plan
- Layout and Access to Field Sampling Locations
- Field Sampling and Testing of Earth Materials
- Laboratory Evaluation of Recovered Field Samples
- Evaluation of Subsurface Conditions

The standard methods do not apply to all conditions or to every site. Nor do they replace education and experience, which together make up engineering judgment. Finally, ASTM D 420 does not apply to environmental investigations.

♦ Reconnaissance of the Project Area

We walked over the site to note land use, topography, ground cover, and surface drainage. We observed general access to proposed sampling points and noted any existing structures.

Checks for Hazardous Conditions - State law requires that we notify the South Carolina (SC 811) before we drill or excavate at any site. SC 811 is operated by the major water, sewer, electrical, telephone, CATV, and natural gas suppliers of South Carolina. SC 811 forwarded our location request to the participating utilities. Location crews then marked buried lines with colored flags within 72 hours. They did not mark utility lines beyond junction boxes or meters. We checked proposed sampling points for conflicts with marked utilities, overhead power lines, tree limbs, or man-made structures during the site walkover.

♦ Boring and Sampling

Electronic Cone Penetrometer (CPT) Soundings

CPT soundings consist of a conical pointed penetrometer which is hydraulically pushed into the soil at a slow, measured rate. Procedures for measurement of the tip resistance and side friction resistance to push generally follow those described by ASTM D-5778, "*Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils*."

A penetrometer with a conical tip having a 60 degree apex angle and a cone base area of 10 cm^2 was advanced into the soil at a constant rate of 20 mm/s. The force on the conical point required to penetrate the soil was measured electronically every 50 mm penetration to obtain the *cone resistance* q_c. A friction sleeve is present on the penetrometer immediately behind the cone tip. The force exerted on the sleeve was measured electronically at a minimum of every 50 mm penetration and divided by the surface area of the sleeve to obtain the *friction* *sleeve resistance value* f_s A pore pressure element mounted immediately behind the cone tip was used to measure the pore pressure induced during advancement of the cone into the soil.

CPT Soil Stratification

Using ASTM D-5778 soil samples are not obtained. Soil classification was made on the basis of comparison of the tip resistance, sleeve resistance and pore pressure values to values measured at other locations in known soil types, using experience with similar soils and exercising engineering judgment.

Plots of normalized tip resistance versus friction ratio and normalized tip resistance versus penetration pore pressure were used to determine soil classification (Soil Behavior Type, SBT) as a function of depth using empirical charts developed by P.K. Robertson (1990). The friction ratio soil classification is determined from the chart in the appendix using the normalized corrected tip stress and the normalized corrected tip stress and the normalized friction ratio.

At some depths, the CPT data fell outside of the range of the classification chart. When this occurred, no data was plotted and a break was shown in the classification profile. This occasionally occurred at the top of a penetration as the effective vertical stress is very small and commonly produced normalized tip resistances greater than 1000.

To provide a simplified soil stratigraphy for general interpretation and for comparison to standard boring logs, a statistical layering and classification system was applied the field classification values. Layer thicknesses were determined based on the variability of the soil classification profile, based upon changes in the standard deviation of the SBT classification number with depth. The average SBT number was determined for each successive 6-inch layer, beginning at the surface. Whenever an additional 6-inch increment deviated from the previous increment, a new layer was started, otherwise, this material was added to the layer above and the next 6-inch section evaluated. The soil behavior type for the layer was determined by the mean value for the complete layer.

Downhole Shear Wave Velocity Test

Shear wave velocity measurements were performed using a cone penetrometer equipped with geophones, or a seismic cone penetrometer (SCPT). The seismic cone penetrometer measures the travel times of surface generated vibrations to geophones mounted on the penetrometer at various incremental depths in the sounding. At a given depth, the travel time of the first arrival is measured and corrected for the horizontal offset of the source at the surface from the sounding. Interval velocities are calculated by dividing the difference in travel times by the vertical distance between successive measurement depths. Measurements were made at 1 meter intervals – the length of commonly available CPT extension rods – unless otherwise noted.

Refusal to CPT Push

Refusal to the cone penetrometer equipment occurred when the reaction weight of the CPT rig was exceeded by the thrust required to push the conical tip further into the ground. At that point the rig tended to lift off the ground. Refusal may have resulted from encountering hard cemented or indurated soils, soft weathered rock, coarse gravel, cobbles or boulders, thin rock seams, or the upper surface of sound continuous rock. Where fills are present, refusal to the CPT rig may also have resulted from encountering buried debris, building materials, or objects.

Hand Auger Borings with Dynamic Cone Penetrometer Testing

Auger borings were advanced using hand operated augers. The soils encountered were identified in the field by cuttings brought to the surface. Soil consistency was qualitatively estimated by the relative difficulty of advancing the augers. Dynamic Cone Penetrometer (DCP) testing was performed in conjunction within the borings in general accordance with ASTM STP 399, "*Dynamic Cone for Shallow In-Situ Penetration Testing*". At selected intervals, the augers were withdrawn and soil consistency measured with a dynamic cone penetrometer. The conical point of the penetrometer was first seated 1-3/4 inches to penetrate any loose cuttings in the boring, then driven two additional 1-3/4 inch increments by a 15 pound hammer falling 20 inches. The number of hammer blows required to achieve this penetration was recorded. When properly evaluated by qualified professional staff, the blow count is an index to the soil strength. Hand auger borings were backfilled with soil cuttings after termination of drilling. Soil cuttings removed from each hole were collected as a bulk sample for laboratory testing.

Hand Auger Borings without Dynamic Cone Penetrometer

Auger borings were advanced using hand operated augers. The soils encountered were identified in the field by cuttings brought to the surface.   Representative samples of the cuttings were placed in glass jars or plastic bags and later transported to the laboratory. Soil consistency was qualitatively estimated by the relative difficulty of advancing the augers.

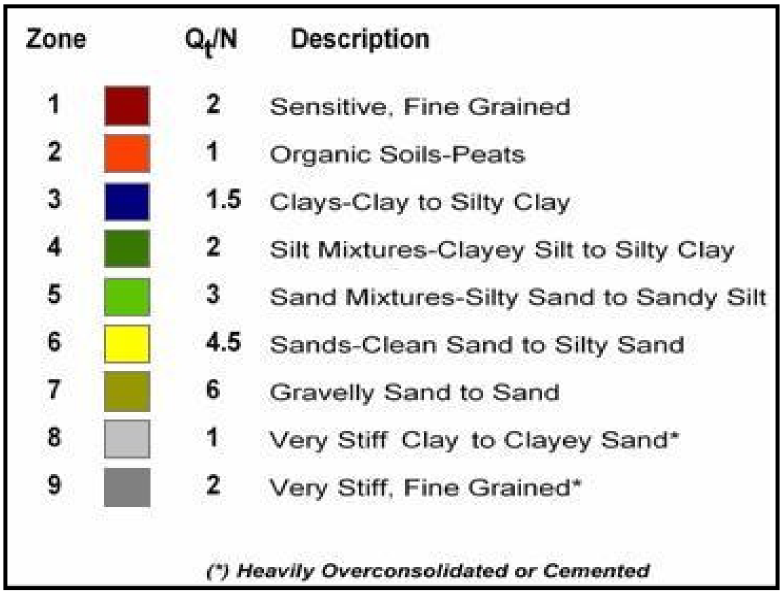
Water Level Measurement

Subsurface water levels in the boreholes were measured during the onsite exploration by measuring depths from the existing grade to the current water level using a tape.

Backfilling of Boreholes

Upon completion of the boreholes and measurement of the water level in the hole, each boring was backfilled with soil cuttings to existing ground surface.

CPT Soil Classification Legend



Zone		Q_t/N	Description
1		2	Sensitive, Fine Grained
2		1	Organic Soils-Peats
3		1.5	Clays-Clay to Silty Clay
4		2	Silt Mixtures-Clayey Silt to Silty Clay
5		3	Sand Mixtures-Silty Sand to Sandy Silt
6		4.5	Sands-Clean Sand to Silty Sand
7		6	Gravelly Sand to Sand
8		1	Very Stiff Clay to Clayey Sand*
9		2	Very Stiff, Fine Grained*

(*) Heavily Overconsolidated or Cemented

Robertson's Soil Behavior Type (SBT), 1990			
Group #	**Description**	**Ic**	
		Min	**Max**
1	Sensitive, fine grained	N/A	
2	Organic soils - peats	3.60	N/A
3	Clays - silty clay to clay	2.95	3.60
4	Silt mixtures - clayey silt to silty clay	2.60	2.95
5	Sand mixtures - silty sand to sandy silt	2.05	2.60
6	Sands - clean sand to silty sand	1.31	2.05
7	Gravelly sand to dense sand	N/A	1.31
8	Very stiff sand to clayey sand (High OCR or cemented)	N/A	
9	Very stiff, fine grained (High OCR or cemented)	N/A	

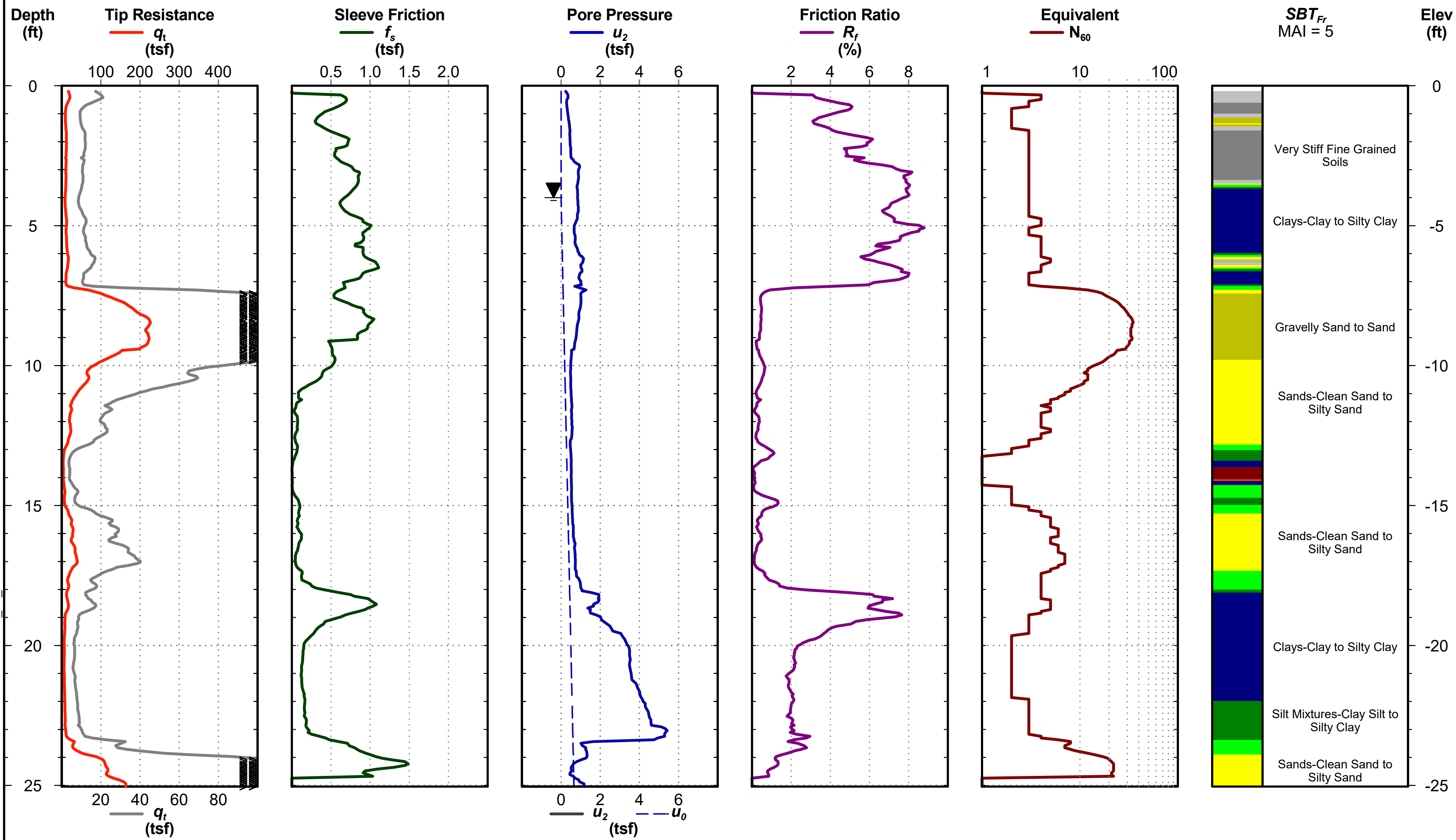
Soil behavior type is based on empirical data and may not be representative of soil classification based on plasticity and grain size distribution.

Relative Density and Consistency Table			
SANDS		SILTS and CLAYS	
Cone Tip Stress, qt (tsf)	Relative Density	Cone Tip Stress, qt (tsf)	Consistency
Less than 20	Very Loose	Less than 5	Very Soft
20 - 40	Loose	5 - 15	Soft to Firm
40 - 120	Medium Dense	15 - 30	Stiff
120 - 200	Dense	30 - 60	Very Stiff
Greater than 200	Very Dense	Greater than 60	Hard

Sounding ID: C-1

New Forestbrook Fire Station at Burcale Road
Myrtle Beach, SC
S&ME Project No: 212687

Date: Aug. 13, 2021
Estimated Water Depth: 4 ft
Rig/Operator: Marooka/T. Whitehead
Total Depth: 25.0 ft
Termination Criteria: Target Depth
Cone Size: 1.75



Cone Penetration Test

CPT REPORT - DYNAMIC \ CPT.GPJ \ LIBRARY 2011_06_28.GDT \ 8/23/21

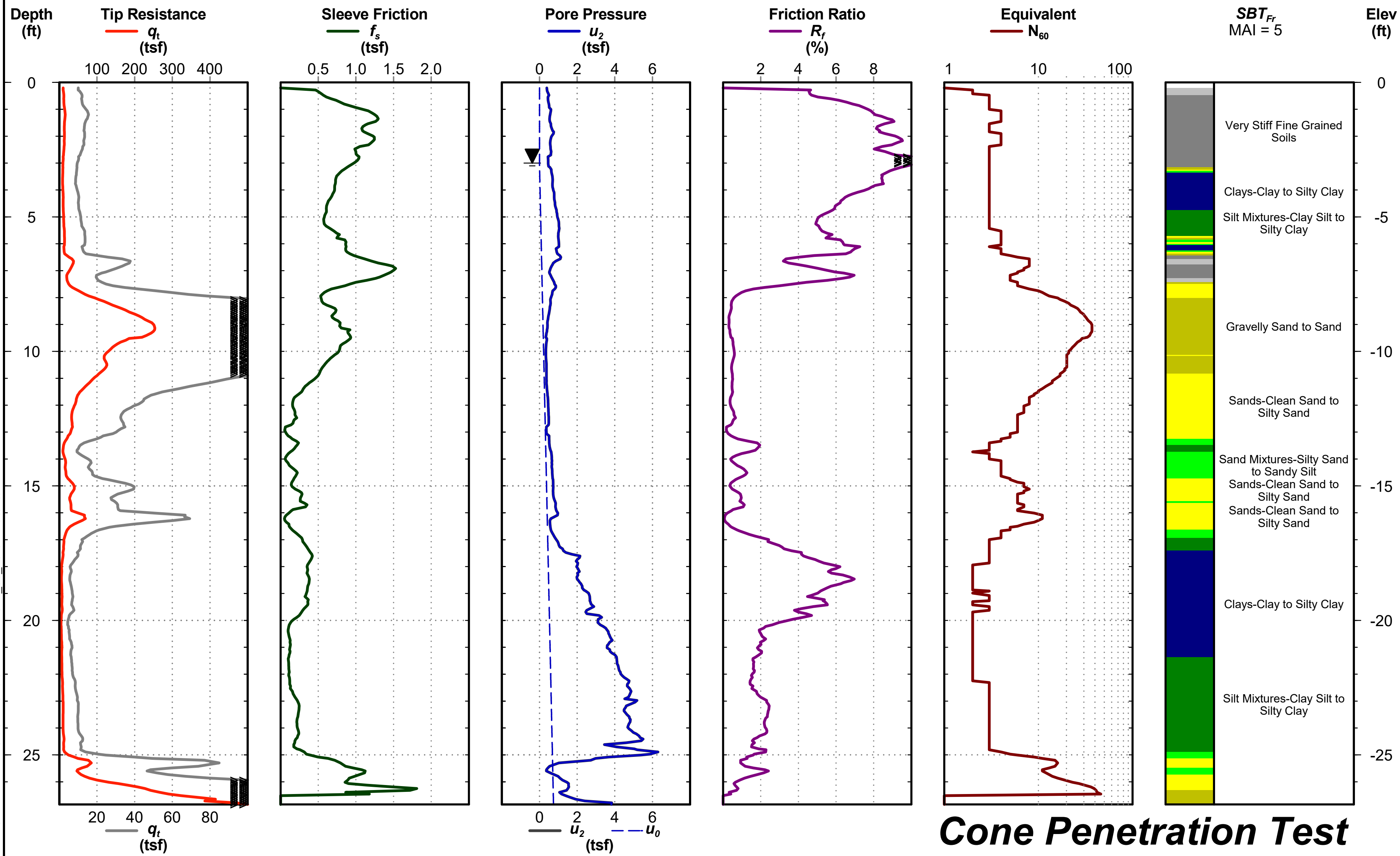
Electronic Filename: C-1_PD.ECP

Sounding ID: C-2

New Forestbrook Fire Station at Burcale Road
Myrtle Beach, SC
S&ME Project No: 212687

Date: Aug. 13, 2021
Estimated Water Depth: 3 ft
Rig/Operator: Marooka/T. Whitehead

Total Depth: 26.8 ft
Termination Criteria: Maximum Reaction Force
Cone Size: 1.75



Cone Penetration Test

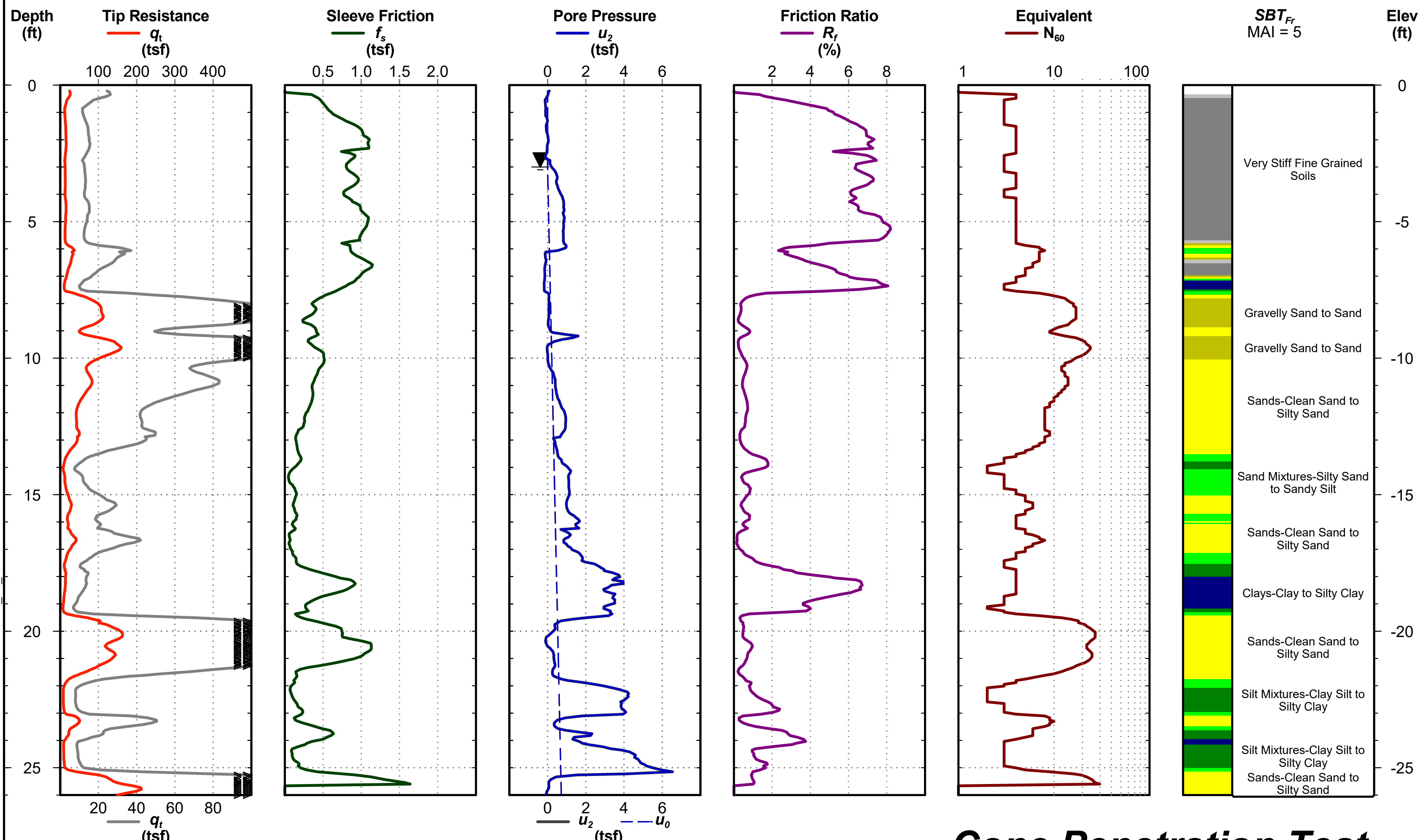
CPT REPORT - DYNAMIC \ CPT.GPJ \ LIBRARY 2011_06_28.GDT \ 8/23/21

Electronic Filename: C-2_PD.ECP

Sounding ID: C-3

New Forestbrook Fire Station at Burcale Road
Myrtle Beach, SC
S&ME Project No: 212687

Date: Aug. 13, 2021
Estimated Water Depth: 3 ft
Rig/Operator: Marooka/T. Whitehead
Total Depth: 26.0 ft
Termination Criteria: Target Depth
Cone Size: 1.75



Cone Penetration Test

Electronic Filename: C-3_PD.ECP

LEGEND TO SOIL CLASSIFICATION AND SYMBOLS

SOIL TYPES

(Shown in Graphic Log)

- Fill
- Asphalt
- Concrete
- Topsoil
- Gravel
- Sand
- Silt
- Clay
- Organic
- Silty Sand
- Clayey Sand
- Sandy Silt
- Clayey Silt
- Sandy Clay
- Silty Clay
- Partially Weathered Rock
- Cored Rock

WATER LEVELS

(Shown in Water Level Column)

▽ = Water Level At Termination of Boring
▼ = Water Level Taken After 24 Hours
◄ = Loss of Drilling Water
HC = Hole Cave

CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	STD. PENETRATION RESISTANCE BLOWS/FOOT
Very Soft	0 to 2
Soft	3 to 4
Firm	5 to 8
Stiff	9 to 15
Very Stiff	16 to 30
Hard	31 to 50
Very Hard	Over 50

RELATIVE DENSITY OF COHESIONLESS SOILS

RELATIVE DENSITY	STD. PENETRATION RESISTANCE BLOWS/FOOT
Very Loose	0 to 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	Over 50

SAMPLER TYPES

(Shown in Samples Column)

- Shelby Tube
- Split Spoon
- Rock Core
- No Recovery

TERMS

Standard Penetration Resistance - The Number of Blows of 140 lb. Hammer Falling 30 in. Required to Drive 1.4 in. I.D. Split Spoon Sampler 1 Foot. As Specified in ASTM D-1586.

REC - Total Length of Rock Recovered in the Core Barrel Divided by the Total Length of the Core Run Times 100%.

RQD - Total Length of Sound Rock Segments Recovered that are Longer Than or Equal to 4" (mechanical breaks excluded) Divided by the Total Length of the Core Run Times 100%.

PROJECT: New Forestbrook Fire Station at Burcale Road Myrtle Beach, South Carolina 212687	**HAND AUGER BORING LOG: C-1**

DATE STARTED: **8/5/21**	DATE FINISHED: **8/5/21**	NOTES: Elevation Unknown.
SAMPLING METHOD: **Grab Sample**	PERFORMED BY: **J. Lighthall**	
WATER LEVEL: **Not encountered.**		

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
		TOPSOIL - Approximately 6 inches thick.		
1 2 3 4		**FAT CLAY (CH)** - Mostly medium plasticity fines, trace fine sand, trace organics, tan, orange, and gray, moist.		
		Boring terminated at 4 ft		

DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

PROJECT: New Forestbrook Fire Station at Burcale Road Myrtle Beach, South Carolina 212687	**HAND AUGER BORING LOG: C-2**

DATE STARTED: **8/5/21**	DATE FINISHED: **8/5/21**	NOTES: Elevation Unknown.
SAMPLING METHOD: **Grab Sample**	PERFORMED BY: **J. Lighthall**	
WATER LEVEL: **Not encountered.**		

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
		TOPSOIL - Approximately 7 inches thick.		
		FAT CLAY (CH) - Mostly medium plasticity fines, few organics, trace fine sand, tan, orange, and gray, moist.		
1				
2				
3				
4		Boring terminated at 4 ft		

DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

PROJECT: New Forestbrook Fire Station at Burcale Road Myrtle Beach, South Carolina 212687	**HAND AUGER BORING LOG: C-3**

DATE STARTED: **8/5/21**	DATE FINISHED: **8/5/21**	NOTES: Elevation Unknown.
SAMPLING METHOD: **Grab Sample**	PERFORMED BY: **J. Lighthall**	
WATER LEVEL: **Not encountered.**		

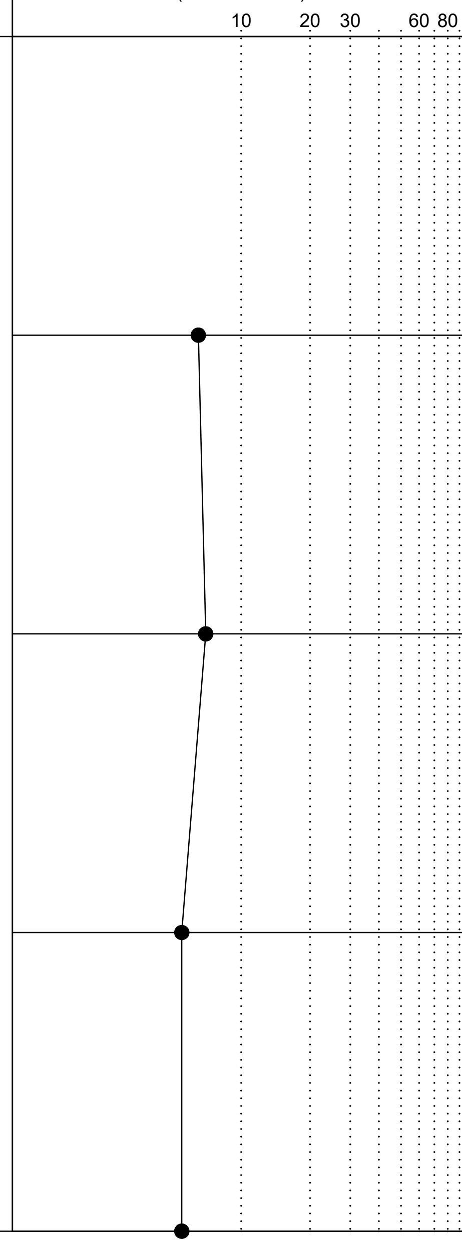
Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
		TOPSOIL - Approximately 8 inches thick.		
1 2 3 4		**FAT CLAY (CH)** - Mostly medium fines, few organics, trace fine sand, tan, orange, and gray, moist.		

Boring terminated at 4 ft

PROJECT:	New Forestbrook Fire Station at Burcale Road Myrtle Beach, South Carolina 212687	**HAND AUGER BORING LOG: P-1**

				NOTES:
DATE STARTED:	**8/5/21**	DATE FINISHED:	**8/5/21**	Elevation Unknown.
SAMPLING METHOD:	**Grab Sample**	PERFORMED BY:	**J. Lighthall**	
WATER LEVEL:	**Not encountered.**			

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL	DYNAMIC CONE PENETRATION RESISTANCE (blows/1.75 in.)	DCP VALUE
		TOPSOIL - Approximately 6 inches thick.				
		FAT CLAY (CH) - Mostly medium fines, few organics, trace fine sand, tan, orange, and gray, dry, firm.				
1		- - - - Moist.				7
2						7
3						6
4		Boring terminated at 4 ft				6



DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

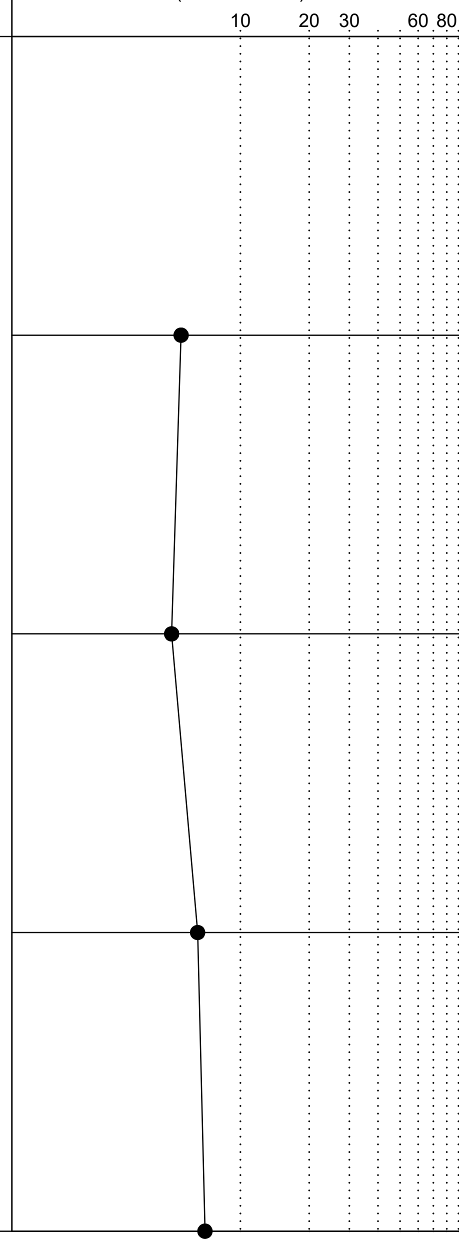
PROJECT:	New Forestbrook Fire Station at Burcale Road Myrtle Beach, South Carolina 212687	**HAND AUGER BORING LOG: P-2**

DATE STARTED:	**8/5/21**	DATE FINISHED:	**8/5/21**
SAMPLING METHOD:	**Grab Sample**	PERFORMED BY:	**J. Lighthall**
WATER LEVEL:	**Not encountered.**		

NOTES:
Elevation Unknown.

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL	DCP VALUE
		TOPSOIL - Approximately 5 inches thick.			
		FAT CLAY (CH) - Mostly medium plasticity fines, few organics, trace fine sand, tan, orange, and gray, dry, firm.			
		---- Moist.			
1					6
2					5
3					7
4		Boring terminated at 4 ft			7

DYNAMIC CONE PENETRATION RESISTANCE (blows/1.75 in.)



DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

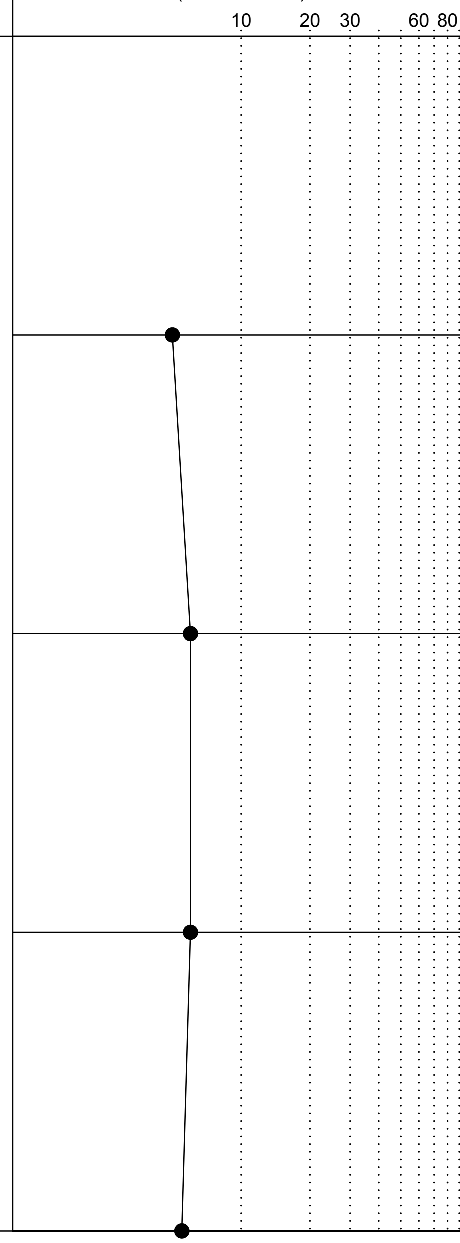
PROJECT: **New Forestbrook Fire Station at Burcale Road**
Myrtle Beach, South Carolina
212687

HAND AUGER BORING LOG: P-3

DATE STARTED:	**8/5/21**	DATE FINISHED:	**8/5/21**
SAMPLING METHOD:	**Grab Sample**	PERFORMED BY:	**J. Lighthall**
WATER LEVEL:	**Not encountered.**		

NOTES:
Elevation Unknown.

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL	DYNAMIC CONE PENETRATION RESISTANCE (blows/1.75 in.)	DCP VALUE
		TOPSOIL - Approximately 7 inches thick.				
		FAT CLAY (CH) - Mostly medium plasticity fines, few organics, trace fine sand, tan, orange, and gray, dry, firm.				
		- - - - Moist.				
1						5
2						6
3						6
4		Boring terminated at 4 ft				6

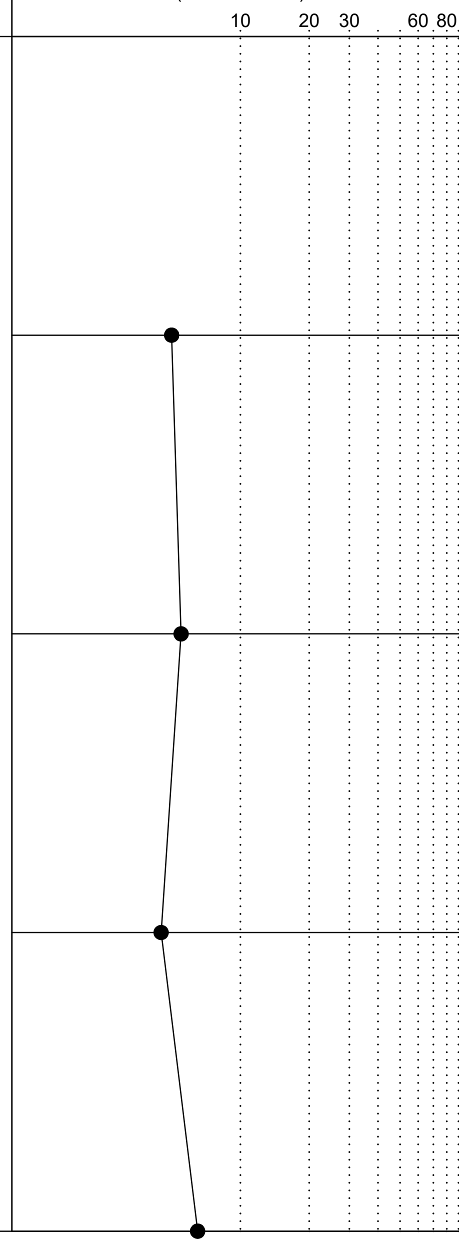


DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

PROJECT: **New Forestbrook Fire Station at Burcale Road** **Myrtle Beach, South Carolina** **212687**	**HAND AUGER BORING LOG: P-4**

DATE STARTED: **8/5/21**	DATE FINISHED: **8/5/21**	NOTES: Elevation Unknown.
SAMPLING METHOD: **Grab Sample**	PERFORMED BY: **J. Lighthall**	
WATER LEVEL: **Not encountered.**		

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL	DYNAMIC CONE PENETRATION RESISTANCE (blows/1.75 in.)	DCP VALUE
0		**TOPSOIL** - Approximately 8 inches thick.				
		FAT CLAY (CH) - Mostly medium plasticity fines, few organics, trace fine sand, tan, orange, and gray, moist, firm.				
1						5
2						6
3						5
4		Boring terminated at 4 ft				7

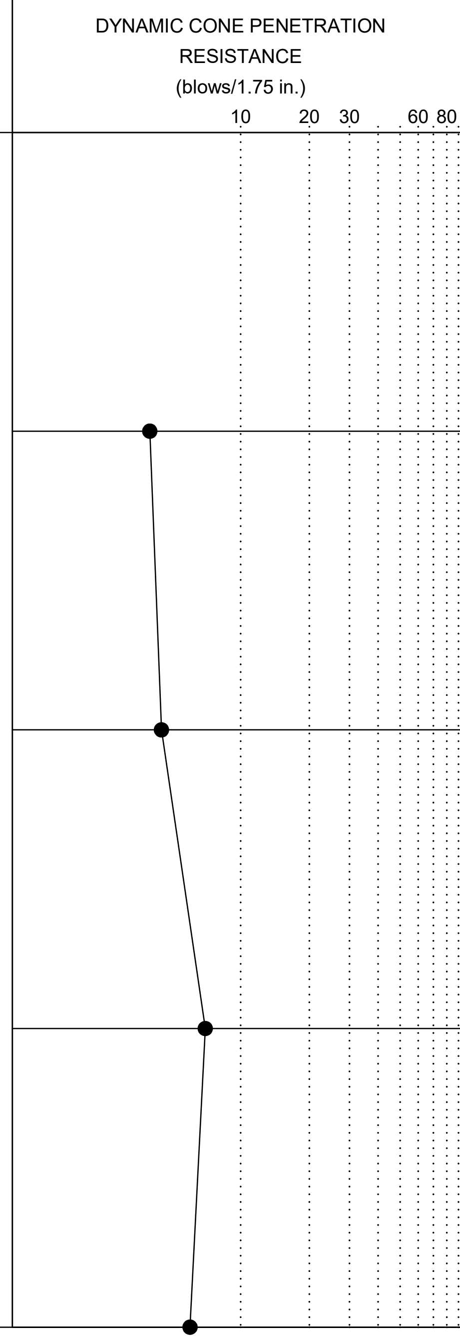


DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

PROJECT: New Forestbrook Fire Station at Burcale Road Myrtle Beach, South Carolina 212687	**HAND AUGER BORING LOG: P-5**

DATE STARTED: **8/5/21**	DATE FINISHED: **8/5/21**	NOTES: Elevation Unknown.	
SAMPLING METHOD: **Grab Sample**	PERFORMED BY: **J. Lighthall**		
WATER LEVEL: **Not encountered.**			

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL	DYNAMIC CONE PENETRATION RESISTANCE (blows/1.75 in.)	DCP VALUE
		TOPSOIL - Approximately 2 inches thick.				
		FAT CLAY (CH) - Mostly medium plasticity fines, trace fine sand, trace organics, tan, orange, and red, dry, soft to firm.				
1		- - - - Moist, soft.				4
2		- - - - Firm.				5
3						7
4		Boring terminated at 4 ft				6



DCP INDEX IS THE DEPTH (IN.) OF PENETRATION PER BLOW OF A 10.1 LB HAMMER FALLING 22.6 IN., DRIVING A 0.79 IN. O.D. 60 DEGREE CONE.

Appendix III – Laboratory Data

♦ Summary of Laboratory Procedures

Examination of Recovered Soil Samples

Soil and field records were reviewed in the laboratory by the geotechnical professional. Soils were classified in general accordance with the visual-manual method described in ASTM D 2488, "*Standard Practice for Description and Identification of Soils (Visual-Manual Method)*". Representative soil samples were selected for classification testing to provide grain size and plasticity data to allow classification of the samples in general accordance with the Unified Soil Classification System method described in ASTM D 2487, "*Standard Practice for Classification of Soils for Engineering Purposes*". The geotechnical professional also prepared the final boring and sounding records enclosed with this report.

Moisture Content Testing of Soil Samples by Oven Drying

Moisture content was determined in general conformance with the methods outlined in ASTM D 2216, "*Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil or Rock by Mass*." This method is limited in scope to Group B, C, or D samples of earth materials which do not contain appreciable amounts of organic material, soluble solids such as salt or reactive solids such as cement. This method is also limited to samples which do not contain contamination.

A representative portion of the soil was divided from the sample using one of the methods described in Section 9 of ASTM D 2216. The split portion was then placed in a drying oven and heated to approximately 110 degrees C overnight or until a constant mass was achieved after repetitive weighing. The moisture content of the soil was then computed as the mass of water removed from the sample by drying, divided by the mass of the sample dry, times 100 percent. No attempt was made to exclude any particular particle size from the portion split from the sample.

Grain Size Analysis of Samples

The distribution of particle sizes greater than 75 mm was determined in general accordance with the procedures described by ASTM D 421, "*Standard Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants*", and D 6913, *"Standard Test Method for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis*,". During preparation samples were divided into two portions. The material coarser than the No. 30 U.S. sieve size fraction was dry sieved through a nest of standard sieves as described in Article 6. Material passing the No. 30 sieve was independently passed through a nest of sieves down to the No. 200 size.

Percent Fines Determination of Samples

A selected specimen of soils was washed over a No. 200 sieve after being thoroughly mixed and dried. This test was conducted in general accordance with ASTM D 1140, "*Standard Test Method for Amount of Material Finer Than the No. 200 Sieve*." Method A, using water to wash the sample through the sieve without soaking the sample for a prescribed period of time, was used and the percentage by weight of material washing through the sieve was deemed the "percent fines" or percent clay and silt fraction.

Liquid and Plastic Limits Testing

Atterberg limits of the soils was determined generally following the methods described by ASTM D 4318, "*Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils*." Albert Atterberg originally defined "limits of consistency" of fine grained soils in terms of their relative ease of deformation at various moisture contents. In current engineering usage, the *liquid lim*it of a soil is defined as the moisture content, in percent, marking the upper limit of viscous flow and the boundary with a semi-liquid state. The *plastic limit* defines the lower limit of plastic behavior, above which a soil behaves plastically below which it retains its shape upon drying. The *plasticity index* (PI) is the range of water content over which a soil behaves plastically. Numerically, the PI is the difference between liquid limit and plastic limit values.

Representative portions of fine grained Group A, B, C, or D samples were prepared using the wet method described in Section 10.1 of ASTM D 4318. The liquid limit of each sample was determined using the multipoint method (Method A) described in Section 11. The liquid limit is by definition the moisture content where 25 drops of a hand operated liquid limit device are required to close a standard width groove cut in a soil sample placed in the device. After each test, the moisture content of the sample was adjusted and the sample replaced in the device. The test was repeated to provide a minimum of three widely spaced combinations of N versus moisture content. When plotted on semi-log paper, the liquid limit moisture content was determined by straight line interpolation between the data points at N equals 25 blows.

The plastic limit was determined using the procedure described in Section 17 of ASTM D 4318. A selected portion of the soil used in the liquid limit test was kneaded and rolled by hand until it could no longer be rolled to a 3.2 mm thread on a glass plate. This procedure was repeated until at least 6 grams of material was accumulated, at which point the moisture content was determined using the methods described in ASTM D 2216.

Compaction Tests of Soils Using Standard Effort

Soil placed as engineering fill is compacted to a dense state to obtain satisfactory engineering properties. Laboratory compaction tests provide the basis for determining the percent compaction and water content needed to achieve the required engineering properties, and for controlling construction to assure the required compaction and water contents are achieved. Test procedures generally followed those described by ASTM D 698, "*Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 lbf/ft^3).*"

The relationship between water content and the dry unit weight is determined for soils compacted in either 4 or 6 inch diameter molds with a 5.5 lbf rammer dropped from a height of 12 inches, producing a compactive effort of 12,400 lbf/ft^3. ASTM D 698 provides three alternative procedures depending on material gradation:

Method A

- All material passes No. 4 sieve size
- 4 inch diameter mold
- Shall be used if 25 percent or less by weight is retained on No. 4 sieve
- Soil in 3 layers with 25 blows per layer

Method B

- All material passes 3/8 inch sieve
- 4 inch diameter mold
- Shall be used if 25 percent by weight, or less, is retained on the 3/8 Inch sieve.
- Soil in 3 layers with 25 blows per layer

Method C

- All material passes ¾ inch sieve
- 6-inch diameter mold
- Shall be used if more than 30 percent by weight, or less, is retained on the 3/4 inch sieve
- Soil in 3 layers with 56 blows per layer

Soil was compacted in the mold in three layers of approximately equal thickness, each compacted with either 25 or 56 blows of the rammer. After compaction of the sample in the mold, the resulting dry density and moisture content was determined and the procedure repeated. Separate soils were used for each sample point, adjusting the moisture content of the soil as described in Section 10.2 (Moist Preparation Method). The procedure was repeated for a sufficient number of water content values to allow the dry density vs. water content values to be plotted and the *maximum dry density* and *optimum moisture content* to be determined from the resulting curvilinear relationship.

Laboratory California Bearing Ratio Tests of Compacted Samples

This method is used to evaluate the potential strength of subgrade, subbase, and base course material, including recycled materials, for use in road and airfield pavements. Laboratory CBR tests were run in general accordance with the procedures laid out in ASTM D 1883, "*Standard Test Method for CBR (California Bearing Ratio) of Laboratory Compacted Soils*." Specimens were prepared in standard molds to a target level of compactive effort within plus or minus 0.5 percent of the optimum moisture content value. While embedded in the compaction mold, each sample was inundated for a minimum period of 96 hours to achieve saturation. During inundation the specimen was surcharged by a weight approximating the anticipated weight of the pavement and base course layers. After removing the sample from the soaking bath, the soil was then sheared by jacking a piston having a cross sectional area of 3 square inches into the end surface of the specimen. The piston was jacked 0.5 inches into the specimen at a constant rate of 0.05 inches per minute.

The CBR is defined as the load required to penetrate a material to a predetermined depth, compared to the load required to penetrate a standard sample of crushed stone to the same depth. The CBR value was usually based on the load ratio for a penetration of 0.10 inches, after correcting the load-deflection curves for surface irregularities or upward concavity. However, where the calculated CBR for a penetration of 0.20 inches was greater than the result obtained for a penetration of 0.10 inches, the test was repeated by reversing the specimen and shearing the opposite end surface. Where the second test indicated a greater CBR at 0.20 inches penetration, the CBR for 0.20 inches penetration was used.

Form No: TR-D2216-T265-1
Revision No. 1
Revision Date: 08/16/17

LABORATORY DETERMINATION OF WATER CONTENT

ASTM D 2216 ☑ AASHTO T 265 ☐

S&ME, Inc. - Myrtle Beach: 1330 Highway 501 Business, Conway, SC 29526

Project #:	212687	Report Date:	8/18/2021
Project Name:	New Forestbrook Fire Station at Burcale Rd	Test Date(s):	8/16/2021
Client Name:	Horry County Maintenance Dept.		
Client Address:	307 Smith St; Conway SC 29526		
Sample by:	J. Lighthall	Sample Date(s):	8/5/2021

Method: **A** **(1%)** ☐ **B** **(0.1%)** ☑ Balance ID. 19608 Calibration Date: 2/28/21 Oven ID. 17745 Calibration Date: 4/5/21

Boring No.	Sample No.	Sample Depth	Tare #	Tare Weight	Tare Wt.+ Wet Wt	Tare Wt. + Dry Wt	Water Weight	Percent Moisture	Note
		ft. or m.		grams	grams	grams	grams	%	
P-1 to P-5	Bulk-1	1'-4'	Muave	84.8	158.0	142.7	15.30	26.4%	

Notes / Deviations / References

ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

Ron Forest, P.E.	RPF	Senior Reviewer	8/18/2021
Technical Responsibility	*Signature*	*Position*	*Date*

Form No: TR-D6913-SSSS-1
Revision No. 1
Revision Date: 9/5/17

SOIL SIEVE ANALYSIS USING SINGLE SIEVE-SET SIEVING

Single Portion

ASTM D6913

S&ME, Inc. - Myrtle Beach: 1330 Highway 501 Business, Conway, SC 29526

Project No:	212687	Report Date:	8/18/2021
Project Name:	New Forestbrook Fire Station at Burcale Rd	**Lab #:**	414
Client Name:	Horry County Maintenance Dept.	Test Date:	8/18/2021
Client Address:	307 Smith St; Conway SC 29526	Date Sampled:	8/5/2021
Boring #:	P-1 to P-5	Sample#:	Bulk-1
Location:	Proposed Pavements	Depth:	1'-4'

Sample Description: Tan, Gray, and Orange Fat Clay (CH)

Estimate Max. Particle Size (99% Passing):	**#10**	Testing Dates:	8/18/21
Method A (1%) ☐	Method B (0.1%) ☑	Material Excluded?	None

Procedure for obtaining Specimen:	Moist ☑	Air-Dried ☐	Oven-Dried ☑
Sampling Method	Stockpile: ☑	Mechanically Split: ☐	Quartered: ☐
Dispersion Process?	Soaked without Dispersant ☐	Soaked with Dispersant ☑	Ultrasonic Bath ☐
Estimated Wet Mass of specimen required:	200		Shaking Apparatus ☑

Specimen: Pan No. **Mauve** B) Tare Wt.	**84.8**	Method B of ASTM D1140 or D6913 Sec. 11.4.3	
A) Total Specimen Wet Wt. + Tare Wt. (g.)	**158.0**	Pan No. **Mauve** Tare Wt.	**84.8**
C) Total Specimen Dry Wt. + Tare Wt. (g.)	**142.7**	Dry Mass of Washed Sample +Tare Wt.	**90.3**
D = (C-B) Total Specimen Dry Weight **(S,M_d)**	**57.9**	Dry Mass of Washed Sample **(S_wM_d)**	**5.5**
E = (A-B) Moist Specimen Mass **(S,M_m)**	**73.2**	Dry Mass passing #200	**52.4**
F=(E-D)/D) Water Content of Specimen	**26.4%**	% Passing #200	**90.5%**

Sieve Size		Cumulative Mass Retained	Increment Mass Retained	SPECS	**% Retained**	**% Passing**
					Total Sample Cumulative Percentages	
Standard	mm.	*CMR_N*	*MR_N*	*SCDOT*	*CPR_N*	*PP_N (Method A)*
1.0"	25.00	0.0	0.00		0.0%	100.0%
3/4"	19.00	0.0	0.00		0.0%	100.0%
1/2"	12.50	0.0	0.00		0.0%	100.0%
3/8"	9.50	0.0	0.00		0.0%	100.0%
#4	4.750	0.0	0.00		0.0%	100.0%
#10	2.000	0.0	0.00		0.0%	100.0%
#30	0.600	0.3	0.30		0.5%	99.5%
#40	0.425	0.6	0.30		1.0%	99.0%
#60	0.250	2.2	1.60		3.8%	96.2%
#100	0.150	4.2	2.00		7.3%	92.7%
#200	0.075	5.4	1.20		9.3%	90.7%
Pan	<0.075	5.5	0.1			

Notes/Deviations/References: $PP_N = 100 (1-(CMR_N / S,M_d))$

Ron Forest, P.E.	RPF	Senior Reviewer	8/18/2021
Technical Responsibility	*Signature*	*Position*	*Date*

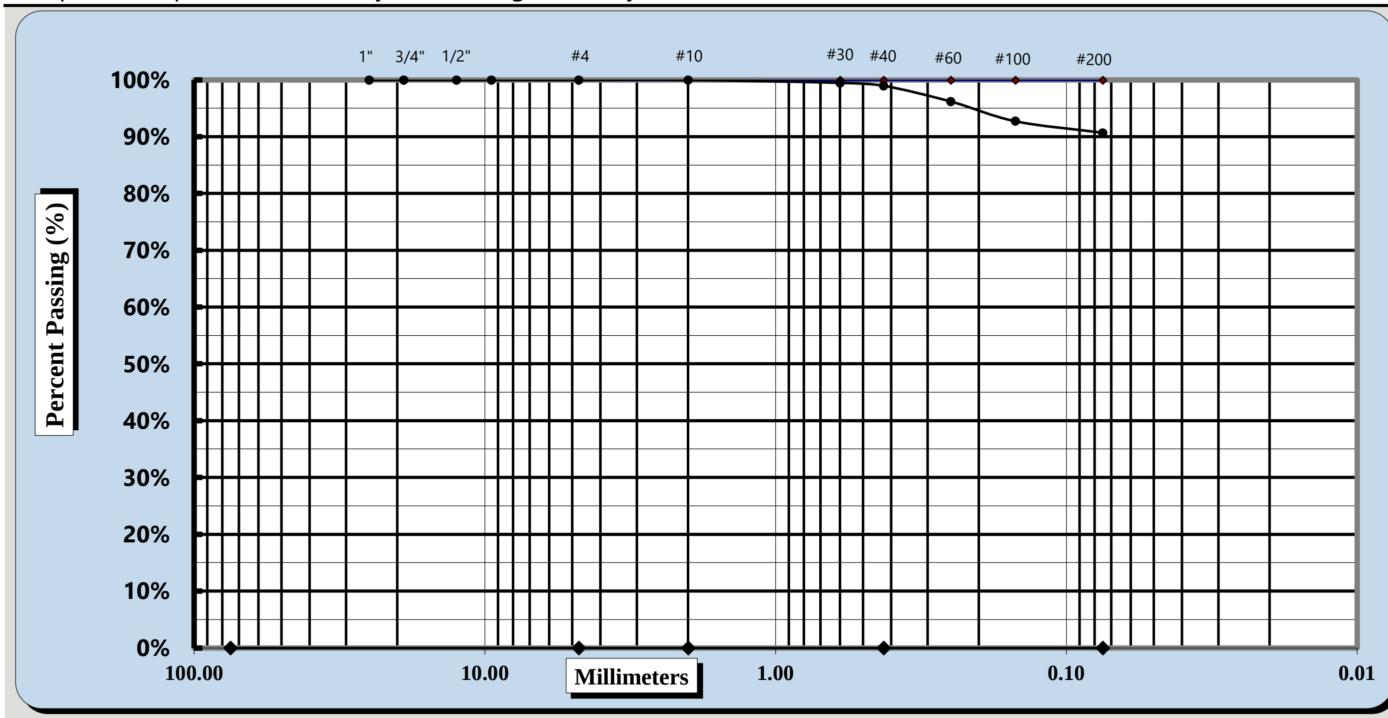
Form No TR-D6913-GR-01
Revision No. 1
Revision Date: 9/5/17

SIEVE ANALYSIS OF SOIL

Single sieve set ASTM D6913

S&ME, Inc. - Myrtle Beach: 1330 Highway 501 Business, Conway, SC 29526

Project #:	212687	Report Date:	8/18/2021
Project Name:	New Forestbrook Fire Station at Burcale Rd	Lab #:	414
Client Name:	Horry County Maintenance Dept.	Test Date:	8/18/2021
Client Address:	307 Smith St; Conway SC 29526	Date Sampled:	8/5/2021
Boring #:	P-1 to P-5	Sample #:	Bulk-1
Location:	Proposed Pavements	Depth:	1'-4'
Sample Description:	Tan, Gray, and Orange Fat Clay (CH)		



Cobbles	< 300 mm (12") and > 75 mm (3")	Fine Sand	< 0.425 mm and > 0.075 mm (#200)
Gravel	< 75 mm and > 4.75 mm (#4)	Silt	< 0.075 and > 0.005 mm
Coarse Sand	< 4.75 mm and >2.00 mm (#10)	Clay	< 0.005 mm
Medium Sand	< 2.00 mm and > 0.425 mm (#40)	Colloids	< 0.001 mm

Method: A Procedure for obtaining Specimen: Moist

Maximum Particle Size	#10	Coarse Sand	0%	Fine Sand	9%
Gravel	0%	Medium Sand	1%	Silt & Clay	91%
Liquid Limit	56	Plastic Limit	22	Plastic Index	34

Notes / Deviations / References:

Ron Forest, P.E.	RPF	Senior Reviewer	8/18/2021
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D4318-T89-90
Revision No. 1
Revision Date: 7/26/17

LIQUID LIMIT, PLASTIC LIMIT, & PLASTIC INDEX

ASTM D 4318 ☒ AASHTO T 89 ☐ AASHTO T 90 ☐

S&ME, Inc. - Myrtle Beach: 1330 Highway 501 Business, Conway, SC 29526

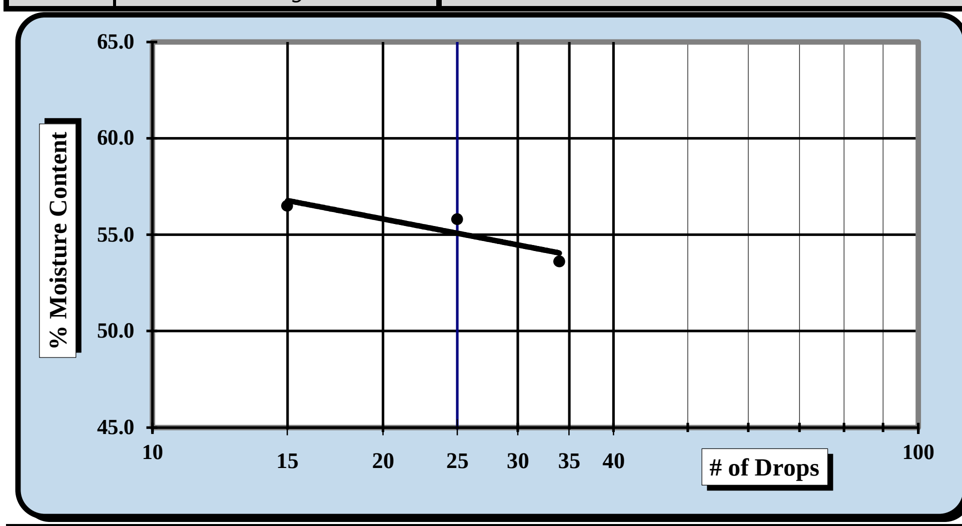
Project #:	212687	Report Date:	8/18/2021
Project Name:	New Forestbrook Fire Station at Burcale Rd	Test Date(s)	8/16/2021
Client Name:	Horry County Maintenance Dept.		
Client Address:	307 Smith St; Conway SC 29526		

Boring #:	P-1 to P-5	Sample #:	Bulk-1	Sample Date:	8/5/2021
Location:	Proposed Pavements	LAB #:	414	Depth:	1'-4'

Sample Description: Tan, Gray, and Orange Fat Clay (CH)

Type and Specification	*S&ME ID #*	*Cal Date:*	*Type and Specification*	*S&ME ID #*	*Cal Date:*
Balance (0.01 g)	00401	2/28/2021	Grooving tool	11368	9/1/2020
LL Apparatus	18801	9/1/2020			
Oven	17745	4/8/2021			

Pan #		Liquid Limit						Plastic Limit		
	Tare #:	36	48	92				21	17	
A	Tare Weight	14.63	14.58	14.59				14.57	14.59	
B	Wet Soil Weight + A	31.25	31.33	31.37				21.47	21.52	
C	Dry Soil Weight + A	25.45	25.33	25.31				20.23	20.27	
D	Water Weight (B-C)	5.80	6.00	6.06				1.24	1.25	
E	Dry Soil Weight (C-A)	10.82	10.75	10.72				5.66	5.68	
F	% Moisture (D/E)*100	53.6%	55.8%	56.5%				21.9%	22.0%	
N	# OF DROPS	34	25	15				*Moisture Contents determined by ASTM D 2216*		
LL	LL = **F** * FACTOR									
Ave.	*Average*							**22.0%**		



One Point Liquid Limit			
N	**Factor**	**N**	**Factor**
20	0.974	26	1.005
21	0.979	27	1.009
22	0.985	28	1.014
23	0.99	29	1.018
24	0.995	30	1.022
25	1.000		

NP, Non-Plastic	☐
Liquid Limit	**56**
Plastic Limit	**22**
Plastic Index	**34**
Group Symbol	**CH**

Multipoint Method ☑
One-point Method ☐

Wet Preparation ☐ Dry Preparation ☑ Air Dried ☑

Notes / Deviations / References:

ASTM D 4318: Liquid Limit, Plastic Limit, & Plastic Index of Soils

Ron Forest, P.E.	RPF	Senior Reviewer	8/18/2021
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D698-2
Revision No. : 1
Revision Date: 07/25/17

MOISTURE - DENSITY REPORT

Quality Assurance

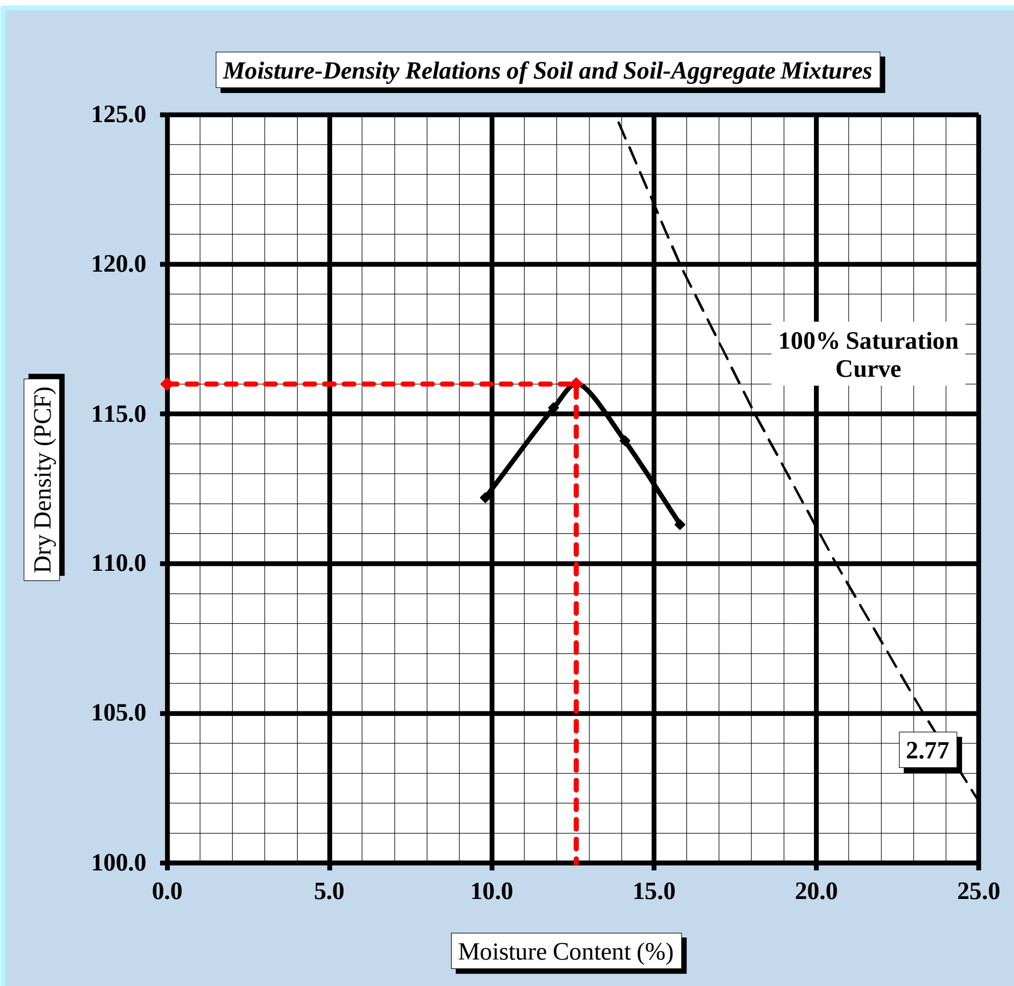
S&ME, Inc. - Myrtle Beach: 1330 Highway 501 Business, Conway, SC 29526

S&ME Project #:	212687	Report Date:	8/18/2021
Project Name:	New Forestbrook Fire Station at Burcale Rd	Test Date(s):	8/13/2021
Client Name:	Horry County Maintenance Dept.		
Client Address:	307 Smith St; Conway SC 29526		

Boring #:	P-1 to P-5	Sample #:	Bulk-1	Sample Date:	8/5/2021
Location:	Proposed Pavements	Lab #:	414	Depth:	1'-4'
Sample Description:	Tan, Gray, and Orange Fat Clay (CH)				

Maximum Dry Density 116.0 PCF. Optimum Moisture Content 12.6%

ASTM D 698 - - Method A



Soil Properties	
Natural Moisture Content	26.4%
Specific Gravity of Soil	--
Liquid Limit	56
Plastic Limit	22
Plastic Index	31
% Passing	
3/4"	100.0%
3/8"	100.0%
#4	100.0%
#10	100.0%
#40	99.0%
#60	96.2%
#200	90.7%
Oversize Fraction	
Bulk Gravity	--
% Moisture	--
% Oversize	--
MDD	--
Opt. MC	--

Moisture-Density Curve Displayed: Fine Fraction ☒ Corrected for Oversize Fraction (ASTM D 4718) ☐
Sieve Size used to separate the Oversize Fraction: #4 Sieve ☒ 3/8 inch Sieve ☐ 3/4 inch Sieve ☐
Mechanical Rammer ☐ Manual Rammer ☒ Moist Preparation ☐ Dry Preparation ☒

References / Comments / Deviations:
ASTM D 2216: Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D 698: Laboratory Compaction Characteristics of Soil Using Standard Effort

Ron Forest, P.E.	RPF	Senior Reviewer	8/18/2021
Technical Responsibility	*Signature*	*Position*	*Date*

Form No. TR-D1883-T193-3
Revision No. 2
Revision Date: 08/11/17

CBR (CALIFORNIA BEARING RATIO) OF LABORATORY COMPACTED SOIL

ASTM D 1883

S&ME, Inc. - Myrtle Beach: 1330 Highway 501 Business, Conway, SC 29526

Project #:	212687	Report Date:	8/18/2021
Project Name:	New Forestbrook Fire Station at Burcale Rd	Test Date(s)	8/13/2021
Client Name:	Horry County Maintenance Dept.		
Client Address:	307 Smith St; Conway SC 29526		

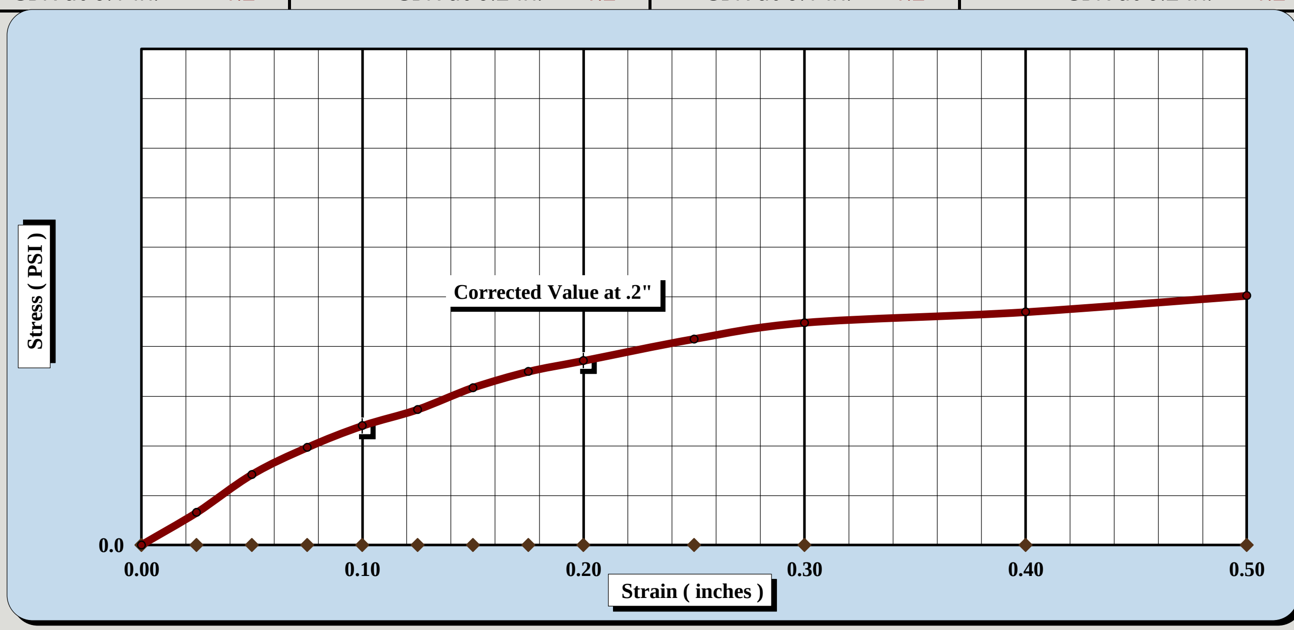
Boring #: P-1 to P-5	Sample #: Bulk-1	Sample Date: 8/5/2021	
Location: Proposed Pavements	LAB #: 414	Depth: 1'-4'	

Sample Description: Tan, Gray, and Orange Fat Clay (CH)

ASTM D 698 *Method A* Maximum Dry Density: 116.0 PCF Optimum Moisture Content: 12.6%

Compaction Test performed on grading complying with CBR spec. % Retained on the 3/4" sieve: 1.0%

Uncorrected CBR Values				Corrected CBR Values			
CBR at 0.1 in.	1.2	CBR at 0.2 in.	1.2	CBR at 0.1 in.	1.2	CBR at 0.2 in.	1.2



CBR Sample Preparation:

The entire gradation was used and compacted in a 6" CBR mold in accordance with ASTM D1883, Section 6.1.1

Before Soaking		*After Soaking*	
Compactive Effort (Blows per Layer)	25		
Initial Dry Density (PCF)	110.1	Final Dry Density (PCF)	108.9
Moisture Content of the Compacted Specimen	13.8%	Moisture Content (top 1" after soaking)	31.4%
Percent Compaction	94.9%	Percent Swell	1.2%

Soak Time:	96 hrs.	Surcharge Weight	20.0	Surcharge Wt. per sq. Ft.	102.0
Liquid Limit	56	Plastic Index	34	Apparent Relative Density	--

Notes/Deviations/References: Liquid Limit: ASTM D 4318, Specific Gravity: ASTM D 854, Classification: ASTM D 2487

Ron Forest, P.E.	RPF	Senior Reviewer	8/18/2021
Technical Responsibility	*Signature*	*Position*	*Date*

S&ME

March 9, 2000

Mr. Dean Penny
Kimley-Horn and Associates
Post Office Box 33068
Raleigh, NC 27636

Reference: Geotechnical Report
US Highway 501
Forestbrook Road Interchange
Frontage Road and Ramp Bridges
S&ME Project No. 1611-99-401

Dear Mr. Penny:

S&ME is pleased to present this report of our geotechnical exploration for the above referenced project. This report is a supplemental report to our geotechnical report submitted in February 2000 for the main alignment construction.

This report presents our recommendations for foundations and surface preparation of the frontage road and ramp bridges over Socastee Swamp.

The following paragraphs briefly summarize our recommendations for foundations:

- The bridges should be supported on 18 inch x 18 inch prestressed concrete piles bearing approximately 35 to 55 feet below ground surface. A working capacity of 60 tons is recommended using a factor of safety of 2.5.
- PDA testing of at least 1 pile at each end bent of each bridge is recommended.

The attached report more fully discusses the above recommendations.

We appreciate the opportunity of working with Kimley-Horn on this project and look forward to our continued working relationship. If you have any questions regarding the contents of this report or if we may be of assistance in any way, please do not hesitate to contact us.

Very truly yours,

SOUTH CAROLINA REGISTERED PROFESSIONAL ENGINEER No. 18448 JOHN WILLIAM BALE

SOUTH CAROLINA S & ME, INC. No. C00473 CERTIFICATE OF AUTHORIZATION

SOUTH CAROLINA REGISTERED PROFESSIONAL ENGINEER No. 14133 JOHN C. LESSLEY

John W. Bale, P.E.
Senior Project Manager

John C. Lessley, P.E.
Chief Engineer

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TABLES

FIGURES

APPENDIX A – FIGURES

APPENDIX B - FIELD DATA

APPENDIX C – LABORATORY DATA

APPENDIX D – LATERAL PSC DEFLECTION DIAGRAMS

1.0 INTRODUCTION

The area of study for this supplemental report consist of five bridges located near the intersection US Highway and Forestbrook Road. US 501 currently is a four lane divided highway that is very congested. Part of the improvement to US Highway 501 includes frontage roads and access ramps. These appurtenant features cross Socastee Swamp near the intersection of US 501 and Forestbrook Road. Each crossing will consist of a flat slab trestle bridge.

2.0 PROJECT INFORMATION

We were provided an unsealed full size drawing of the interchange encompassing the bridges from Kimley-Horn in December 1999 and an electronic copy on February 28, 2000.

Our understanding of the requirements of the geotechnical exploration is based on discussions between Messrs. Cecil Narron of Kimley-Horn Associates and John Lessley and John Bale of S&ME. The project lies within mainline stations 530+00 to 548+00 along US 501. Main aspects of the project consist of the following.

- *Bridge at Ramp B* (Stations 543+10 – 544+60) – this will be a new 2 lane flat slab bridge with 2 end abutments and 4 interior bents at equal 30 foot spacings crossing Socastee Swamp north of the west bound lanes of US 501. Abutment fills at this location will be approximately 10 feet.

- *Bridge on Dick Scobee Drive (Forestbrook Road)* (Stations 38+10 – 39+60) – this will be a new 4 lane flat slab bridge constructed in two phases over Socastee Swamp. An existing 2 lane bridge is currently located near station 38+60 to 39+60. We understand the existing bridge will be removed and replaced with a single 4 lane flat slab bridge constructed in two phases to allow continued use of the road. This bridge will consist of 5 equal spans of 30 feet with end bents and 4 interior bents. The existing bridge as will as approach fills will be raised approximately 3 feet.

- *Bridge at Ramp C* (Stations 543+30 – 544+80) – this will be a new 2 lane flat bridge consisting of end abutments and 4 interior bents at equal 30 foot spacings crossing Socastee Swamp south of the east bound lanes of US 501. Fill height of approach embankments at this bridge will be approximately 7 feet.

- *Bridge on Frontage Road 6F* (Stations 481+70 – 483+20) - this will be a new 4 lane flat slab bridge with end abutments and 4 interior bents at equal 30 foot spacings. Fill for approaches will be approximately 7 to 10 feet.

- *Bridge on Frontage Road 6E* (Stations 454+00 – 455+50) – this will be a new 4 lane flat slab bridge structure crossing Socastee Swamp. The bridge will consist of 5 equal spans of 30 feet with end bents and 4 interior bents. The approaches to this bridge will have a maximum of 9 to 10 feet of fill. This location was not accessible to our drill rig, but was explored in 1993 with 3 previous borings located near the bridge location.

3.0 GEOTECHNICAL EXPLORATION

The purpose of the geotechnical exploration was to identify the general subsurface stratification underlying retaining walls and to support foundation recommendations for design and construction of these structures. Our objectives may be summarized as follows:

- Provide a geotechnical profile and interpretation of conditions along the alignment.
- Provide recommendations for preparation or improvement of pavement support soils.
- Provide recommendations of applicable foundation types for the abutments and bents along the interchanges.

Limitation of Scope

In preparing our scope of work, we made the following assumptions based on conversations with Kimley-Horn and Associates:

- Exploration of proposed borrow sources for roadway fill will be performed by others and did not form a part of our services.
- Exploration for pavement design will be performed by others.
- The assessment of site environmental conditions or determination of the presence or absence of contamination in the soil, ground water, or air was beyond our scope. Identification of wetlands, endangered or threatened wildlife or plant species,

assessment of noise impacts or cultural resources of the site was also beyond our scope of services.

3.1 Previous Work

S&ME performed a number of borings in this area in 1993 and 1994 as a subcontractor to Parsons Brinckerhoff, the previous designer. S&ME was strictly a drilling subcontractor, with Parsons Brinckerhoff providing a supervisory engineer or geologist who examined and logged the recovered samples and prepared field boring logs for drafting. Parsons Brinkerhoff staff provided in-house geotechnical consultation to their team during this work, but no geotechnical report by them has been provided in preparation of this report.

We have no plan in our files indicating the locations where the 1993 borings were performed. Each log has a northerly and easterly coordinate typed on it, which for this exploration we assumed to be consistent with the coordinate system indicated on the plans provided to us by the SCDOT.

Using the coordinates indicated on the boring logs, we plotted each boring on the right of way plans provided to us by Kimley-Horn Associates. Of the 46 boring logs reviewed, 11 borings were determined to be near the proposed bridges and have been considered in our analysis. Due to accessibility of our drill rig, the bridge located on road 6E was analyzed exclusively using data from 3 previous borings.

In addition to these borings, S&ME conducted 50 borings in 1999 along the main alignment. Of these borings, 2 were considered to lie close enough to consider in our analysis of the bridges.

3.2 Field Exploration Program

In order to explore subsurface conditions at each bridge location we proposed to perform 10 additional soil test borings. Due to accessibility difficulties at the bridge along Frontage Road 6E, 8 new exploratory borings were drilled between February 16 and February 19, 2000. These borings were numbered B-201 through B-208 to distinguish them from earlier borings.

The field procedures employed by S&ME, Inc. generally follow the recommended practices set forth by ASTM D 420, "Recommended Practice for Investigating and Sampling Soils and Rocks for Engineering Purposes". This recommended practice lists recognized methods for determining soil and rock distribution and groundwater conditions.

3.2.1 Layout and Elevations

Boring locations are approximately indicated on the Boring Location Plans included as Figures A-2 and A-3. Boring locations were selected by S&ME, Inc. and approved by Kimley-Horn and Associates and the SCDOT prior to drilling.

- Borings were laid out by S&ME, Inc. personnel by taping distances and estimating right angles from existing roadways or from survey stakes established at the bridge location.
- Boring elevations were estimated based on topographic maps provided and USGS guadrangle maps

Due to the size of the site and the scale of the drawings, boring locations and elevations shown on the attached location plan and subsurface profiles should be considered approximate and not exact. Approximate easting and northing coordinates and approximate depths of borings are tabulated in Table 1.

Table 1 – Stationing, Coordinates, and Drilling Depths

Boring	Easting	Northing	Station	Mud Rotary Drill Depth (ft)	Remarks
New Borings					
B-201	2625250	272510	544+80	50	Ramp B
B-202	2625150	272620	543+30	50	Ramp B
B-203	2625570	273100	381+10	50	Dick Scobee Drive
B-204	2625530	273240	381+40	50	Dick Scobee Drive
B-205	2624970	272500	543+00	50	Ramp C
B-206	2625000	272490	543+30	50	Ramp C
B-207	2624910	271980	483+10	60	Frontage Road 6F
B-208	2624790	272000	481+80	60	Frontage Road 6F
Previous Borings					
B-117	2625188	272638	543+30	70	Ramp B
B-124	2625042	272369	544+80	70	Ramp C
B-43	2625107	272400	543+80	43	Ramp C
B-23	2625250	273826	454+20	63	Frontage Road 6E
PB-24	2625333	273768	455+00	60	Frontage Road 6E
PB-25	2625326	273848	454+90	78	Frontage Road 6E
PB-26	2625626	273199	39+20	33	Dick Scobee Drive
PB-27	2625656	273178	39+40	72	Dick Scobee Drive
PB-28	2625688	273171	39+70	59	Dick Scobee Drive
PB-29	2624800	271993	482+00	59	Frontage Road 6F
B-30	2624768	271948	481+90	67	Frontage Road 6F
B-32	2624873	271940	482+90	68	Frontage Road 6F

3.2.2 Sampling and Penetration Testing

The subsurface conditions encountered during drilling were reported on a field test boring record by the chief driller. The record contains information concerning the boring method, samples attempted and recovered, indications of the presence of various materials such as coarse gravel, cobbles, etc., and observations between samples such as very hard drilling lenses. Therefore, field boring records contain both factual and interpretive information. The field boring records are on file at our office.

- A rotary drilling process was used to advance each hole and a bentonite drilling fluid was circulated in the bore holes to stabilize the sides and flush the cuttings.

- Standard penetration tests (SPT) were conducted in the borings at intervals of between 2.5 and 5 feet.

- Representative portions of the split spoon samples were sealed in glass jars and returned to our office. We will store those samples not consumed in laboratory tests, available for inspection, until foundation construction is complete.

3.2.3 Groundwater

Groundwater levels were able to be measured at the time of drilling due to the relatively shallow depths to water. Stabilized groundwater levels were measured 24 hours after drilling.

Stabilized groundwater levels shown on the test boring records represent the conditions only at the time of our exploration and may fluctuate with seasonal variations in rainfall. Normally the highest seasonal groundwater levels occur in late winter and early spring and lowest levels in late summer and fall. The sediments penetrated by our borings at shallow depth will readily admit infiltration from the surface after rainfall. Thus we anticipate that local groundwater conditions may be heavily influenced by localized storms.

3.2.4 Boring Records and Profiles

The soil samples and the field boring records were reviewed by a geotechnical engineer in the laboratory. The engineer classified the samples in general accordance with ASTM 2488 – "Visual-Manual Procedure for Classification of Soil and Rock Samples", and prepared final boring records which are the basis for the recommendations in this report.

- Test Boring Records are attached in Appendix B, graphically showing soil descriptions and penetration resistances.

- Boring data have been incorporated into cross sections indicating the generalized soil profile along the alignment for the proposed bridges and retaining walls. These were developed by interpolation of subsurface data between adjacent borings and are included in Appendix A.

- While we feel that the interpolated profiles are reasonable, variations in the subsurface conditions between the borings can be expected to occur.

- The strata breaks on the Soil Test Boring Records and on the cross sections represent interpreted density or gradational variations in the soils and are intended as approximate boundaries, as transitions between strata are likely gradual.

3.3 Tests for Physical Properties of Soils

A laboratory testing program was conducted to provide data on representative engineering properties of the soils encountered in the borings for development of foundation recommendations. These tests were performed on representative split-spoon samples obtained in the borings. The testing was conducted in general accordance with applicable ASTM procedures. The results of individual tests are presented in Appendix C and summarized on the "Soil Data Summary" table.

Laboratory tests performed are broken down by test as follows:

Table 2 – Laboratory Test Summary

Laboratory Test	Quantity
Grain Size Analysis (Wash #200)	6
Natural Moisture Content	6
Atterberg Limits	5

3.3.1 Grain Size Analysis (Wash 200)

Grain size analysis was performed to determine the particle size distribution of selected samples.

- The percentage of soils coarser than a 0.074mm sieve opening was determined by washing the samples through a #200 sieve. Soil weight retained on the sieve was recorded.

3.3.2 Natural Moisture Content

The moisture content was determined for selected fine grained soil samples in general accordance with ASTM D 2216.

- A representative portion of each sample was weighed and then placed in an oven and dried at 110 degrees C for at least 15 to 16 hours.

- After removal from the oven, the soil was again weighed. The weight of the moisture lost during drying was thus determined.

- The moisture content of the sample was then calculated as the weight of moisture divided by the dry weight of soil, expressed in percent.

3.3.3 Atterberg Limits

Liquid limit and plastic limit tests aid in the classification and stratification of the soils and provide an indication of the soil behavior with moisture changes.

- The Liquid Limit is the moisture content at which the soil will flow as a heavy viscous liquid, as determined in accordance with ASTM D 423.

- The Plastic Limit is the moisture content at which the soil begins to lose its plasticity, as determined in accordance with ASTM D 424.

- The Plasticity Index is bracketed by the Liquid Limit (LL) and the Plastic Limit (PL) and indicates the moisture content range of plastic soil behavior.

- The Liquidity Index is determined by comparing the natural moisture content of the soils to the Plastic Limit and Plastic Index, and is a measure of the state of soil moisture relative to the range of plastic soil behavior.

3.3.4 Previous Laboratory Testing:

As indicated, S&ME has data available from previous laboratory testing of samples obtained from borings performed near the proposed bridge locations. In an effort to reduce redundant laboratory testing, data from this testing was correlated to subsurface soils encountered at the bridges based on visual classifications, SPT-values and soil classification testing performed during this study and previous studies.

4.0 SITE CONDITIONS

The following sections describe the surface conditions in the vicinity of the proposed alignment and include a discussion of general topographical features and regional geology.

The physiography for this site was described in detail in our February, 2000 report.

4.2 Geologic Site Features

Geologic site features were described in detail in our February, 2000 report.

4.3 Geology

The general geology for the site was described in detail in our February, 2000 report.

4.4 Soil Stratigraphy

The generalized soil strata have been divided on the basis of depth and visual appearance of the recovered samples, consistency as measured by SPT methods, laboratory classification or strength tests, and indicated origin based on geologic or soils mapping. The purpose was to segregate soils into layers with approximately similar physical properties in terms of foundation performance. The classification tests have included water content, grain size distribution, and plasiticity tests.

Subsurface stratification is indicated on the attached profiles in Figures A-4 through A-8.

4.4.1 Pleistocene to Recent Deposits

- **Recent Fill (F)** - Soils present within the existing roadway fills along Dick Scobee Drive were not widely explored due to the need to offset borings well clear of traffic lanes, however one boring was extended through the roadway shoulder. The boring encountered apparent fill sands to a depth of 6 feet. SPT N-values within the fill sands penetrated range from 8 to 19 blows per foot, representing medium dense soil compaction. CPT point stress values within fill zones tested in previous borings range up to 75 tsf. Boring data imply generally good compaction of the sands during placement where they penetrated fills directly below or adjacent to the existing roadway, but compaction at some distance from the roadway appears somewhat indifferent, with SPT N-values less than 10 being common in previous borings.

- **Interbedded Sands and Silts and Clays (C-1) (C-2)** - silts, clays and sands are generally bedded together in varying thicknesses with beds of shells or shell hash. Recovered samples are generally, tan to brown, gray or green-gray in color. Consistency is highly variable, with SPT values obtained in the borings near the bridges ranging from 0 to as high as 50 blows for 5 inches of penetration. Soil parameters for these layers are discussed in more detail later in the report. These layers will be referred to as the stiff clay (C-1) and soft clay (C-2) layers and are labeled as such on Figures A-4 through a-8.

- **Very Soft Silts and Clays (O)** - river or marsh deposits in low-lying marshlands or waterways. These soils were encountered at relatively shallow depths. Where encountered, these soils ranged in thickness from 3 to 9 feet and lie at a depth of roughly 5 to 20 feet below the surface.

  Sampled materials consist of dark gray or black, somewhat organic plastic silts or clays with little apparent strength. Vegetable matter or plant debris were present in some samples. Most samples were saturated, containing free water, and were heavily remolded. Samples also exhibited a very pronounced organic odor. SPT N-values obtained in these soils range from weight-of-hammer to 4 blows with an average of approximately 3 blows.

  These soils appear to correspond to the peaty clays described in SCGS borings in the locale and which are termed the "Horry Clay" by the SCGS geologists. Borings and soundings indicate a relatively continuous layer of very soft clays and peat at the Frontage Road 6F Bridge, Frontage Road 6E and Dick Scobee Drive and discontinuous zones at Ramp B and Ramp C.

4.4.2 Pee Dee Formation

- The Pee Dee formation consists of overconsolidated marine silts and clays and sands laid down approximately 30 million years ago. Samples were typically firm or dense silt, clay or silty sand interbedded with thin layers of very hard limestone or sand. Standard penetration resistance ranged from 6 to 100+ blows per foot.

4.5 Groundwater

Groundwater was measured at depths of less than 1 foot to 2 feet during drilling. Groundwater elevations recorded in the borings are near the seasonal high groundwater levels estimated for representative site soils identified in U.S.D.A. soil maps. Fluctuations in groundwater levels may occur with rainfall variation, construction, surface runoff, and other factors.

Soils similar to those encountered at the site often contain one or more beds of fully saturated sands "perched" on interlayered seams of relatively impervious fine-grained, very dense or very hard soils. During or following periods of higher than average precipitation this "perched" water may emerge as springs along slopes or in graded cuts.

5.0 ROADWAY EMBANKMENTS AND RETAINING WALLS

Of the 5 bridge locations we understand that Ramps B and C will include construction of approach embankments which will incorporate MSE walls for support of the embankments. Performance of these embankments and walls were addressed in detail in our previous report.

The bridge located on Dick Scobee Drive (Forestbrook Road) will require additional fill adjacent to the existing bridge to allow for widening the bridge. Fill depths are not expected to exceed 10 to 12 feet and will be sloped into the adjacent swamp.

Fill for bridges along Frontage Roads 6E and 6F are also expected to be nominal, and less than 10 feet. Borings were not performed along the whole alignment of these roads to provide detailed analysis of settlement caused by fill placement. We understand the embankment fill will be sloped into the adjacent swamp.

5.1 Embankment Material

The source of embankment soils for the project is not presently known. Therefore the properties of the borrow soils have been assumed in our stability analysis. The predominant type of fill in the Myrtle Beach area is sand with low percentages of silt or clay size particles and therefore low cohesion. Stability analyses were performed assuming compacted embankment material with zero cohesion and an angle of internal friction of 34 degrees, and a mass unit weight of 110 pcf. Due to the wide range in soil types and properties which may be conceivably used on this project, we recommend that the embankment fills to be used on this project be located and their engineering properties investigated prior to construction.

5.2 Static Settlements under New Embankment Fills

We estimated settlements of the bridge approach fills due to consolidation of the bearing materials for each of the bridges. As indicated settlements associated with embankment fills for retaining walls on Ramps B and C are addressed in our previous report. Since fill heights are typically low, for settlement analysis we considered instantaneous placement of the fill to full height. Settlements for each subsurface layer were estimated by multiplying the increase in overburden stress by the layer thickness and then dividing by the compression modulus for the material, as determined from consolidation testing, field dilatometer, SPT, or CPT tests from previous studies, as well as our general experience with similar soils in the Myrtle Beach area.

Distribution of stresses below the stabilized soil mass was estimated by assuming the bearing soils to constitute a semi-infinite elastic continuum using the Westergaard stress distribution. The fill mass was assumed to represent a perfectly flexible foundation 30 to 80 feet in width and of infinite length. Settlements were then computed at the centerline of the roadway at the approaches to the bridges assuming fill depths as outlined. Estimated settlements for each bridge are tabulated below.

Table 3 – Estimated Static Settlements of Bridge Approaches

Location	Fill Depth	Settlement @ Centerline West Approach inches	Settlement @ Centerline East Approach inches
Frontage Road 6E	10	2.7	2.2
Frontage Road 6F	10	8.4	7.3
Dick Scobee Drive	10	1.8	2.6
Ramp B	10	3.5	1.7
Ramp C	10	2.1	4.0

As indicated by Table 3 settlements at each bridge approach are not excessive with the exception of the bridge on Frontage Road 6F. This location encountered approximately 2 to 3 feet of highly organic soft soils at the surface with an additional 2 layers of soft clays encountered with depth. If the upper zone is removed and replaced with compacted fill, settlements will be reduced to values on the order of 4.5 to 5 inches.

5.3 Settlement due to Earthquake Forces

We considered potential for liquefaction of sandy soils using the empirical Seed method, which characterized the stress state of the soil by a cyclic stress ratio. Liquefaction of fine grained soils at the site is generally not a concern based on the quantity of clay sized particles.

Two locations are considered susceptible to volumetric strain due to liquefaction. The bridge on Frontage Road 6E from boring PB-25 to PB-24, near station 455+00 to 456+00 encountered 2 to 7 feet of potentially liquefiable clean sands. Boring B-201 along Ramp B encountered a zone of approximately 5 feet of susceptible soils. We estimate volumetric compression at these two areas to be 3 inches and 1.5 inches respectively. Borings located along Dick Scobee Drive appear to also have isolated pockets of very loose clean sands, however these sands tend to be relatively thin or at deeper depths. Volumetric compression in this area is estimated to be less than 1 inch. Volumetric compression in the remaining areas is also estimated to be less than 1 inch.

5.4 Secondary Settlement

We estimated secondary consolidation assuming a secondary consolidation coefficient of 0.01, to be approximately ½ inch the first year after construction. Total secondary consolidation is estimated to be on the order of 1.5 inches or less.

5.5 Global Stability

Global stability for bridge approaches at Ramps B and C were presented in our previous report. Static stability of embankments at Dick Scobee Drive and Frontage Roads 6E and 6F will exceed a factor of safety of 1.3 assuming a minimum slope of 2 horizontal to 1 vertical for the fill depths outlined in this report based on the soils encountered. The only areas subjected to potential liquefaction and resulting loss of support soils are at the Ramp B and Frontage Road 6E bridges where very loose clean sands up to 7 feet in thickness were encountered. However, these areas appear to be isolated pockets and therefore instability due to soil liquefaction does not appear to be indicated.

5.6 Surface Preparation

The following recommendations are given for surface preparation in fill areas:

- Strip organics and organic laden material from within the footprint of the roadway sections. Waste organics outside road areas.

- The majority of fine grained soils encountered on-site are AASHTO A-6 or A-7 classified soils with liquid limits exceeding 40 percent. These soils should not be placed as fill within 18 inches of the subgrade. Where these soils lie at or new proposed subgrade elevation and have become wet or disturbed by previous tree cutting operations they should be undercut to firm material.

- All imported fill shall be free of deleterious materials and meet the SCDOT standard specifications for its intended use. It is anticipated most new fill will be imported.

- Where fill depths are greater than 6 feet, a thickened initial lift of 18 to 24 inches of new fill may be placed on the existing ground surface and the surface densified after removing large organics and topsoil. Trees may be cut off flush with the surface and the root ball left in place.

- Where fill depths will be less than 6 feet, proofroll the exposed soils after stripping or undercutting by making repeated passes with a heavily loaded dump truck or pan. Roll only during dry weather to avoid degrading an acceptable subgrade. Areas of rutting or pumping soils may require selective undercutting or further stabilization prior to fill placement. The exposed soil may be stabilized by dumping 12 inches of crushed stone on the surface and working it in with tracked equipment, prior to placing the initial lift of backfill.

- Proofrolling should be observed by the geotechnical engineer.

5.7 Fill Placement and Compaction

The organic laden soils and majority of clay soils encountered in our borings near the ground surface are not suitable for use as compacted fill. We give the following recommendations for placement of fill material:

- Before beginning to place fill, sample and test each proposed fill material to determine maximum dry density, optimum moisture content and natural moisture content.

- Place new fill in maximum 8 inch loose lifts and compact to at least 95 percent of maximum dry density (ASTM D-698 Standard Proctor). Fill should be moisture conditioned to within plus or minus 3 percent of optimum moisture as determined by ASTM D - 698

- Fill placement should be witnessed by an experienced soils technician working under the guidance of the geotechnical engineer. Conduct at least one field density test every 4,000 cubic yards in mass grading and for each 50 cubic feet of fill placed in confined areas such as wall backfill or trenches. We recommend controlling moisture to within three percentage points of optimum moisture.

6.0 BRIDGE FOUNDATION RECOMMENDATIONS

Considering the anticipated loads and the subsurface profile encountered by our borings, prestressed, precast concrete piles appear feasible for support of the bridges. Resulting pile lengths are relativley short. The following paragraphs provide analysis and recommendations for PSC piles for support of bridges.

6.1 Prestressed Concrete Piles

Prestressed piles are those fabricated using high ultimate strength reinforcement subjected to a presetressing force in such manner as to place the concrete in compression.

1. Precast piles should be constructed of Class A concrete, cast in the horizontal position. Reinforcing strands, bars, or spirals should be of new steel without splices and be secured in the form in a manner to provide the required embedment.

2. Piles may be transported and driven after the concrete is at least 6 days old and is expected to achieve a 28-day compressive strength of 5000 psi, or after 3 days if the concrete has attained a compressive strength of 5000 psi. Concrete piles shall be designed for bending stresses associated with lifting, and pick-up points on the piles should be appropriately marked.

3. Precast concrete piles are typically driven to the minimum criteria established for the required bearing capacity and built up to grade if necessary. Build-ups, when necessary due to overdriving or damage, are constructed of the same quality concrete as the pile. If no further driving is required, build-ups may be of Class A concrete.

4. Criteria outlined in the "Manual on Design and Construction of Driven Pile Foundations" recommend allowable maximum compressive driving stresses not exceed 0.85 times the compressive strength of the pile concrete minus the effective prestress. Tensile driving stresses are limited to three times the square root of the compressive strength plus the effective prestress. Considering a concrete compressive strength of 5000 psi and a residual prestress of 750 psi, it should be possible to install concrete piles to a ultimate capacity of

at least 150 tons while keeping driving stresses within the specified limits with use of a properly matched hammer and pile cushion.

5. Our analyses consider conditions similar to those given at representative borings at each bridge location. Bearing depths will likely vary across the site but will in general be between 35 to 55 feet. Cast pile lengths should be determined based on the results of index piles at representative locations within the site and static or dynamic load tests.

6.2 Driven Pile Vertical Capacity

We estimated static capacity using the method outlined in the NAVFACS Design Manual DM 7.2 (1984). Static computations were compared to capacities obtained using the computer code SPT91, developed by the University of Florida and the Florida Dept. of Transportation. This approach applies uncorrected standard penetration resistance to compute pile capacity.

The following figures show recommended ultimate capacities versus depth for 18 inch by 18 inch precast concrete piles.

Figure 1
Ultimate Capacity 18 inch by 18 inch
Prestressed Concrete Piles

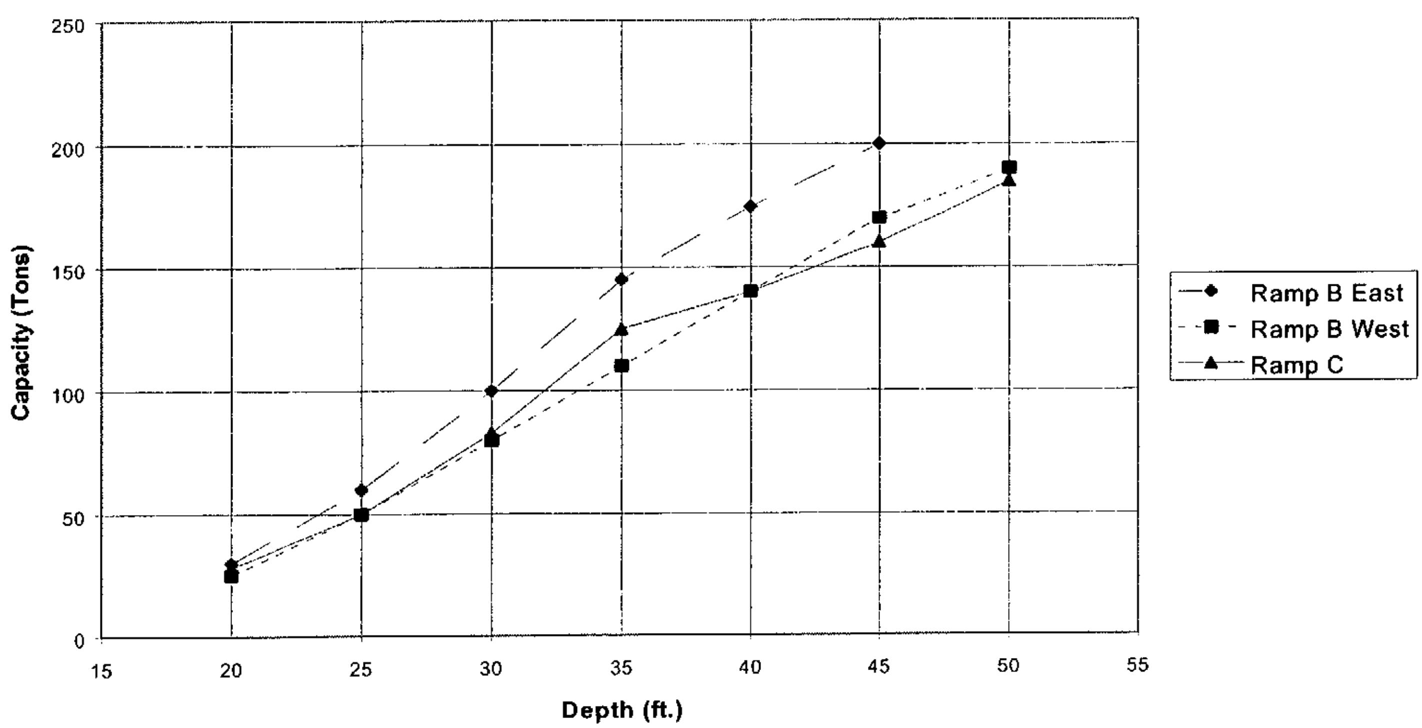
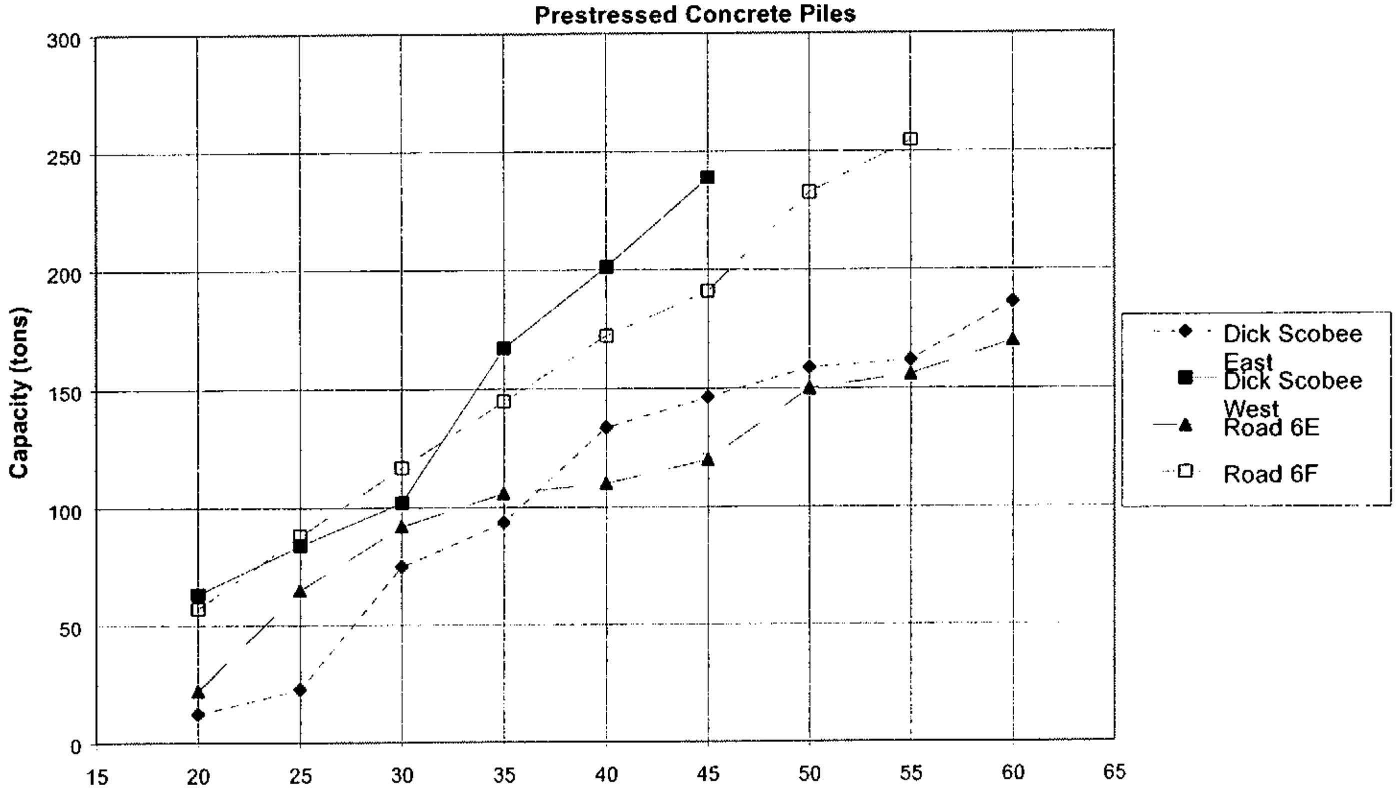


Figure 2
Ultimate Capacity 18 inch by 18 inch
Prestressed Concrete Piles



A factor of safety of at least 3 should be used in design where pile capacity is verified entirely on the basis of empirical driving formulas. If dynamic or static load tests are conducted on the piles the factor of safety may be reduced to 2.5 or 2.0 for dynamic or static load testing, respectively. Due to the limited depth of fill and the relatively limited long term settlement of the fill and bearing soils we do not anticipate downdrag to have a significant impact on pile capacity. We anticipate settlements will be limited to elastic compression of the piles under working loads, or approximately ¼ inch.

Due to a scour depth of up to 10 feet, 10 tons of capacity should be deducted from those shown on the figures for interior bents subjected to scour. Based on our analysis we provide the following estimated bearing depths below existing ground surface assuming a working load of 60 tons and a factor of safety of 2.5.

Table 4 – Estimated Pile Tip Bearing Depths

Location		Estimated Bearing Depth (ft)
Ramp B	West	40 – 45
Ramp B	East	35 – 40
Ramp C		45 – 50
Dick Scobee	West	35 – 40
Dick Scobee	East	50 – 55
Road 6E		55 – 60
Road 6F		35 – 40

Based on our analysis and assumptions, 18 inch x 18 inch square prestressed concrete piles will likely bear at approximately 35 to 50 feet below ground surface. We understand the closest spacing for 18 inch x 18 inch approximately 7.5 feet. This spacing is not close enough to require group effect reductions in capacity.

6.3 Driven Pile Lateral Capacity

Lateral forces will be applied to the piles in the transverse direction by wind loads (static) and earthquake forces (dynamic) and in the longitudinal direction by braking. Pile lateral behavior was modeled for typical 18 inch by 18 inch square precast piles using LPile computer code developed by Professor Lyman Reese. Deflections of piles due to lateral loading was determined at the pile head for a lateral load applied to the pile head under a 120,000 pound axial load. For the analysis we assumed the pile head would terminate 5 feet below the bridge deck to accommodate pile caps and bridge decking. We also assumed scour to elevations 4.0. 1.9, 3.0, 4.7 and 5.4 for bridges at Ramp B, Dick Scobee, Ramp C, Frontage Road 6F and Frontage Road 6E, respectively. For piles at the abutment we assumed soil support to the top of the pile, stopping 5 feet below the bridge deck. Piles were modeled as free headed at the bridge deck, using soil spring constants estimated from composition and density in the upper 20 feet of the borings. Table 5 summarizes soil subgrade reactions used in our analysis. The following figures show anticipated lateral deflections under a cyclic loading and a range of lateral loads considering the conditions described above.

Table 4 Soil Subgrade Reactions

Soil Type	Static Subgrade Coeff. (Ks) Pounds/in^3	Dynamic Subgrade Coeff. (Kd) Pounds/in^3
New Embankment fill	100	100
Upper Clays	30	30
Intermediate Sands	50	50
Lower Clays and Silts	30	30
PeeDee Formation	2000	800

Figure 3
Pile Head Deflection Free Head Pile With Scour

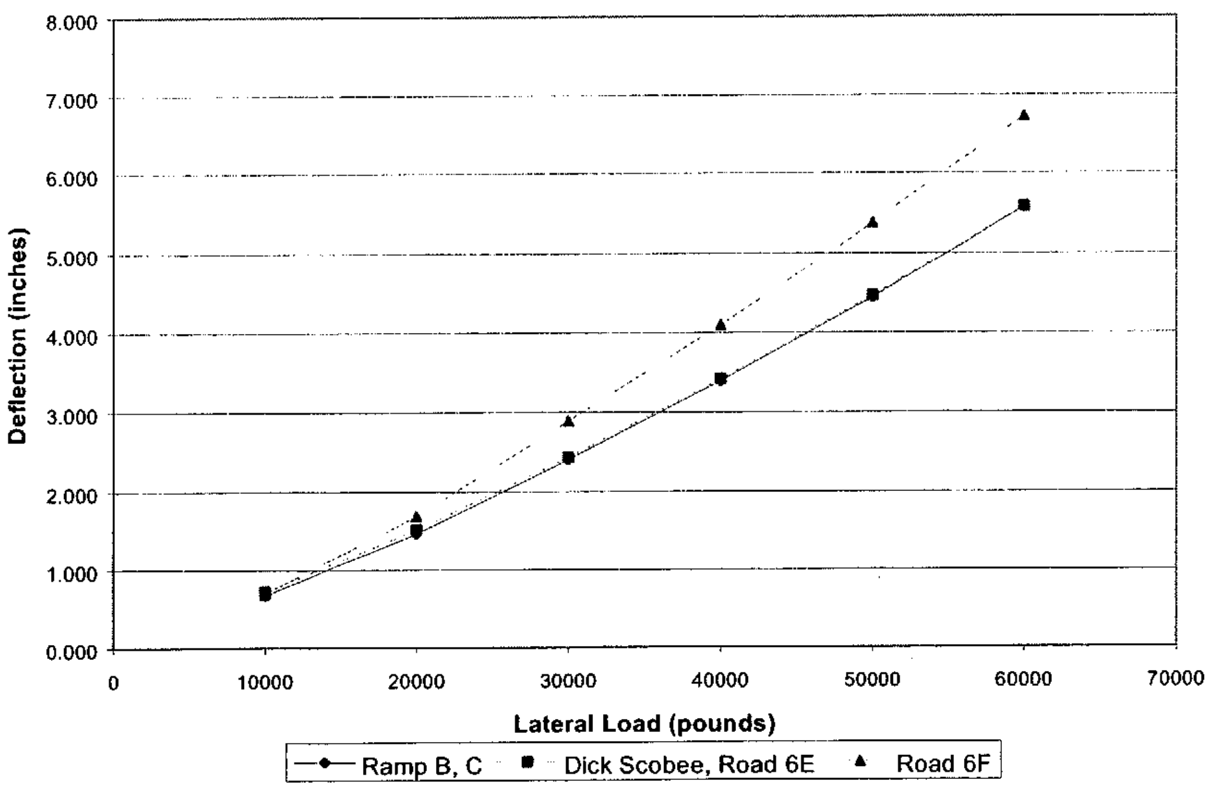
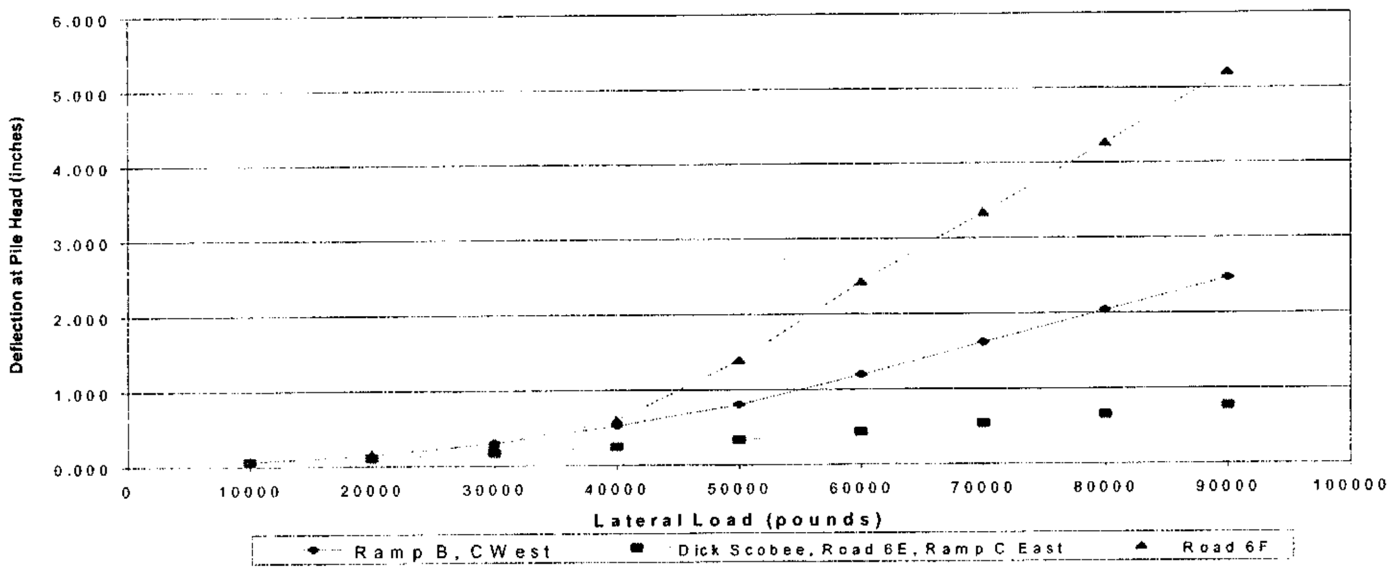


Figure 4
Pile Head Deflection Free Head Pile WithOut Scour



Coefficients of horizontal subgrade reaction for soils penetrated by the piles are summarized in Table 5.

Based on the above pile deflection assumptions the depth to fixity will be approximately 30 to 35 feet below the pile head for 18 inch by 18 inch piles as shown in Figures presented in appendix D.

Lateral resistance to bridge movements under earthquake loads offered by the abutments may be modeled by a coefficient of lateral subgrade reaction of 200 pci.

6.4 Pile Hammer Selection and Driving Criteria

Compatibility of the pile driving equipment, the soil conditions and the pile type being driven are all essential elements to achieving the required penetration and capacity. Criteria for terminating driving should take into account the hammer used, pile weight, allowable pile stresses, and required capacity.

1. Based on a typical 18 inch by 18 inch precast concrete piles 40 to 55 feet long, the pile driving hammer should be rated by the manufacturer from 50 to 60 ft-kips. of energy with a minimum hammer weight of 25 to 28 kips. Pile hammer type, hammer base, and cushion material should be provided to the geotechnical engineer for review prior to driving.

2. For soil bearing piles, the final rate of penetration should be estimated for the selected hammer type and energy using the latest version of the GRLWEAP computer code by Goble Rausche Likens and Associates, or equivalent. Input parameters for use in the analysis are tabulated below:

	18 inch x 18 inch
Skin Quake	0.1 inch
Toe Quake	0.1 inch
Skin Damping	0.05
Toe Damping	0.15 s/ft
Skin Friction Dist.	8
Percent Skin Frict.	40 %

3. Hammer selection should be based on piles driven to a final penetration of 2 to 3 blows per inch at the desired capacity.

4. Regardless of driving resistance, no pile should be embedded less than 5 feet into the Pee Dee Material.

5. Leads are required on the hammer and should be fixed at the top and adjustable on the bottom. Piles should be installed as plumb as possible, or at the designated batter, with the pile, hammer and leads in alignment to prevent impact bowing.

6. Hard driving at depth will require use of steel tips or shoes on all concrete piles.

6.5 Index Piles

We recommend installation of index piles prior to production driving to verify appropriate cast lengths for concrete piles. We recommend index piles be installed at each end of each bridge. Index pile installation should be used as a guide to predrilling requirements, if required, and for the purpose of assisting in the estimating of pile casting lengths and to confirm that the contractor's equipment and installation methods and procedures are acceptable.

6.6 Pile Driving Analyzer Testing

Pile capacities may be verified using dynamic methods in lieu of static tests, but will require use of a factor of safety of 2.5 in computation of working capacity relative to ultimate capacity. The Pile Driving Analyzer instrumentation commonly used in dynamic tests, typically consists of attaching two accelerometers and two transducers to near the top of a pile during driving. For each hammer blow, the PDA converts the strain and acceleration signals into force and velocity vs. time waves and records this data for analysis. Several quantities, including force, displacement, energy, momentum, resistance, time, velocity, and wave force, are then computed. A capacity estimate is then made using the Case Method or similar wave propagation theory equation, using assumed damping factors characteristic for these soils.

1. A minimum of 2 index piles at each bridge should be monitored during initial driving using a Pile Driving Analyzer Model GCXS or equivalent.

2. The PDA cannot determine gain or loss in capacity with time unless a pile restrikes test is performed at some later time after the pile has been driven. We recommend at least 1 to 2 piles be restruck at least 3 days after initial driving to guage time dependent set-up.

3. PDA capacity predictions are highly dependent on selection of the proper damping parameters for the soils at the site. The best way to select the Case damping factor is to correlate PDA output with static load tests or perform one iterative analysis using CAPWaPC or similar compute codes. We recommend at least PDA test be analyzed using the CAPWAPC code to verify damping parameters assumed in PDA test and more closely estimate pile compressive capacity.

6.4.7 Production Pile Driving

1. All production piles should be installed using the same equipment and to approximately the same depth and hammer blow count as applicable test piles.

2. Predrilling will be required to penetrate near surface layers of fat clay. We recommend predrilling be performed using a minimum 16-inch diameter auger to at least 20 feet.

3. Production pile installation should be observed by an experience inspector or engineering technician working under the guidance and supervision of the geotechnical engineer.

4. Records of all piles driven should be prepared on an appropriate driving log. This should include
    - size, length, head cut-off elevation, toe elevation, location;
    - sequence of driving;
    - number of blows per ft. or inch;
    - pre-augering, diameter and depth;
    - jetting, time and depth;
    - pile hammer and cushion arrangement; and,
    - movement of adjacent piles.

6.8 Construction Plans and Specifications Preparation and Review

It is recommended that this office be provided the opportunity to make a general review of the foundation and earthwork plans and specifications prepared from the recommendations presented in this report. We would than suggest any modifications so that our recommendations are properly interpreted and implemented. S&ME can also help prepare specifications for foundation installation and testing, if desired. Our report has been written in a guideline recommendation format and is not appropriate for use as a specification without in-part being reworded into a specification-type format. It is recommended that this report not be made a part of the contract documents; however, it should be made available to prospective contractors for information purposes.

7.0 LIMITATIONS OF REPORT

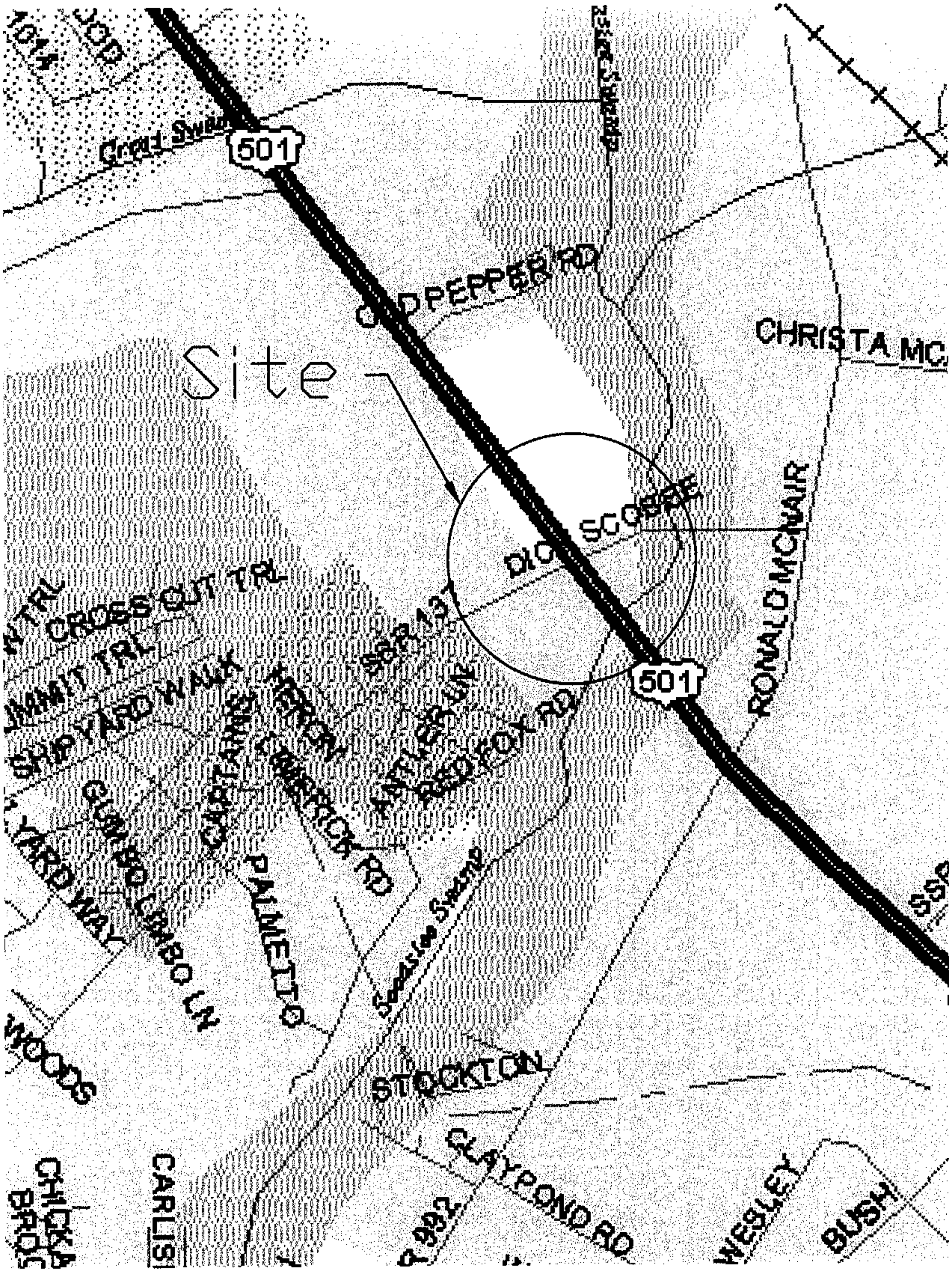
This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations in this report are based on the applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, express or implied, is made.

The analyses and recommendations submitted herein are based, in part, upon the data obtained from the subsurface exploration. The nature and extent of variations between the borings will not become evident until construction. If variations appear evident, then we will re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the building are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and conclusions modified or verified in writing by the submitting engineers.

The assessment of site environmental conditions or determination of the presence or absence of contamination in the soil, ground water, or air was beyond our scope. Identification of wetlands, endangered or threatened wildlife or plant species, assessment of noise impacts or cultural resources of the site was also beyond our scope of services.

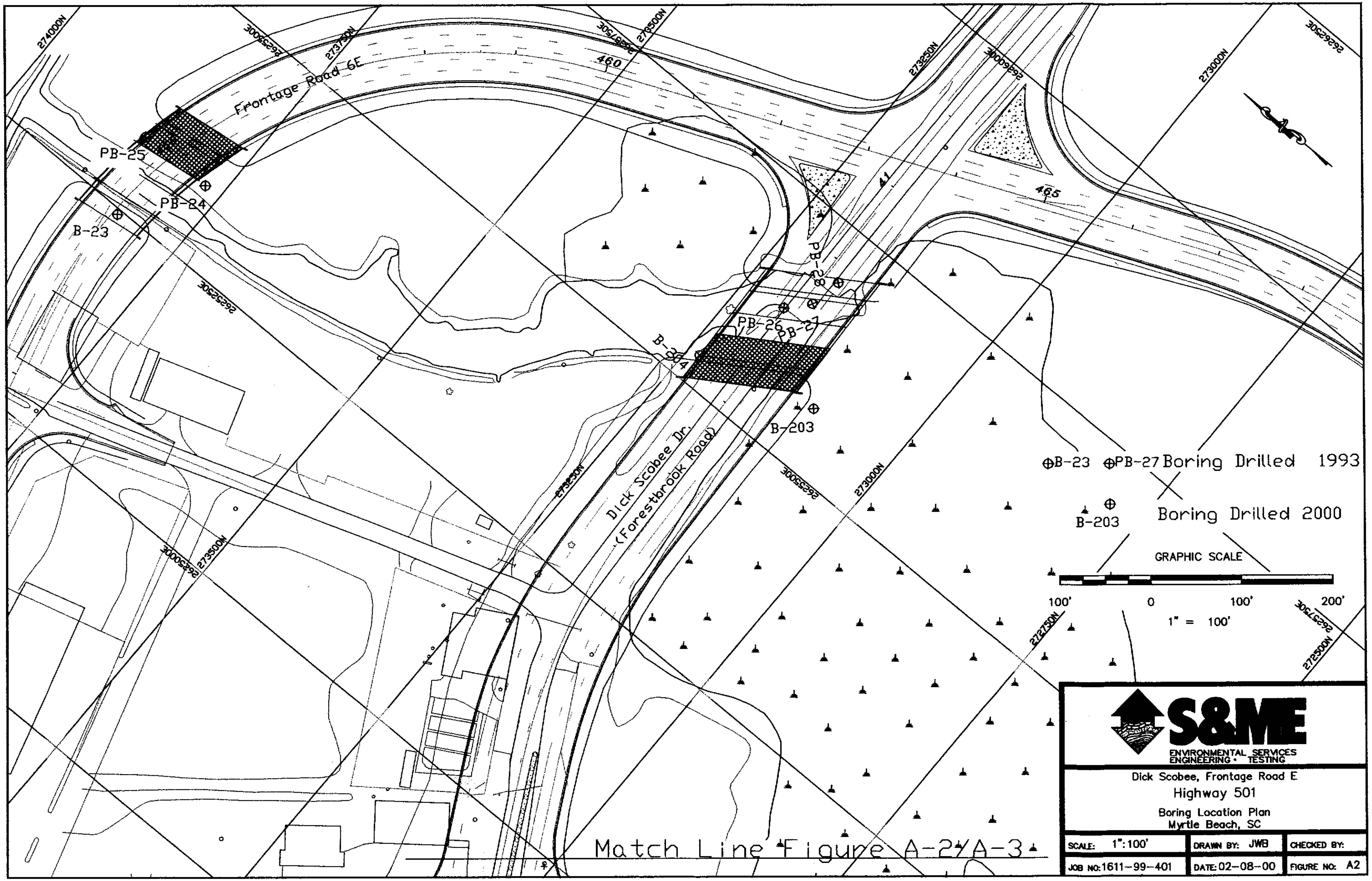
We recommend that S&ME be provided the opportunity to review the final design plans and specifications in order that earthwork and foundation recommendations are properly interpreted and implemented.

Appendix A
Figures



SCALE: NTS	S&ME ENVIRONMENTAL SERVICES ENGINEERING · TESTING	Site Vicinity Plan Highway 501 STREET ADDRESS Myrtle Beach, South Carolina	FIGURE NO:	A1
CHECKED BY:				
DRAWN BY: JWB				
DATE: 03-08-00		JOB NO: 1611-99-401		

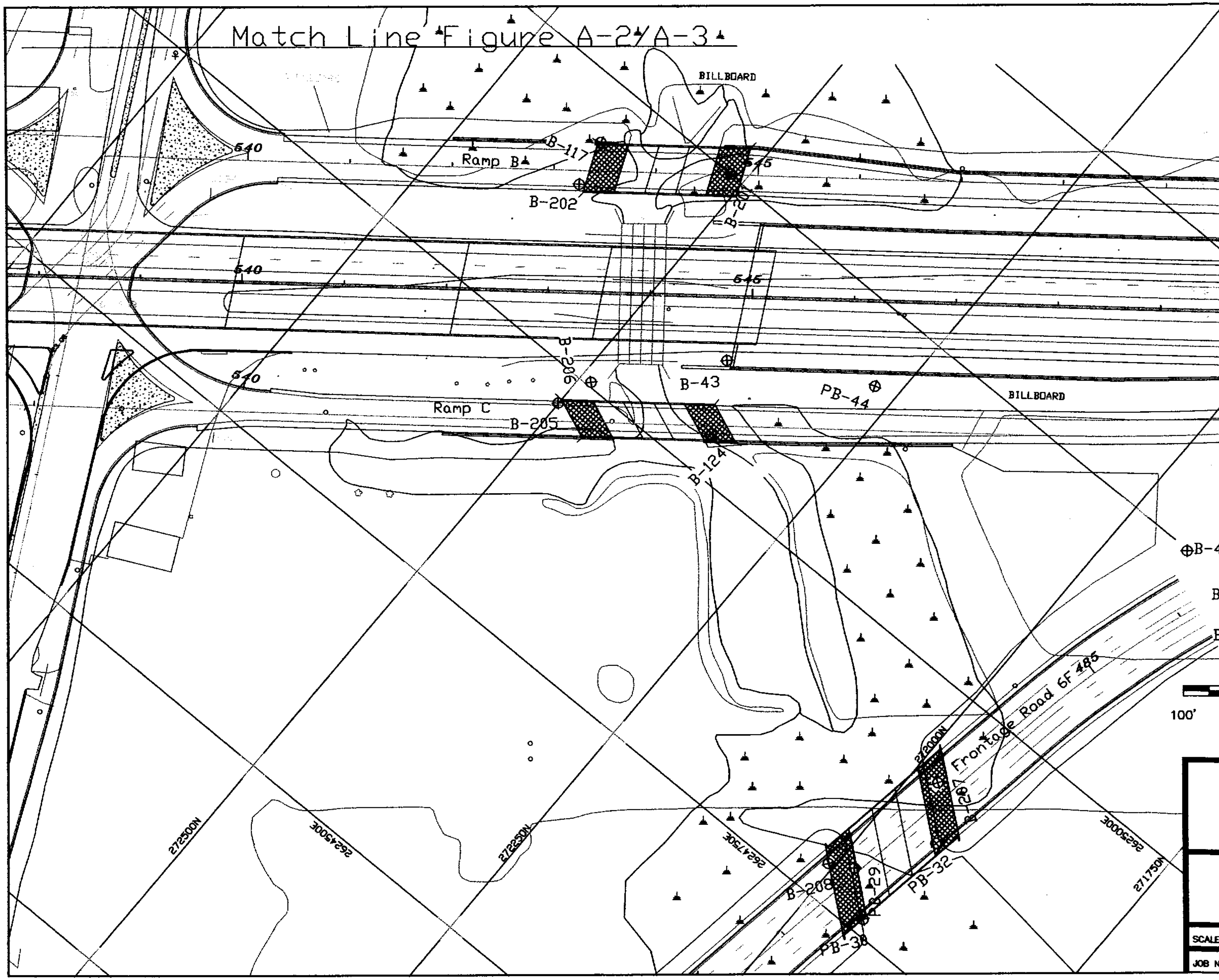
s:\client\401\bridges\drawings\figa1.dwg



S&ME
ENVIRONMENTAL SERVICES
ENGINEERING • TESTING

Dick Scobee, Frontage Road E
Highway 501
Boring Location Plan
Myrtle Beach, SC

SCALE: 1":100'	DRAWN BY: JWB	CHECKED BY:
JOB NO: 1611-99-401	DATE: 02-08-00	FIGURE NO: A2



⊕B-43 ⊕PB-30 Boring Drilled 1993

⊕ B-117 Boring Drilled 1999

⊕ B-208 Boring Drilled 2000

GRAPHIC SCALE

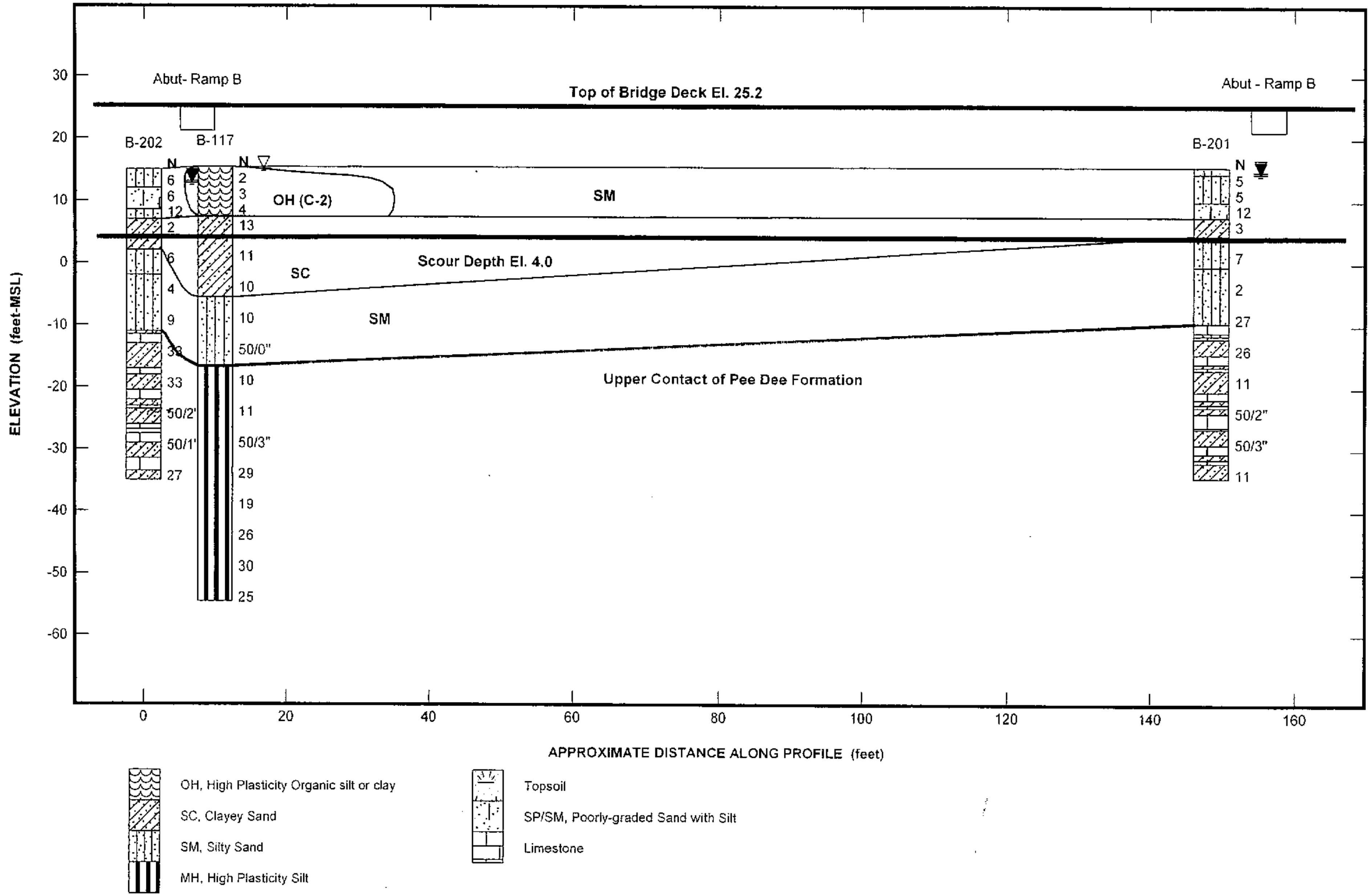
100' 0 100' 200'

1" = 100'

S&ME ENVIRONMENTAL SERVICES ENGINEERING • TESTING

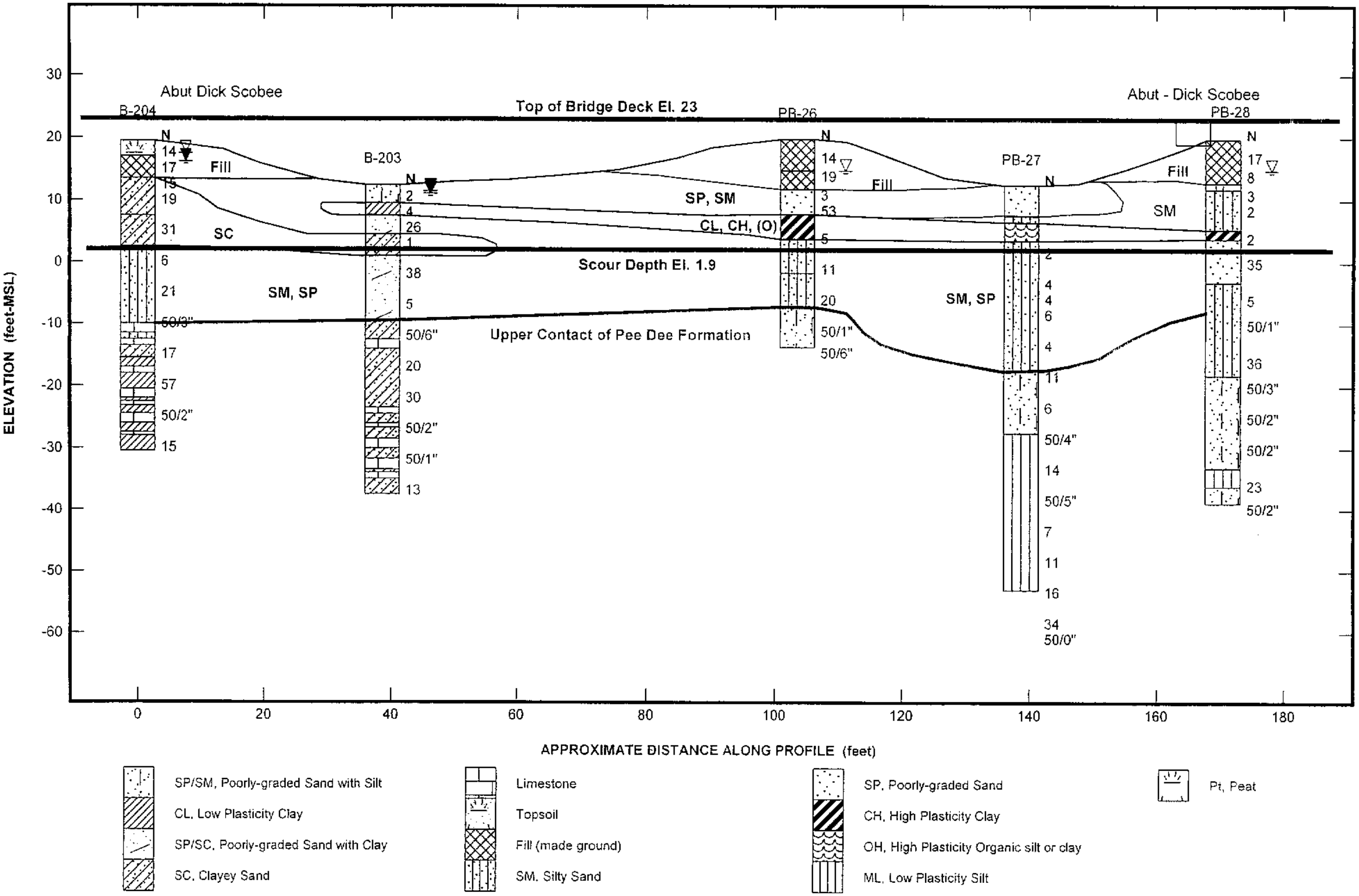
Ramp B & C, Frontage Road F
Highway 501
Boring Location Plan
Myrtle Beach, SC

SCALE: 1":100'	DRAWN BY: JWB	CHECKED BY:
JOB NO: 1611-99-401	DATE: 03-08-00	FIGURE NO: A3



N = Standard Penetration Test resistance value (blows per foot). The depicted stratigraphy is shown for illustrative purposes only. The actual subsurface conditions will vary between boring locations.

JOB NO:	1611-990-401	**S&ME** **134 Suber Road** Columbia, south Carolina 29210	Figure A-4, Ramp B, Soil Profile Highway 501 Myrtle Beach, South Carolina
DATE:	3/8/00		



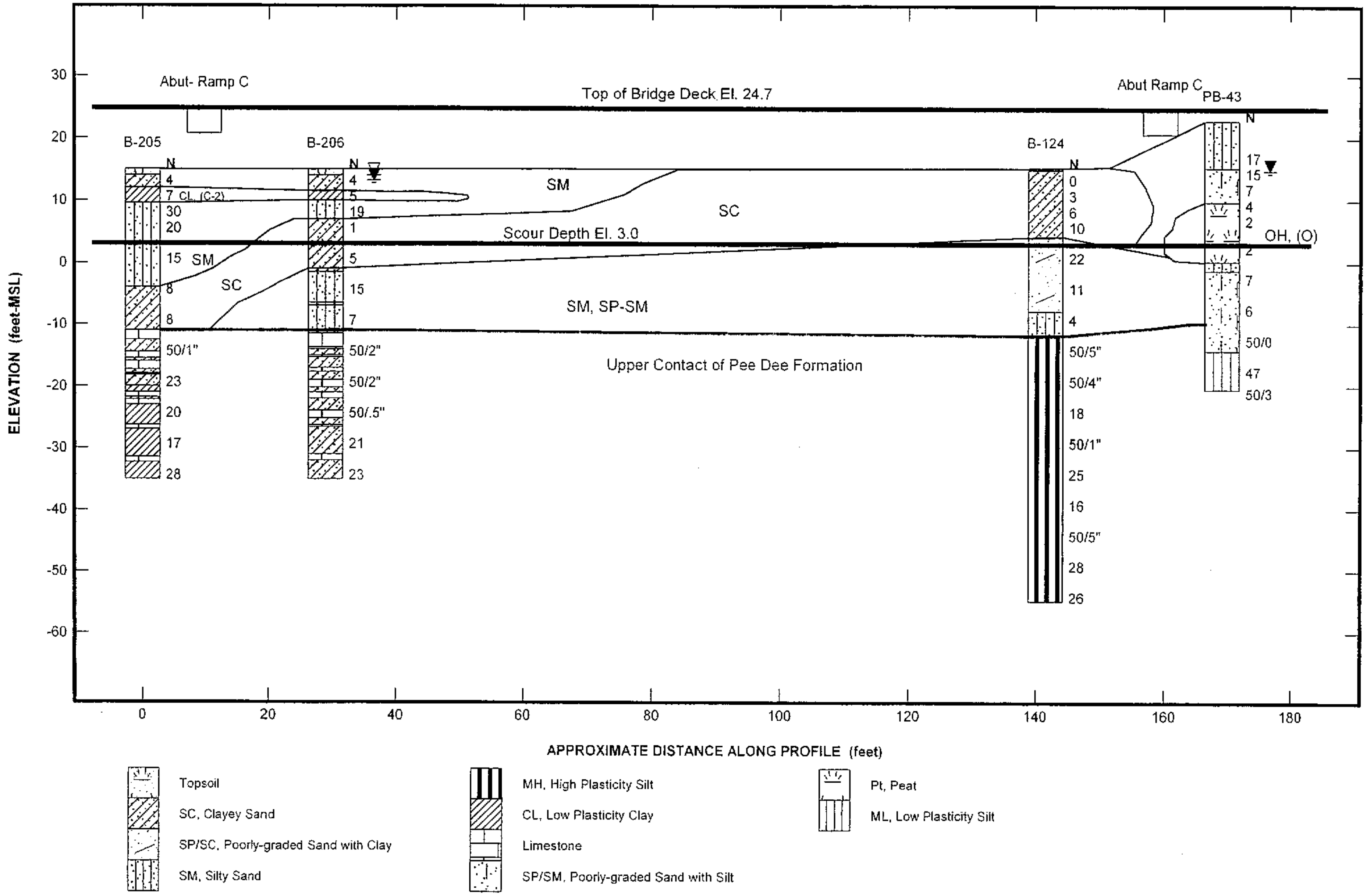
N = Standard Penetration Test resistance value (blows per foot). The depicted stratigraphy is shown for illustrative purposes only. The actual subsurface conditions will vary between boring locations.

JOB NO: 1611-990-401

DATE: 3/9/00

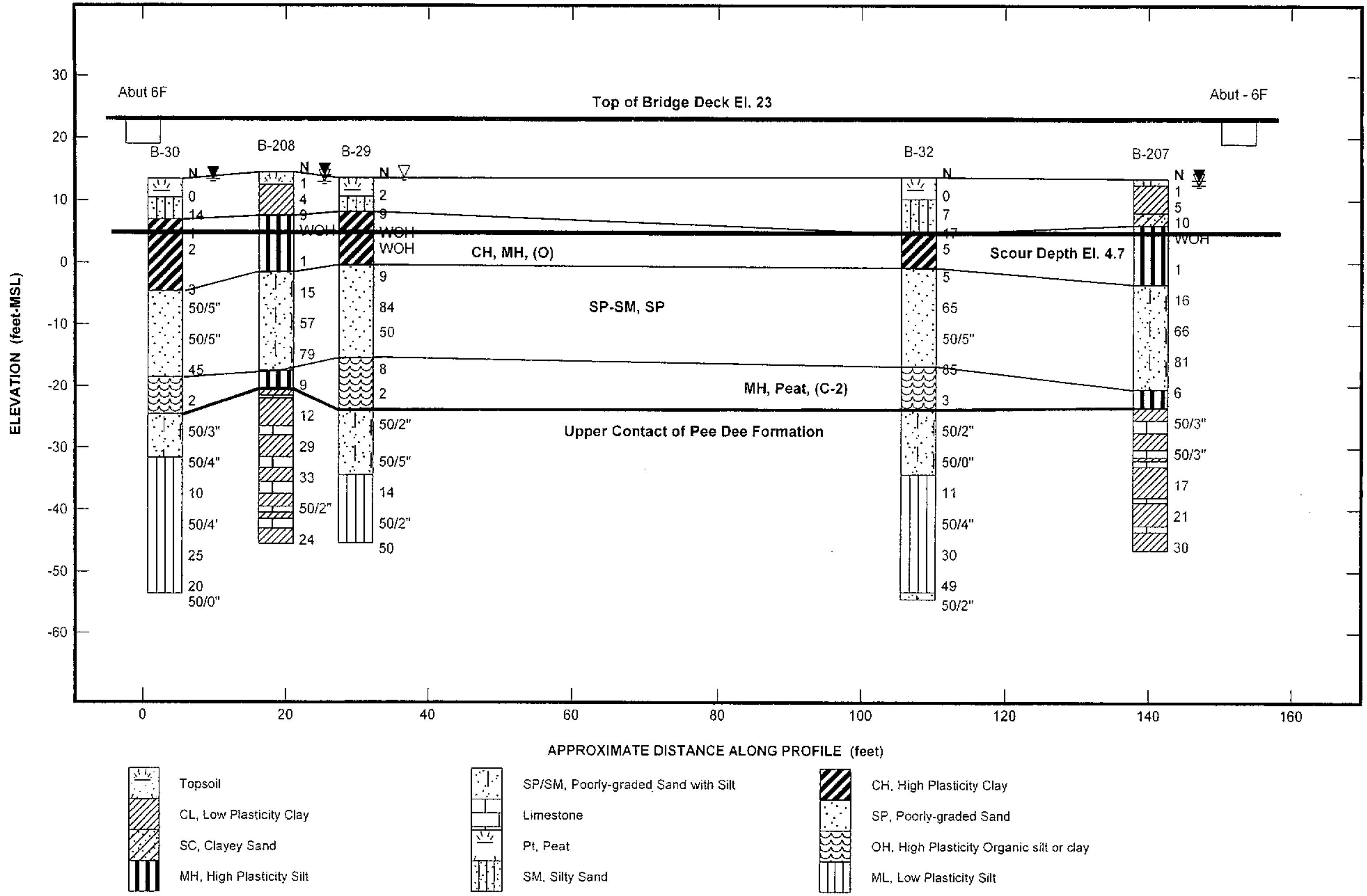
S&ME, Inc.
134 Suber Road
Columbia, South Carolina 29210

Figure A-5, Dick Scobee Drive, Soil Profile
Highway 501
Myrtle Beach, South Carolina



N = Standard Penetration Test resistance value (blows per foot). The depicted stratigraphy is shown for illustrative purposes only. The actual subsurface conditions will vary between boring locations.

JOB NO: 1611-990-401	S&ME 134 Suber Road Columbia, South Carolina 29210	Figure A-6, Ramp C, Soil Profile Highway 501 Myrtle Beach, South Carolina
DATE: 3/8/00		

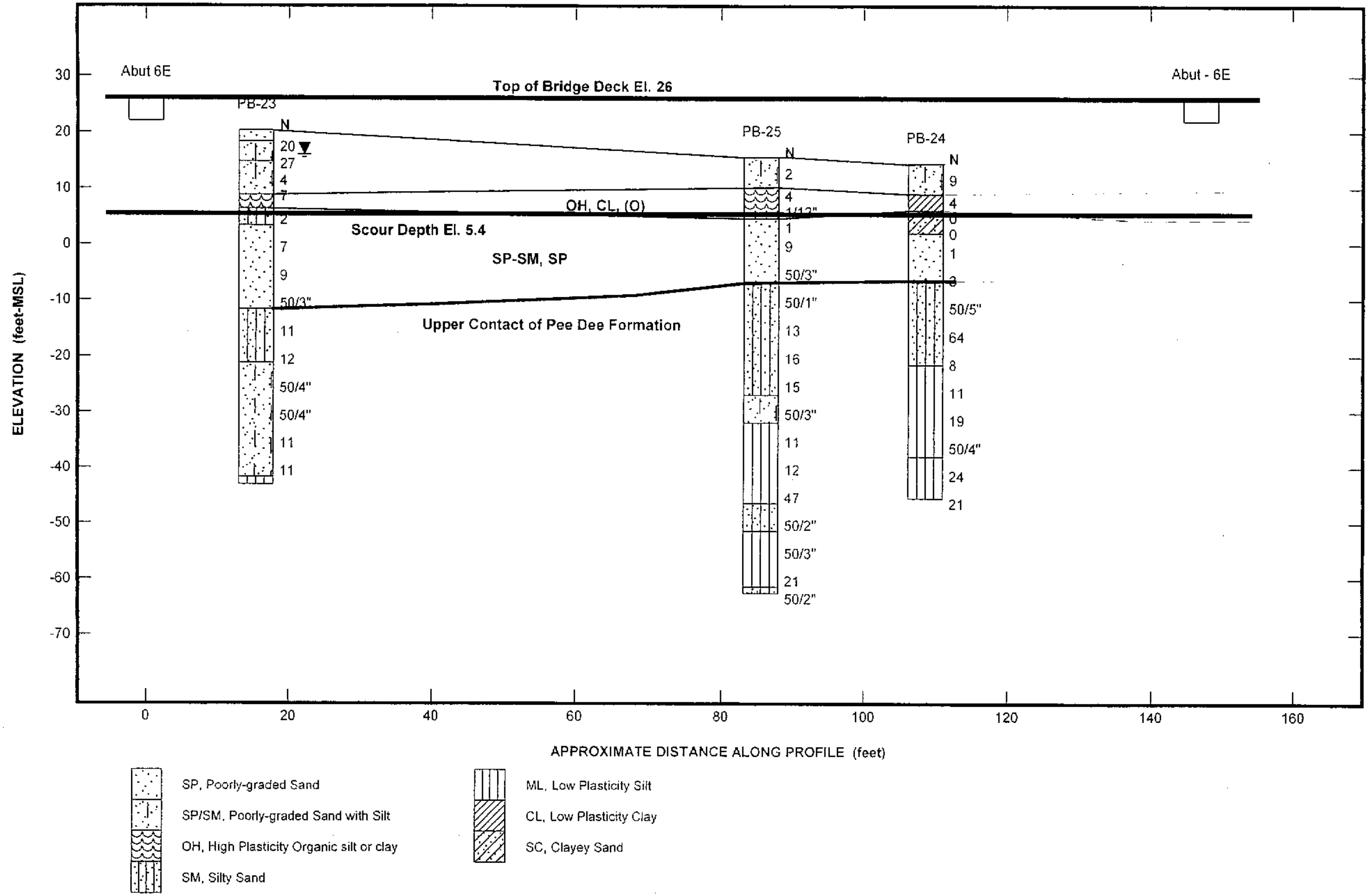


N = Standard Penetration Test resistance value (blows per foot). The depicted stratigraphy is shown for illustrative purposes only. The actual subsurface conditions will vary between boring locations.

JOB NO:	1611-990-401
DATE:	3/8/00

S&ME
134 Suber Road
Columbia, South Carolina 29210

Figure A-7, Frontage Road 6F, Soil Profile
Highway 501
Myrtle Beach, South Carolina



N = Standard Penetration Test resistance value (blows per foot). The depicted stratigraphy is shown for illustrative purposes only. The actual subsurface conditions will vary between boring locations.

JOB NO:	1611-990-401	**S&ME** **134 Suber Road** Columbia, South Carolina 29210	Figure A-8, Frontage Road 6E, Soil Profile Highway 501 Myrtle Beach, South Carolina
DATE:	3/8/00		

Appendix B

Field Data

Appendix B – 1
Soil Test Borings
2000

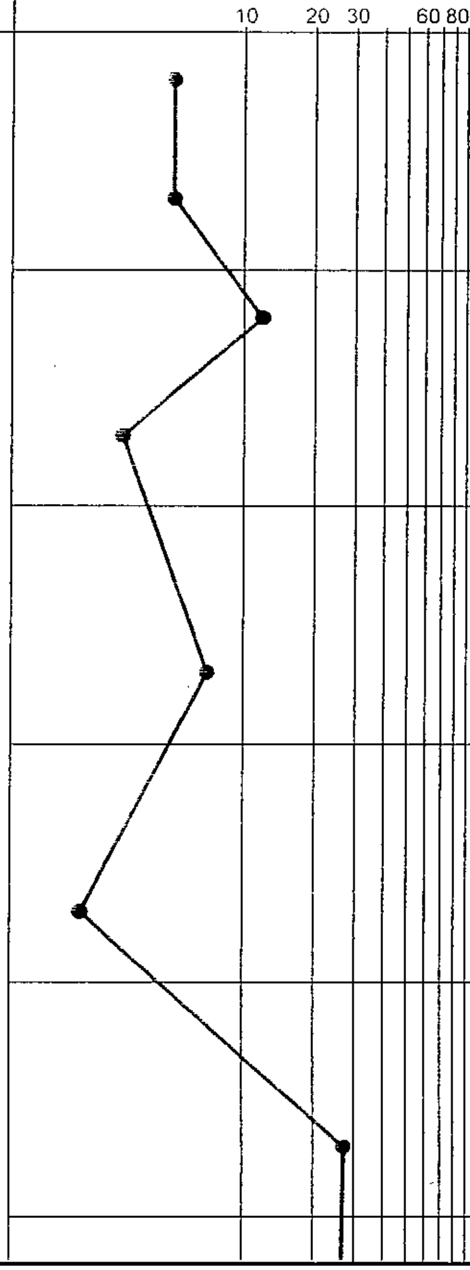
BORING LOG B-201

PROJECT:	Highway 501 Myrtle Beach, South Carolina 1611-990-401
DATE STARTED: 2/18/00	DATE FINISHED: 2/18/00
DRILLING METHOD: Mud Rotary	
CASING LENGTH:	DRILLER: Stone
WATER LEVEL: 0.67 feet at time of boring, 1 foot at 24 hours	

NOTES: **Approximate Station 544+80, N272510, E2625250, Ramp B**

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (feet-MSL)	SAMPLE NO/TYPE	N VALUE
	TOPSOIL - Silty sand, mostly fine sand, moist, dark gray to brown, organics			
	SILTY SAND (SM) - Mostly fine sand, some to few low plasticity fines, very moist to wet, brown to black, loose, roots to 5 ft.		1 SS	5
			2 SS	5
5		10.5		
	POORLY GRADED SAND WITH SILT (SP-SM) - Mostly fine sand, few to some low plasticity fines, wet, brownish gray, medium dense		3 SS	12
	CLAYEY SAND (SC) - Mostly fine sand, some low to medium plasticity fines, very moist to wet, dark gray, very loose		4 SS	3
10		5.5		
	SILTY SAND (SM) - Mostly fine sand, some low plasticity fines, many shells, wet, gray, loose, reacts with 10% HCl		5 SS	7
15		0.5		
	SILTY SAND (SM) - Mostly fine sand, some low plasticity fines, shells, very moist, dark gray, very loose, occasionally grades to clayey sand (SC), shells decrease with depth, reacts with 10% HCl		6 SS	2
20		-4.5		
	- medium dense		7 SS	27
25	**LIMESTONE** - Limestone lense, 1.5 ft. thick, gray, very hard	-9.5		

STANDARD PENETRATION TEST DATA (blows/ft)



BORING_LOG 99-401.GPJ S&ME.GDT 3/1/00

1. *BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
2. *PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*

S&ME

PROJECT: Highway 501
Myrtle Beach, South Carolina
1611-990-401

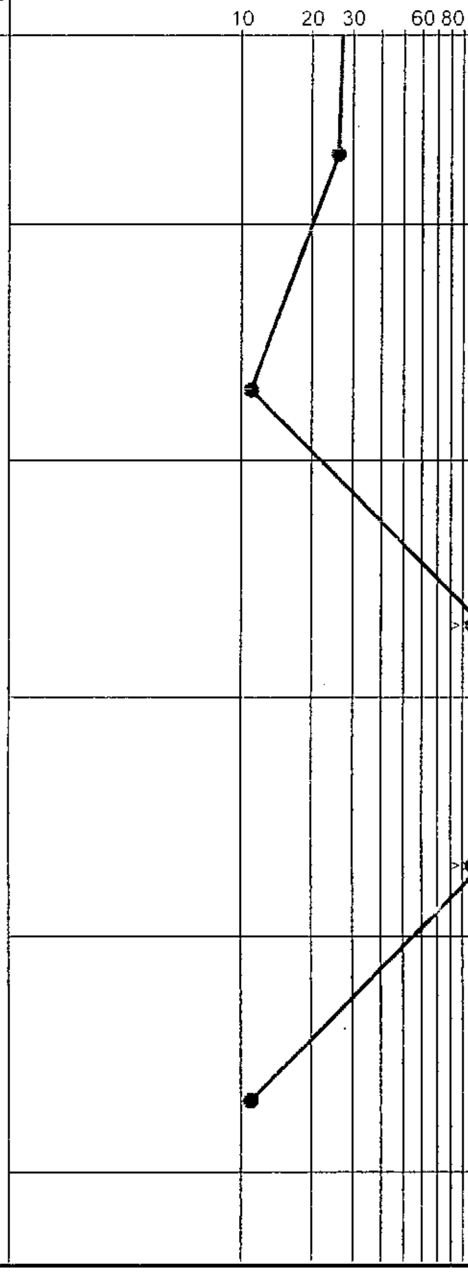
BORING LOG B-201

DATE STARTED: 2/18/00	DATE FINISHED: 2/18/00
DRILLING METHOD: Mud Rotary	
CASING LENGTH:	DRILLER: Stone
WATER LEVEL: 0.67 feet at time of boring, 1 foot at 24 hours	

NOTES: **Approximate Station 544+80, N272510, E2625250, Ramp B**

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (feet-MSL)	SAMPLE NO/TYPE	N VALUE
	SILTY SAND (SM) - Same as above, medium dense, (16 ft. to 25 ft.)			
	LIMESTONE - Limestone lense, 0.5 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Mostly fine sand, some medium plasticity fines, interlayered with 0.5 ft. to 2.5 ft. very hard limestone lenses, very moist, dark gray, medium, dense, reacts with 10% HCl, commonly grades to sandy lean clay (CL)		8 SS	26
30	**LIMESTONE** - Limestone lense, 1.5 ft. thick, gray, very hard	-14.5		
	CLAYEY SAND (SC) - Same as above, (27.5 ft. to 30 ft.)			
	LIMESTONE - Limestone lense, 0.5 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, (27.5 ft. to 30 ft.)		9 SS	11
35		-19.5		
	LIMESTONE - Limestone lense, 1.2 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, (27.5 ft. to 30 ft.)			
	LIMESTONE - Limestone lense, 0.5 ft. thick, gray, very hard		10 SS	50/ 2"
	CLAYEY SAND (SC) - Same as above, sand content decreasing, (27.5 ft. to 30 ft.)			
40	**LIMESTONE** - Limestone lense, 2.5 ft. thick, gray, very hard	-24.5		
	CLAYEY SAND (SC) - Same as above, (27.5 ft. to 30 ft.)		11 SS	50/ 3"
45	**LIMESTONE** - Limestone lense, 1.5 ft. thick, gray, very hard	-29.5		
	CLAYEY SAND (SC) - Same as above, (27.5 ft. to 30 ft.)			
	LIMESTONE - Limestone lense, 0.6 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, stiff, (27.5 ft. to 30 ft.), sand content increases with depth		12 SS	11
50	- Boring terminated at 50 feet.	-34.5		

STANDARD PENETRATION TEST DATA (blows/ft)



BORING LOG 99-401.GPJ S&ME.GDT 3/1/00

1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S&ME

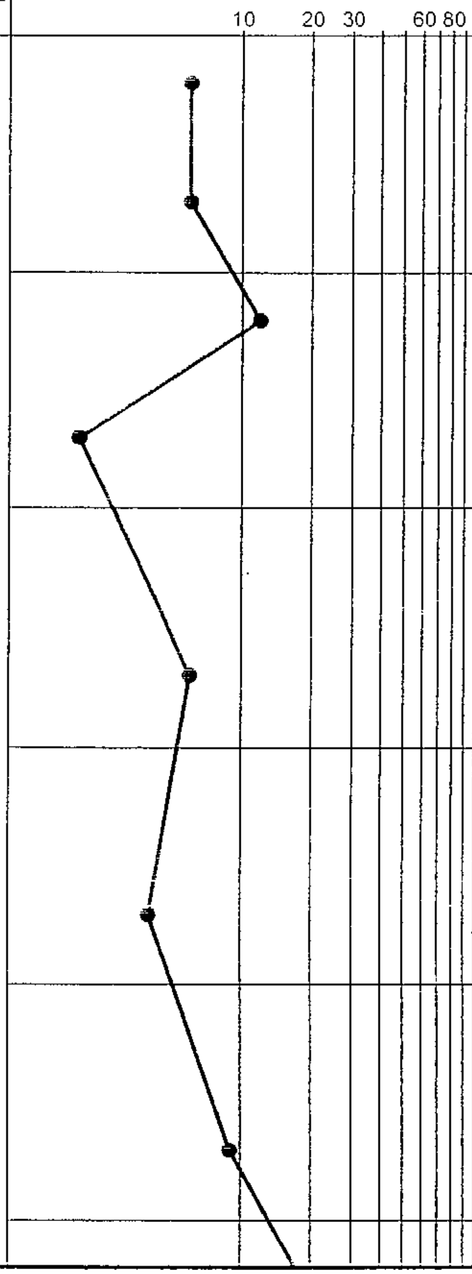
PROJECT: Highway 501 Myrtle Beach, South Carolina 1611-990-401	**BORING LOG B-202**

DATE STARTED:	2/18/00	DATE FINISHED:	2/18/00
DRILLING METHOD:	Mud Rotary		
CASING LENGTH:		DRILLER:	Stone
WATER LEVEL:	2.2 feet at time of boring, 1.9 feet at 24 hours		

NOTES: **Approximate Station 543+30, N272620, E2625150, Ramp B**

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION ((feet-MSL)	SAMPLE NO/TYPE	N VALUE
	SILTY SAND (SM) - Mostly medium sand, some fine and coarse sand, some to few low plasticity fines, shells, very moist to wet, brownish gray, loose		1 SS	6
	- wet			
	POORLY GRADED SAND WITH SILT (SP-SM) - Mostly fine sand, few to some low plasticity fines, wet, brown to black, loose, organics		2 SS	6
5		10.0		
	- medium dense		3 SS	12
	SILTY SAND (SM) - Mostly fine sand, some to few low plasticity fines, very moist to wet, brownish gray, medium dense, large roots			
	CLAYEY SAND (SC) - Mostly fine sand, some low plasticity fines, moist, dark gray, very loose		4 SS	2
10		5.0		
	SILTY SAND (SM) - Mostly fine sand, some low plasticity fines, many shells, wet, gray, loose, reacts with 10% HCl		5 SS	6
15		0.0		
	SILTY SAND (SM) - Mostly fine sand, some low plasticity fines, few shells, very moist to wet, dark gray, very loose, reacts with 10% HCl		6 SS	4
20		-5.0		
	- loose		7 SS	9
25		-10.0		

STANDARD PENETRATION TEST DATA (blows/ft)



BORING LOG 99-401.GPJ S&ME.GDT 3/1/00

1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S&ME

BORING LOG B-202

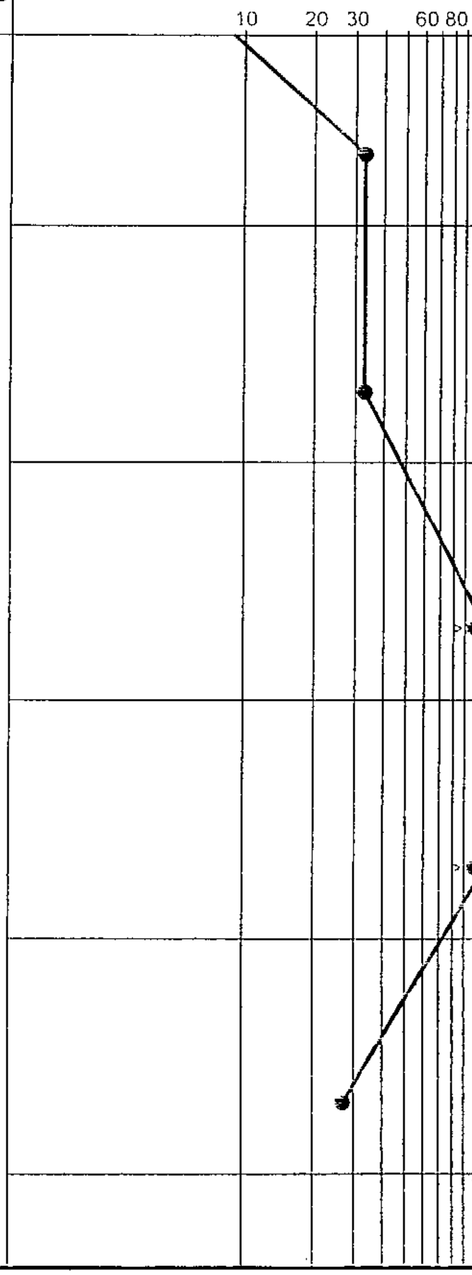
PROJECT: Highway 501
Myrtle Beach, South Carolina
1611-990-401

DATE STARTED: 2/18/00	DATE FINISHED: 2/18/00
DRILLING METHOD: Mud Rotary	
CASING LENGTH:	DRILLER: Stone
WATER LEVEL: 2.2 feet at time of boring, 1.9 feet at 24 hours	

NOTES: Approximate Station 543+30, N272620, E2625150, Ramp B

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION ((feet-MSL)	SAMPLE NO/TYPE	N VALUE
	CLAYEY SAND (SC) - Mostly fine sand, some medium plasticity fines, interlayered with 0.5 ft. to 2.0 ft. very hard limestone lenses, very moist, dark gray, medium dense, reacts with 10% HCl, commonly grades to sandy lean clay (CL)			
	LIMESTONE - Limestone lense, 1.5 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, dense, (26 ft. to 26.5 ft.)		8 SS	33
30		-15.0		
	LIMESTONE - Limestone lense, 1 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, dense, (26 ft. to 26.5 ft.)		9 SS	33
35		-20.0		
	LIMESTONE - Limestone lense, 1.5 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, dense, (26 ft. to 26.5 ft.)			
	LIMESTONE - Limestone lense, 0.5 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, dense, (26 ft. to 26.5 ft.)		10 SS	50/ 2"
40		-25.0		
	LIMESTONE - Limestone lense, 0.8 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, dense, (26 ft. to 26.5 ft.)			
	LIMESTONE - Limestone lense, 1.5 ft. thick, gray, very hard		11 SS	50/ 1"
	CLAYEY SAND (SC) - Same as above, dense, (26 ft. to 26.5 ft.)			
45		-30.0		
	LIMESTONE - Limestone lense, 2 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, medium dense, (26 ft. to 26.5 ft.)		12 SS	27
50	- Boring terminated at 50 feet.	-35.0		

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

BORING LOG 99-401.GPJ S&ME.GDT 3/1/00

S&ME

PROJECT: Highway 501 Myrtle Beach, South Carolina 1611-990-401	**BORING LOG B-203**

DATE STARTED: **2/19/00** | DATE FINISHED: **2/19/00**

DRILLING METHOD: **Mud Rotary**

CASING LENGTH: | DRILLER: **Stone**

WATER LEVEL: **1.4 feet at time of boring, 1 foot at 24 hours**

NOTES: **Approximate Station 38+10, N273100, E2625570, Dick Scobee**

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (feet-MSL)	SAMPLE NO/TYPE	N VALUE
	POORLY GRADED SAND WITH SILT (SP-SM) - Mostly fine sand, few to some low plasticity fines, wet, gray, very loose, roots		1 SS	2
	LEAN CLAY WITH SAND (CL) - Mostly medium plasticity fines, some fine sand, very moist dark gray to brown, soft		2 SS	4
5	**POORLY GRADED SAND WITH CLAY (SP-SC)** - Mostly fine to medium sand, some medium plasticity fines, wet, brownish gray, medium dense	7.5	3 SS	26
	CLAYEY SAND (SC) - Mostly fine sand, some low plasticity fines, moist, light gray, very loose		4 SS	1
10		2.5		
	POORLY GRADED SAND WITH CLAY (SP-SC) - Mostly fine to medium sand, some medium plasticity fines, shells, wet, brownish gray, dense, large shells with depth, reacts with 10% HCl		5 SS	38
15		-2.5		
	- loose		6 SS	5
20		-7.5		
	CLAYEY SAND (SC) - Mostly fine sand, some medium plasticity fines, interlayered with 0.5 ft. to 1.8 ft. very hard limestone lenses, very moist, dark gray, medium dense, reacts with 10% HCl, commonly grades to sandy lean clay (CL)		7 SS	50/ 6"
25	**LIMESTONE** - Limestone lense, 1.5 ft. thick, gray, very hard	-12.5		

STANDARD PENETRATION TEST DATA (blows/ft): 10, 20, 30, 60, 80

BORING LOG 99-401.GPJ S&ME.GDT 3/1/00

1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.

2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S&ME

PROJECT: Highway 501 Myrtle Beach, South Carolina 1611-990-401	**BORING LOG B-203**

DATE STARTED: **2/19/00**	DATE FINISHED: **2/19/00**
DRILLING METHOD: **Mud Rotary**	
CASING LENGTH:	DRILLER: **Stone**
WATER LEVEL: **1.4 feet at time of boring, 1 foot at 24 hours**	

NOTES: **Approximate Station 38+10, N273100, E2625570, Dick Scobee**

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (feet-MSL)	SAMPLE NO/TYPE	N VALUE
	CLAYEY SAND (SC) - Same as above, medium dense, (22 ft. to 25 ft.), clay lenses common		8 SS	20
30		-17.5		
			9 SS	30
35		-22.5		
	LIMESTONE - Limestone lense, 1.0 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, medium dense, (22 ft. to 25 ft.)			
	LIMESTONE - Limestone lense, 0.7 ft. thick, gray, very hard		10 SS	50/ 2"
40	**CLAYEY SAND (SC)** - Same as above, medium dense, (22 ft. to 25 ft.)	-27.5		
	LIMESTONE - Limestone lense, 1.5 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, medium dense, (22 ft. to 25 ft.)		11 SS	50/ 1"
45	**LIMESTONE** - Limestone lense, 1.8 ft. thick, gray, very hard	-32.5		
	CLAYEY SAND (SC) - Same as above, medium dense, (22 ft. to 25 ft.)			
	LIMESTONE - Limestone lense, 1.0 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, medium dense, (22 ft. to 25 ft.)		12 SS	13
50	- Boring terminated at 50 feet.	-37.5		

STANDARD PENETRATION TEST DATA (blows/ft): 10 20 30 60 80

BORING LOG 99-401.GPJ S&ME.GDT 3/1/00

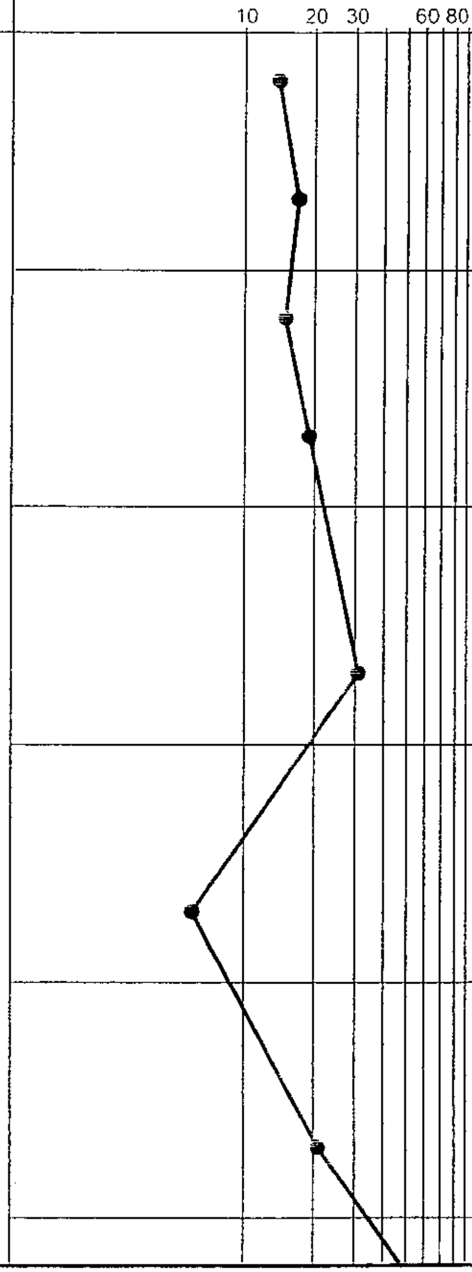
1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S&ME

PROJECT: Highway 501 Myrtle Beach, South Carolina 1611-990-401		BORING LOG B-204
DATE STARTED: 2/19/00	DATE FINISHED: 2/19/00	NOTES: Approximate Station 38+40, N273240, E2625530, Dick Scobee
DRILLING METHOD: Mud Rotary		
CASING LENGTH:	DRILLER: Stone	
WATER LEVEL: 2 feet at time of boring, 3.3 feet at 24 hours		

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION ((feet-MSL)	SAMPLE NO/TYPE	N VALUE
		TOPSOIL - Fill, Silty sand, mostly fine sand, moist, dark gray to brown, organics, large roots		1 SS	14
		FILL, CLAYEY SAND (SC) - Mostly fine sand, some low plasticity fines, moist, dark gray, medium dense, roots		2 SS	17
5			14.5		
		CLAYEY SAND (SC) - Mostly fine sand, some low plasticity fines, moist, brownish gray, medium dense, commonly grades to poorly graded sand with clay (SP-SC)		3 SS	15
				4 SS	19
10			9.5		
		CLAYEY SAND (SC) - Mostly fine sand, some low to medium plasticity fines, many shells, some large shells, very moist to wet, dark gray, dense, reacts with 10% HCl		5 SS	31
15			4.5		
		SILTY SAND (SM) - Mostly fine sand, some low plasticity fines, shells and occasional limestone lenses, very moist, dark gray, loose, occasionally grades to clayey sand (SC), reacts with 10% HCl		6 SS	6
20			-0.5		
		- medium dense		7 SS	21
25			-5.5		

STANDARD PENETRATION TEST DATA (blows/ft)



BORING LOG 99-401.GPJ S&ME.GDT 3/1/00

1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.

2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S&ME

PROJECT: Highway 501
Myrtle Beach, South Carolina
1611-990-401

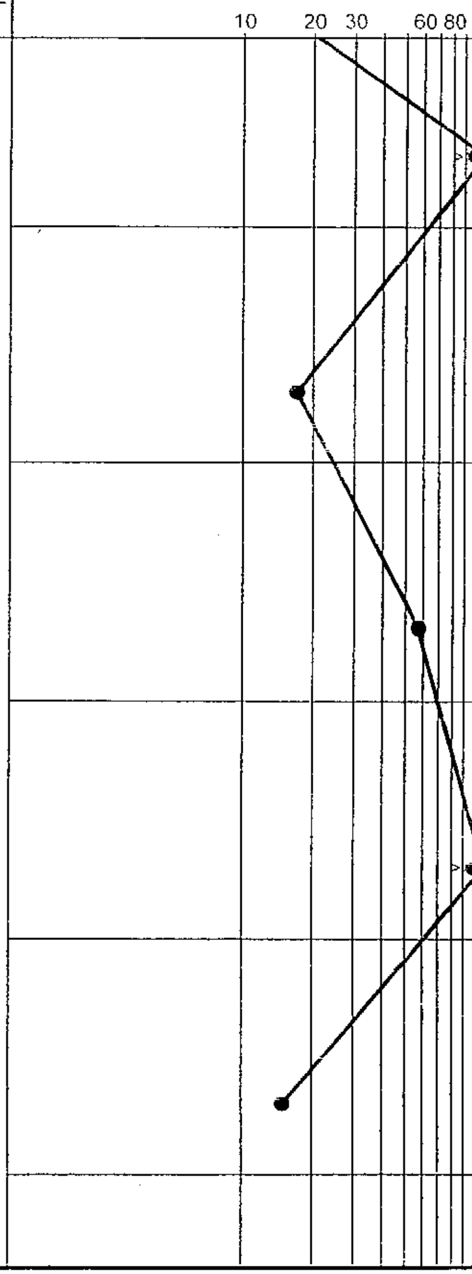
BORING LOG B-204

DATE STARTED: 2/19/00	DATE FINISHED: 2/19/00
DRILLING METHOD: Mud Rotary	
CASING LENGTH:	DRILLER: Stone
WATER LEVEL: 2 feet at time of boring, 3.3 feet at 24 hours	

NOTES: Approximate Station 38+40, N273240, E2625530, Dick Scobee

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION ((feet-MSL)	SAMPLE NO/TYPE	N VALUE
			8 SS	50/ 3"
30	**LIMESTONE** - Limestone lense, 1.5 ft. thick, gray, very hard	-10.5		
	SILTY SAND (SM) - Same as above, (17 ft. to 29.5 ft.)			
	LIMESTONE - Limestone lense, 1.0 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Mostly fine sand, some low to medium plasticity fines, very moist to wet, dark gray, medium dense, reacts with 10% HCl		9 SS	17
35	**SANDY LEAN CLAY (CL)** - Mostly medium plasticity fines, some fine sand, interlayered with 0.5 ft. to 1.5 ft. very hard limestone lenses, very moist, dark gray, hard, reacts with 10% HCl, occasionally grades to a clayey sand (SC)	-15.5		
	LIMESTONE - Limestone lense, 1.0 ft. thick, gray, very hard			
	SANDY LEAN CLAY (CL) - Same as above, very hard, (35 ft. to 36.5 ft.)		10 SS	57
40	**LIMESTONE** - Limestone lense, 1.5 ft. thick, gray, very hard	-20.5		
	SANDY LEAN CLAY (CL) - Same as above, stiff, (35 ft. to 36.5 ft.)			
	LIMESTONE - Limestone lense, 0.7 ft. thick, gray, very hard			
	SANDY LEAN CLAY (CL) - Same as above, stiff, (35 ft. to 36.5 ft.)		11 SS	50/ 2"
	LIMESTONE - Limestone lense, 1.5 ft. thick, gray, very hard			
45		-25.5		
	SANDY LEAN CLAY (CL) - Same as above, stiff, (35 ft. to 36.5 ft.)			
	LIMESTONE - Limestone lense, 0.5 ft. thick, gray, very hard			
	SANDY LEAN CLAY (CL) - Same as above, stiff, (35 ft. to 36.5 ft.)		12 SS	15
50	- Boring terminated at 50 feet.	-30.5		

STANDARD PENETRATION TEST DATA (blows/ft)



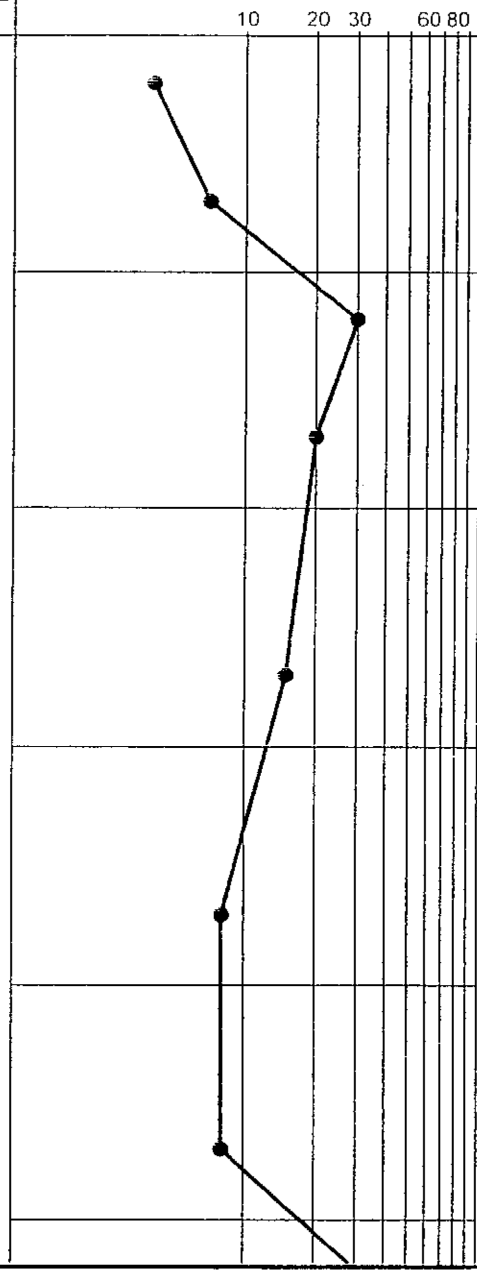
1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S&ME

PROJECT: Highway 501 Myrtle Beach, South Carolina 1611-990-401	**BORING LOG B-205**
DATE STARTED: 2/16/00 — DATE FINISHED: 2/16/00	NOTES: Approximate Station 543+00, N272500, E2624970, Ramp C
DRILLING METHOD: Mud Rotary	
CASING LENGTH: — DRILLER: Stone	
WATER LEVEL:	

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION ((feet-MSL)	SAMPLE NO/TYPE	N VALUE
	TOPSOIL - Silty sand, mostly fine sand, dark gray to brown, moist, organics			
	CLAYEY SAND (SC) - Mostly fine sand, some low plasticity fines, moist, dark gray to brown, loose		1 SS	4
	LEAN CLAY WITH SAND (CL) - Mostly medium plasticity fines, some fine sand, very moist, dark gray to brown, soft		2 SS	7
5		10.0		
	SILTY SAND (SM) - Mostly fine sand, some to few low plasticity fines, very moist to wet, brownish gray, dense, commonly grades to poorly graded sand with silt (SP-SM)		3 SS	30
	- medium dense		4 SS	20
10		5.0		
	SILTY SAND (SM) - Mostly fine sand, some low plasticity fines, shells and occasional limestone lenses, very moist, dark gray, medium dense, occasionally grades to clayey sand (SC)		5 SS	15
15		0.0		
			6 SS	8
	CLAYEY SAND (SC) - Mostly fine sand, some low to medium plasticity fines, very moist to wet, dark gray, loose, interlayered with 0.2 ft. to 1.5 ft. thick very hard limestone lenses, occasionally grades to sandy lean clay (CL)			
20		-5.0		
			7 SS	8
25		-10.0		

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

BORING LOG 99-401.GPJ S&ME.GDT 3/1/00

S&ME

PROJECT: Highway 501
Myrtle Beach, South Carolina
1611-990-401

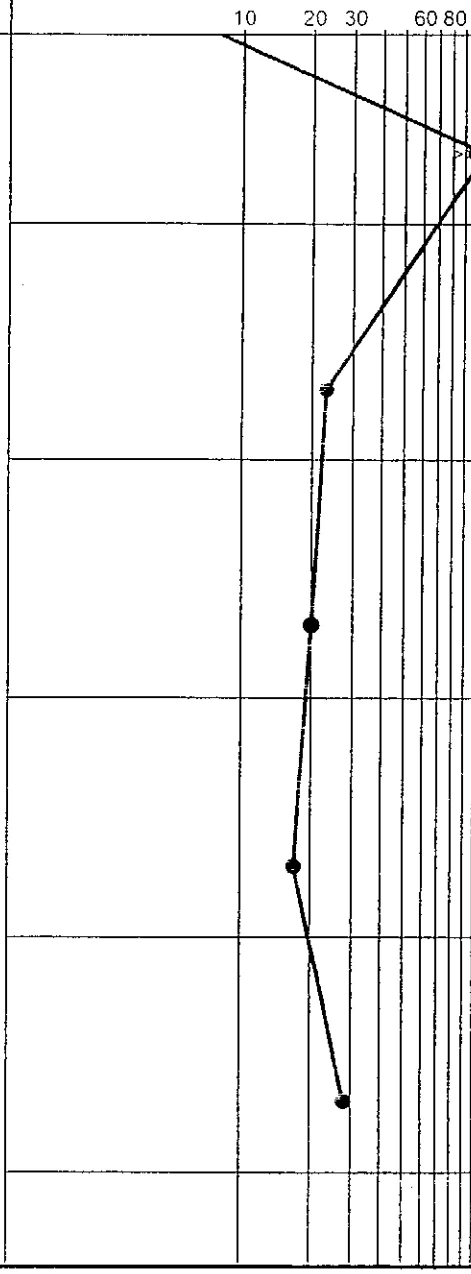
BORING LOG B-205

DATE STARTED: 2/16/00	DATE FINISHED: 2/16/00
DRILLING METHOD: Mud Rotary	
CASING LENGTH:	DRILLER: Stone
WATER LEVEL:	

NOTES: Approximate Station 543+00, N272500, E2624970, Ramp C

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION ((feet-MSL)	SAMPLE NO/TYPE	N VALUE
		LIMESTONE - Limestone lense, 1.5 ft. thick, gray, very hard			
		CLAYEY SAND (SC) - Same as above, loose, (19 ft. 26 ft.)		8 SS	50/ 1"
30		**LIMESTONE** - Limestone lense, 1.0 ft. thick, gray, very hard	-15.0		
		CLAYEY SAND (SC) - Same as above, loose, (19 ft. 26 ft.)			
		LIMESTONE - Limestone lense, 1.3 ft. thick, gray, very hard			
		CLAYEY SAND (SC) - Same as above, loose, (19 ft. 26 ft.)			
		LIMESTONE - Limestone lense, 0.2 ft. thick, gray, very hard			
		CLAYEY SAND (SC) - Same as above, medium dense, (19 ft. 26 ft.)		9 SS	23
35		**SANDY LEAN CLAY (CL)** - Mostly medium plasticity fines, some low plasticity fines, some fine sand, interlayered with 0.7 ft. to 0.8 ft. very hard limestone lenses, very moist, dark gray, stiff, reacts with 10% HCl, occasionally grades to a clayey sand (SC)	-20.0		
		LIMESTONE - Limestone lense, 0.8 ft. thick, gray, very hard			
		SANDY LEAN CLAY (CL) - Same as above, (35 ft. to 36 ft.)			
		LIMESTONE - Limestone lense, 0.8 ft. thick, gray, very hard			
		SANDY LEAN CLAY (CL) - Same as above, hard, (35 ft. to 36 ft.)		10 SS	20
40			-25.0		
		LIMESTONE - Limestone lense, 0.7 ft. thick, gray, very hard			
		SANDY LEAN CLAY (CL) - Same as above, hard, (35 ft. to 36 ft.)		11 SS	17
45			-30.0		
		LIMESTONE - Limestone lense, 0.8 ft. thick, gray, very hard			
		SANDY LEAN CLAY (CL) - Same as above, hard, (35 ft. to 36 ft.)		12 SS	28
50		- Boring terminated at 50 feet.	-35.0		

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S&ME

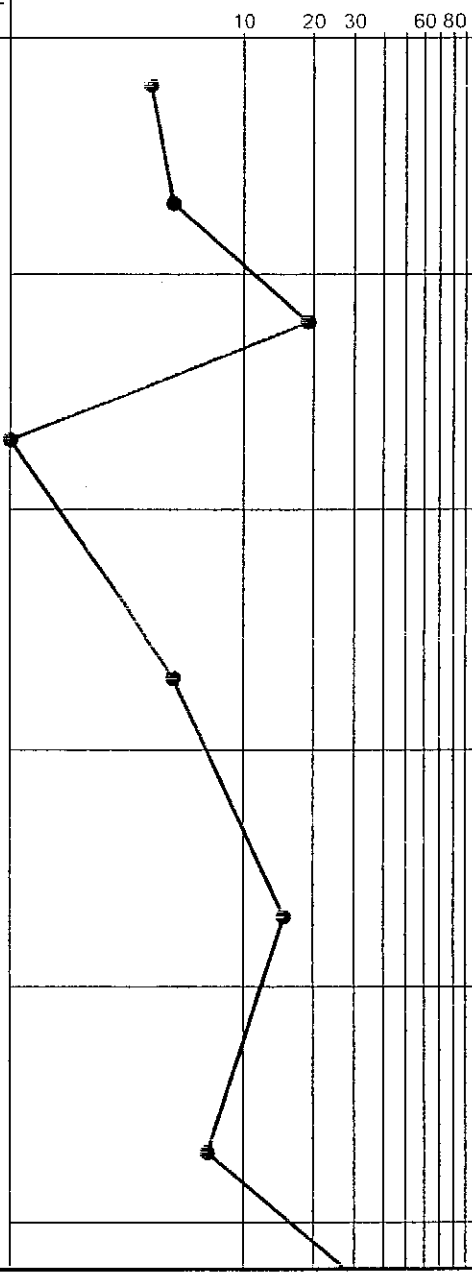
PROJECT: Highway 501 Myrtle Beach, South Carolina 1611-990-401	**BORING LOG B-206**

DATE STARTED:	**2/16/00**	DATE FINISHED:	**2/16/00**
DRILLING METHOD:	**Mud Rotary**		
CASING LENGTH:		DRILLER:	**Stone**
WATER LEVEL:	**1 foot at time of boring, 1.8 feet at 24 hours**		

NOTES: **Approximate Station 543+30, N272490, E2625000, Ramp C**

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (feet-MSL)	SAMPLE NO/TYPE	N VALUE
	TOPSOIL - Silty sand, mostly fine sand, moist, dark gray to brown, organics		1 SS	4
	CLAYEY SAND (SC) - Mostly fine sand, some low plasticity fines, moist, dark gray, loose		2 SS	5
	LEAN CLAY WITH SAND (CL) - Mostly medium plasticity fines, some fine sand, very moist, dark gray to brown, soft			
5	**SILTY SAND (SM)** - Mostly fine sand, some to few low plasticity fines, very moist to wet, brownish gray, medium dense	10.0	3 SS	19
	CLAYEY SAND (SC) - Mostly fine to medium sand, some low to medium plasticity fines, wood chips common, wet, dark gray, very loose, grades to poorly graded sand with silt (SP-SM) with depth		4 SS	1
10		5.0		
	- loose		5 SS	5
15		0.0		
	CLAYEY SAND (SC) - same as above but very hard lense			
	SILTY SAND (SM) - Mostly fine sand, some low plasticity fines, shells and occasional limestone lenses, very moist, dark gray, medium dense, occasionally grades to clayey sand (SC)		6 SS	15
20		-5.0		
	SILTY SAND (SM) - Same as above but very hard lense			
	SILTY SAND (SM) - Same as above, loose, (16.5 ft. to 21.5 ft.)		7 SS	7
25		-10.0		

STANDARD PENETRATION TEST DATA (blows/ft)



BORING LOG 99-401.GPJ S&ME.GDT 3/1/00

1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S&ME

PROJECT: Highway 501 Myrtle Beach, South Carolina 1611-990-401	**BORING LOG B-206**

DATE STARTED: 2/16/00	DATE FINISHED: 2/16/00
DRILLING METHOD: Mud Rotary	
CASING LENGTH:	DRILLER: Stone
WATER LEVEL: 1 foot at time of boring, 1.8 feet at 24 hours	

NOTES: Approximate Station 543+30, N272490, E2625000, Ramp C

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION ((feet-MSL)	SAMPLE NO/TYPE	N VALUE
	CLAYEY SAND (SC) - Mostly fine sand, some medium plasticity fines, interlayered with 0.3 ft. to 2.5 ft. very hard limestone lenses, very moist, dark gray, medium dense, reacts with 10% HCl, commonly grades to a sandy lean clay (CL)			
	LIMESTONE - Limestone lense, 2.5 ft. thick, gray, very hard		8 SS	50/ 2"
	CLAYEY SAND (SC) - Same as above, (26 ft. to 26.5 ft.)			
30	LIMESTONE - Limestone lense, 0.3 ft. thick, gray, very hard	-15.0		
	CLAYEY SAND (SC) - Same as above, (26 ft. to 26.5 ft.)			
	LIMESTONE - Limestone lense, 0.7 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, (26 ft. to 26.5 ft.)		9 SS	50/ 2"
	LIMESTONE - Limestone lense, 1.2 ft. thick, gray, very hard			
35	CLAYEY SAND (SC) - Same as above, (26 ft. to 26.5 ft.)	-20.0		
	LIMESTONE - Limestone lense, 1.0 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, (26 ft. to 26.5 ft.)		10 SS	50/ .5"
	LIMESTONE - Limestone lense, 1.2 ft. thick, gray, very hard			
40	CLAYEY SAND (SC) - Same as above, (26 ft. to 26.5 ft.)	-25.0		
	LIMESTONE - Limestone lense, 1.3 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, (26 ft. to 26.5 ft.)		11 SS	21
45		-30.0		
	LIMESTONE - Limestone lense, 1.0 ft. thick, gray, very hard			
	CLAYEY SAND (SC) - Same as above, (26 ft. to 26.5 ft.)		12 SS	23
50	- Boring terminated at 50 feet.	-35.0		

STANDARD PENETRATION TEST DATA (blows/ft): 10 20 30 60 80

1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

BORING LOG 99-401.GPJ S&ME.GDT 3/1/00

S&ME

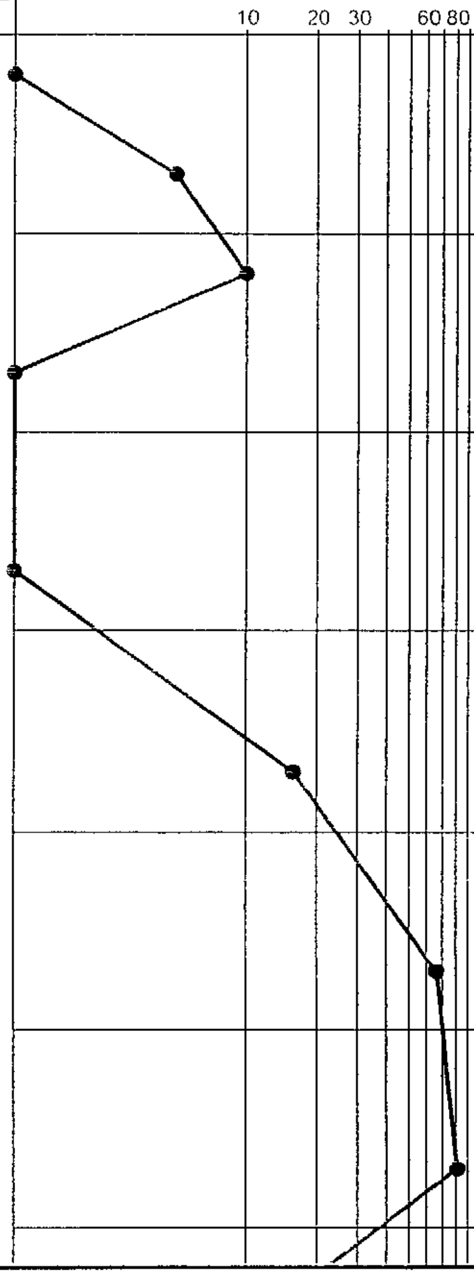
PROJECT: Highway 501 Myrtle Beach, South Carolina 1611-990-401	**BORING LOG B-207**

DATE STARTED: **2/17/00**	DATE FINISHED: **2/17/00**
DRILLING METHOD: **Mud Rotary**	
CASING LENGTH:	DRILLER: Stone
WATER LEVEL: **1 foot at time of boring, 0.25 feet at 24 hours**	

NOTES: **Approximate Station 483+10, N271980, E2624910, Frontage Road 6F**

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION ((feet-MSL)	SAMPLE NO/TYPE	N VALUE
	TOPSOIL - Silty sand, mostly fine sand, moist, dark gray to brown, organics			
	LEAN CLAY WITH SAND (CL) - Mostly medium plasticity fines, some fine sand, very moist dark gray to brown, soft		1 SS	1
			2 SS	5
5		8.5		
	CLAYEY SAND (SC) - Mostly fine sand, some low plasticity fines, moist, dark gray, loose		3 SS	10
	ELASTIC SILT (MH) - Mostly low to medium plasticity fines, some fine sand, very moist to wet, black, very soft, roots		4 SS	WOH
10		3.5		
	- large roots		5 SS	1
15		-1.5		
	POORLY GRADED SAND WITH SILT (SP-SM) - Mostly fine sand, few to some low plasticity fines, wet, gray, medium dense, occasionally grades to silty sand (SM)		6 SS	16
20		-6.5		
	- very dense		7 SS	66
25		-11.5		
			8 SS	81
30		-16.5		

STANDARD PENETRATION TEST DATA (blows/ft)



BORING LOG 99-401.GPJ S&ME.GDT 3/1/00

1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S&ME

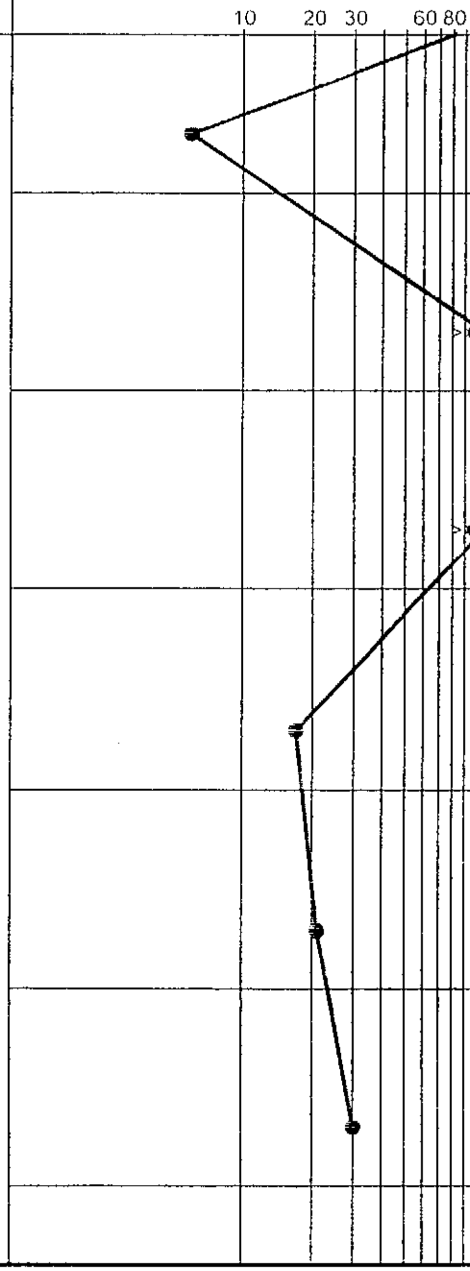
BORING LOG B-207

PROJECT:	Highway 501 Myrtle Beach, South Carolina 1611-990-401
DATE STARTED: 2/17/00	DATE FINISHED: 2/17/00
DRILLING METHOD: Mud Rotary	
CASING LENGTH:	DRILLER: Stone
WATER LEVEL: 1 foot at time of boring, 0.25 feet at 24 hours	

NOTES: Approximate Station 483+10, N271980, E2624910, Frontage Road 6F

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION ((feet-MSL)	SAMPLE NO/TYPE	N VALUE
			9 SS	6
35	**ELASTIC SILT (MH)** - Mostly low to medium plasticity fines, some fine sand, very moist to wet, black, soft	-21.5		
	SANDY LEAN CLAY (CL) - Mostly medium plasticity fines, some fine sand, interlayered with 0.8 ft. to 2.0 ft. very hard limestone lenses, very moist, dark gray, stiff, reacts with 10% HCl, commonly grades to a clayey sand (SC)		10 SS	50/ 3"
40	**LIMESTONE** - Limestone lense, 2.0 ft. thick, gray, very hard	-26.5		
	SANDY LEAN CLAY (CL) - Same as above, stiff, (37 ft. to 39 ft.)			
	LIMESTONE - Limestone lense, 1.3 ft. thick, gray, very hard		11 SS	50/ 3"
45	**SANDY LEAN CLAY (CL)** - Same as above, (37 ft. to 39 ft.)	-31.5		
	LIMESTONE - Limestone lense, 1.0 ft. thick, gray, very hard			
	SANDY LEAN CLAY (CL) - Same as above, hard, (37 ft. to 39 ft.)		12 SS	17
50		-36.5		
	LIMESTONE - Limestone lense, 0.8 ft. thick, gray, very hard			
	SANDY LEAN CLAY (CL) - Same as above, hard, (37 ft. to 39 ft.)		13 SS	21
55		-41.5		
	LIMESTONE - Limestone lense, 1.0 ft. thick, gray, very hard			
	SANDY LEAN CLAY (CL) - Same as above, hard, (37 ft. to 39 ft.), more sand than above		14 SS	30
60	- Boring terminated at 60 feet.	-46.5		

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

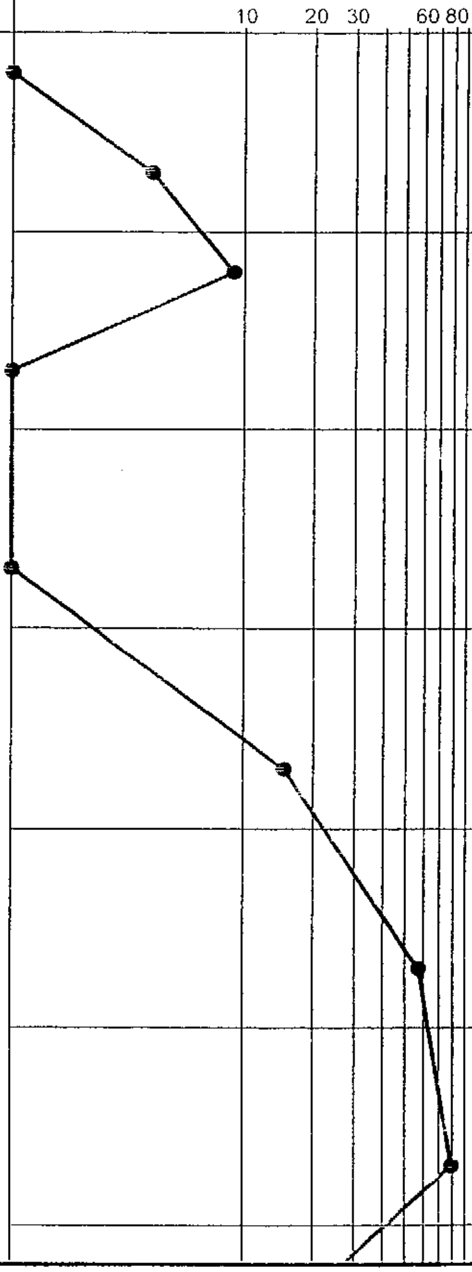
S&ME

PROJECT: Highway 501 Myrtle Beach, South Carolina 1611-990-401	BORING LOG B-208

DATE STARTED: 2/17/00	DATE FINISHED: 2/17/00	NOTES: Approximate Station 481+80, N272000, E2624790, Frontage Road 6F
DRILLING METHOD: Mud Rotary		
CASING LENGTH:	DRILLER: Stone	
WATER LEVEL: 1.5 feet at time of boring, 0.5 feet at 24 hours		

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (feet-MSL)	SAMPLE NO/TYPE	N VALUE
	TOPSOIL - Silty sand, mostly fine sand, moist, dark gray to brown, organics		1 SS	1
	LEAN CLAY WITH SAND (CL) - Mostly medium plasticity fines, some fine sand, very moist to wet, brown, soft, occasionally grades to a clayey sand (SC) and sandy lean clay (CL)		2 SS	4
5	- stiff, gray	9.5	3 SS	9
	ELASTIC SILT (MH) - Mostly low to medium plasticity fines, some fine sand, very moist to wet, black, very soft, roots		4 SS	WOH
10		4.5		
			5 SS	1
15		-0.5		
	POORLY GRADED SAND WITH SILT (SP-SM) - Mostly fine sand, few to some low plastic fines, wet, gray, medium dense, occasionally grades to silty sand (SM)		6 SS	15
20		-5.5		
	- very dense		7 SS	57
25		-10.5		
			8 SS	79
30		-15.5		

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

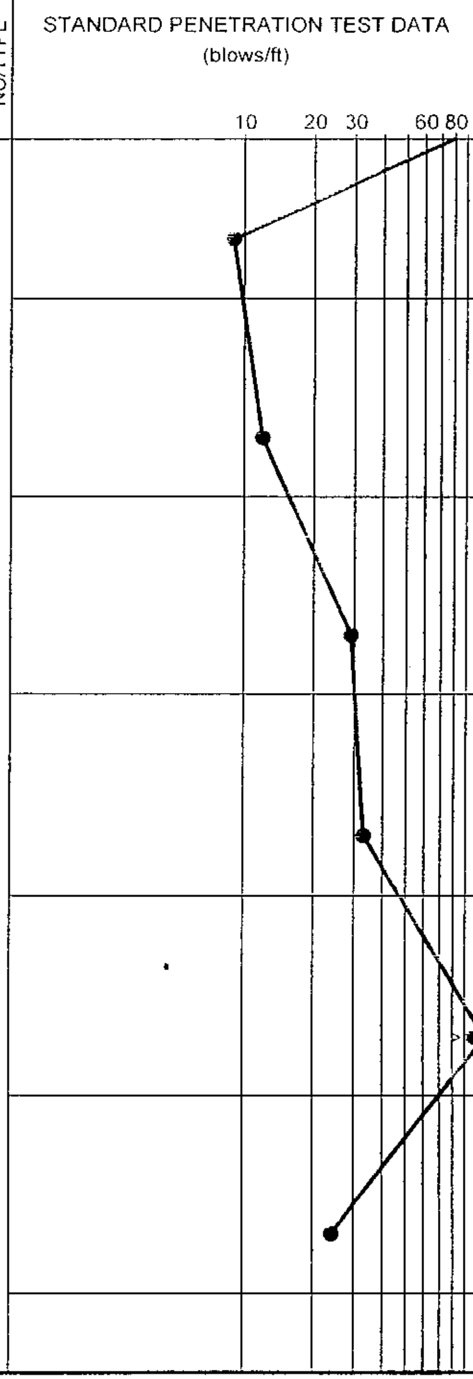
S&ME

PROJECT: Highway 501, Myrtle Beach, South Carolina, 1611-990-401	**BORING LOG B-208**

DATE STARTED: 2/17/00	DATE FINISHED: 2/17/00
DRILLING METHOD: Mud Rotary	
CASING LENGTH:	DRILLER: Stone
WATER LEVEL: 1.5 feet at time of boring, 0.5 feet at 24 hours	

NOTES: Approximate Station 481+80, N272000, E2624790, Frontage Road 6F

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (feet-MSL)	SAMPLE NO/TYPE	N VALUE
	ELASTIC SILT (MH) - Mostly low to medium plasticity fines, some fine sand, very moist to wet, black, stiff		9 SS	9
35	**SANDY LEAN CLAY (CL)** - Mostly medium plasticity fines, some fine sand, interlayered with 0.5 ft. to 1.5 ft. very hard limestone lenses, very moist, dark gray, stiff, reacts with 10% HCl, commonly grades to a clayey sand (SC)	-20.5		
	LIMESTONE - Limestone lense, 0.5 ft. thick, gray, very hard			
	SANDY LEAN CLAY (CL) - Same as above, (35 ft. to 36 ft.)		10 SS	12
40		-25.5		
	LIMESTONE - Limestone lense, 1.5 ft. thick, gray, very hard			
	SANDY LEAN CLAY (CL) - Same as above, hard, (35 ft. to 36 ft.)		11 SS	29
45		-30.5		
	LIMESTONE - Limestone lense, 1.8 ft. thick, gray, very hard			
	SANDY LEAN CLAY (CL) - Same as above, hard, (35 ft. to 36 ft.)		12 SS	33
50	**LIMESTONE** - Limestone lense, 2.0 ft. thick, gray, very hard	-35.5		
	SANDY LEAN CLAY (CL) - Same as above, (35 ft. to 36 ft.), more sand from 52 ft. to 60 ft. in clay layers		13 SS	50/ 2"
	LIMESTONE - Limestone lense, 1.0 ft. thick, gray, very hard			
55	**SANDY LEAN CLAY (CL)** - Same as above, hard, (35 ft. to 36 ft.)	-40.5		
	LIMESTONE - Limestone lense, 1.5 ft. thick, gray, very hard			
	SANDY LEAN CLAY (CL) - Same as above, hard, (35 ft. to 36 ft.)		14 SS	24
60	- Boring terminated at 60 feet.	-45.5		



BORING LOG 99-401.GPJ S&ME.GDT 3/1/00

1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S&ME

Appendix B – 2

Soil Test Borings

1999

PROJECT: Highway 501 Preliminary Geotech Myrtle Beach, SC 1611-99-401	**BORING LOG B-117**

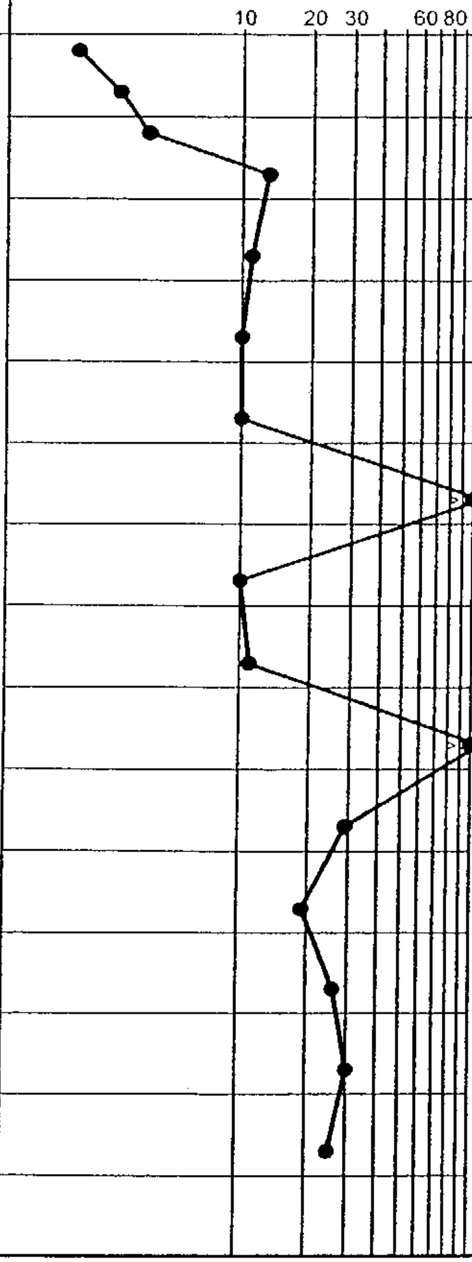
DATE STARTED: 3/13/99	DATE FINISHED: 3/14/99
DRILLING METHOD: Mud Rotary	
CASING LENGTH:	DRILLER: Tony
WATER LEVEL: 3.2 inches at time of boring	

NOTES: Approximate Station 543+30 150'L, N272638 E2625188

DEPTH (feet)	MATERIAL DESCRIPTION
0–8	**ORGANIC CLAY (OH)** - mostly high plasticity fines and organics, dark brown and black, very soft to soft.
8–21	**CLAYEY SAND (SC)** - mostly fine to medium sand, some medium plasticity fines, gray, medium dense. - becoming fine.
21–32	**SILTY SAND (SM)** - mostly fine to medium sand, some low plasticity fines and shells, green-gray, loose. - very dense.
32–70	**SANDY ELASTIC SILT (MH)** - mostly medium plasticity fines, dome fine sand, green-gray, stiff. (PP=0.25 TSF) - very stiff to very hard. - trace sand. (PP=1.75 TSF)
70	Boring terminated at 70 feet.

DEPTH (feet)	ELEVATION (feet-MSL)	SAMPLE NO/TYPE	N VALUE
		1 SS	2
		2 SS	3
5	10.4	3 SS	4
10	5.4	4 SS	13
15	0.4	5 SS	11
20	-4.6	6 SS	10
25	-9.6	7 SS	10
30	-14.6	8 SS	50/ 0"
35	-19.6	9 SS	10
40	-24.6	10 SS	11
45	-29.6	11 SS	50/ 3"
50	-34.6	12 SS	29
55	-39.6	13 SS	19
60	-44.6	14 SS	26
65	-49.6	15 SS	30
70	-54.6	16 SS	25

STANDARD PENETRATION TEST DATA (blows/ft)



1 BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

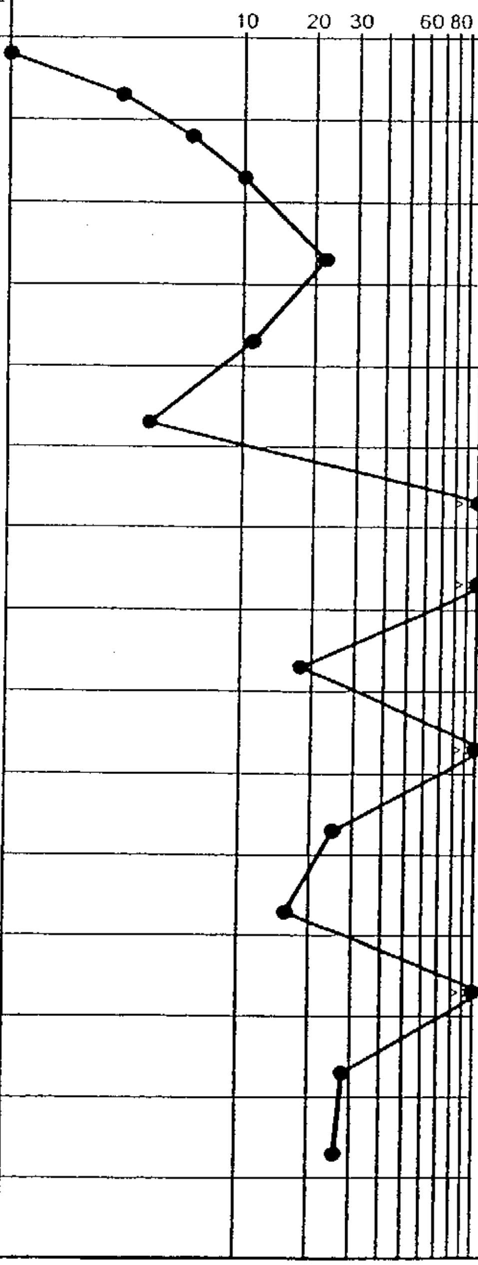
S&ME
134 Suber Road
Columbia, SC 29210

PROJECT: Highway 501 Preliminary Geotech Myrtle Beach, SC 1611-99-401	**BORING LOG B-124**

DATE STARTED: 3/12/99	DATE FINISHED: 3/12/99	NOTES: Approximate Station 544+80 140'R, N272369 E2625042
DRILLING METHOD:		
CASING LENGTH:	DRILLER:	
WATER LEVEL:		

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (feet-MSL)	SAMPLE NO/TYPE	N VALUE
0	TOPSOIL - APPROXIMATELY 3 INCHES			
	CLAYEY SAND (SC) - mostly fine sand, some high plasticity fines, very loose.		1 SS	0
			2 SS	3
5	- becoming brown loose.	10.2	3 SS	6
10		5.2	4 SS	10
	POORLY GRADED SAND WITH CLAY (SP-SC) - mostly fine to coarse sand, some medium plasticity fines, gray, medium dense.		5 SS	22
15		0.2		
			6 SS	11
20		-4.8		
	SILTY SAND (SM) - mostly fine to coarse sand, some low plasticity fines and shells, dark green-gray, loose.		7 SS	4
25		-9.8		
	SANDY ELASTIC SILT (MH) - mostly medium plasticity fines, some fine sand, dark green-gray, very hard. (PP=0.25 TSF)		8 SS	50/ 5"
30		-14.8		
	- interlayered very stiff to hard seams.		9 SS	50/ 4"
35		-19.8		
			10 SS	18
40	- (PP=0.75 TSF)	-24.8		
	- w/ limestone fragments.		11 SS	50/ 1"
45		-29.8		
			12 SS	25
50		-34.8		
			13 SS	16
55	- (PP=1.25 TSF)	-39.8		
			14 SS	50/ 5"
60		-44.8		
	- trace sand.		15 SS	28
65		-49.8		
	- (PP=1.5 TSF)		16 SS	26
70	Boring terminated at 70 feet.	-54.8		

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S&ME
134 Suber Road
Columbia, SC 29210

Appendix B – 3

Soil Test Borings

1993

PROJECT: **US 501 PHASE VI**
MYRTLE BEACH, SOUTH CAROLINA
1131-93-431

BORING LOG B-23

DATE STARTED: **11/17/93** DATE FINISHED: **11/17/93**

DRILLING METHOD: **MUD ROTARY**

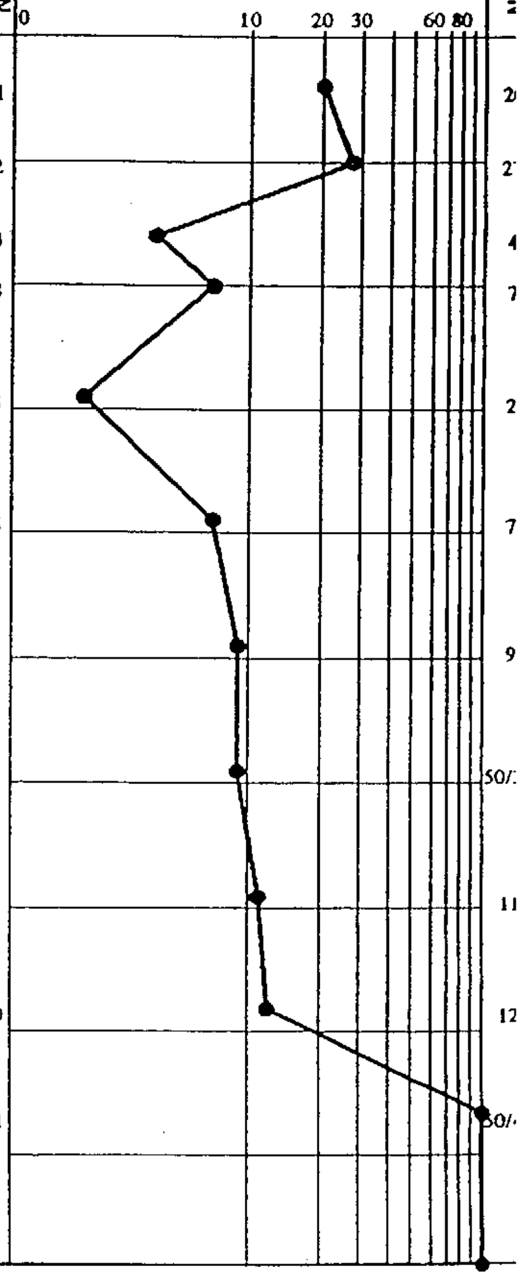
CASING LENGTH: DRILLER: **NORWOOD**

WATER LEVEL: **4.3 FEET AFTER 24 HOURS**

NOTES: **N 273,826.42 E 2,625,250.01**
EL 20.3

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N
	Brown, medium dense, medium to fine SAND, some fine gravel, little wood (FILL)(SP)			
			SS-1	2
5	Brown, medium dense, medium to fine SAND, little silt (FILL)(SP-SM)	15.3	SS-2	2
	Yellowish gray, soft, silty CLAY, little (-) fine sand (CL)		SS-3	4
10	Gray, loose, fine SAND, trace silt, wood (SP-SM)	10.3	SS-4	7
	Top 18" gray, soft organic clayey SILT, trace silt, wood (OH),(pp=0.25 tsf)			
15	Bottom 6" gray, very loose, fine SAND and organic clayey silt (SM)	5.3	SS-5	2
20	Brownish gray, loose, medium to fine SAND, trace (+) silt, little wood (SP)	0.3	SS-6	7
25	Gray, loose, calcareous, medium to fine SAND and fine gravel, little cemented fine sand, shells (SP)	-4.7	SS-7	9
30	Same	-9.7	SS-8	50/
35	Dark gray, medium dense, fine SAND, some (+) clayey silt (SM)	-14.7	SS-9	11
40	Same, cemented fine SAND at tip (SM)	-19.7	SS-10	12
	Dark gray, very dense, fine SAND and clayey silt, cemented fine SAND at tip (SM/ML)		SS-11	50/
45		-24.7		
	Same			

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

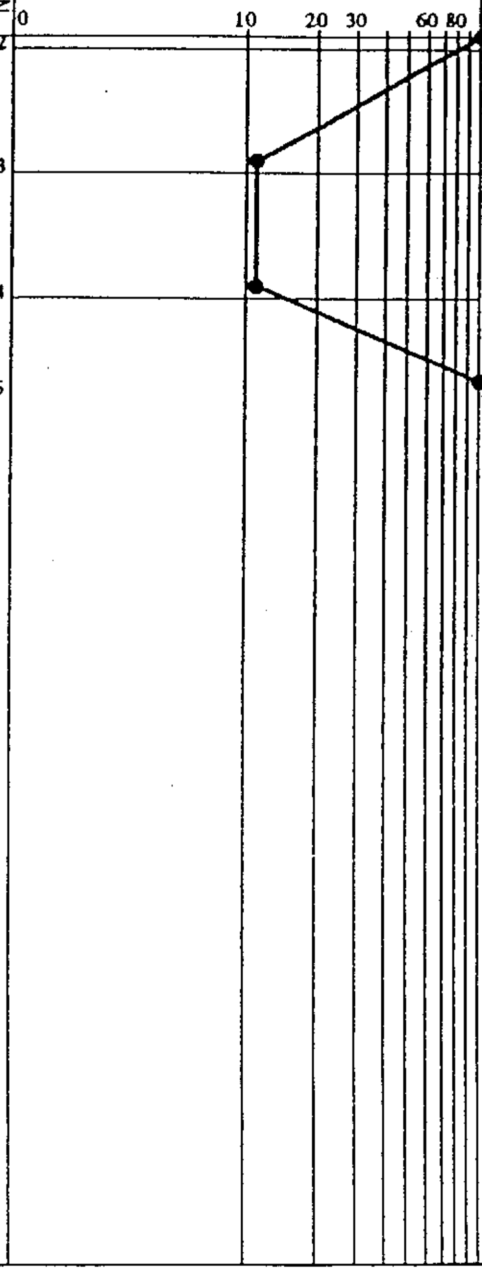
S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

PROJECT: US 501 PHASE VI MYRTLE BEACH, SOUTH CAROLINA 1131-93-431	BORING LOG B-23

ATE STARTED: 11/17/93	DATE FINISHED: 11/17/93	NOTES: N 273,826.42 E 2,625,250.01 EL 20.3
DRILLING METHOD: MUD ROTARY		
CASING LENGTH:	DRILLER: NORWOOD	
WATER LEVEL: 4.3 FEET AFTER 24 HOURS		

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
50			-29.7	SS-12	50/4"
		Dark gray, medium dense, fine SAND and clayey silt (SM/ML)			
55			-34.7	SS-13	11
		Same			
60			-39.7	SS-14	11
		Dark gray, hard SILT & clay, some (+) fine sand, cemented fine sand and silt (ML)		SS-15	50/4"
		BOTTOM OF HOLE AT 63.33 FEET			

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

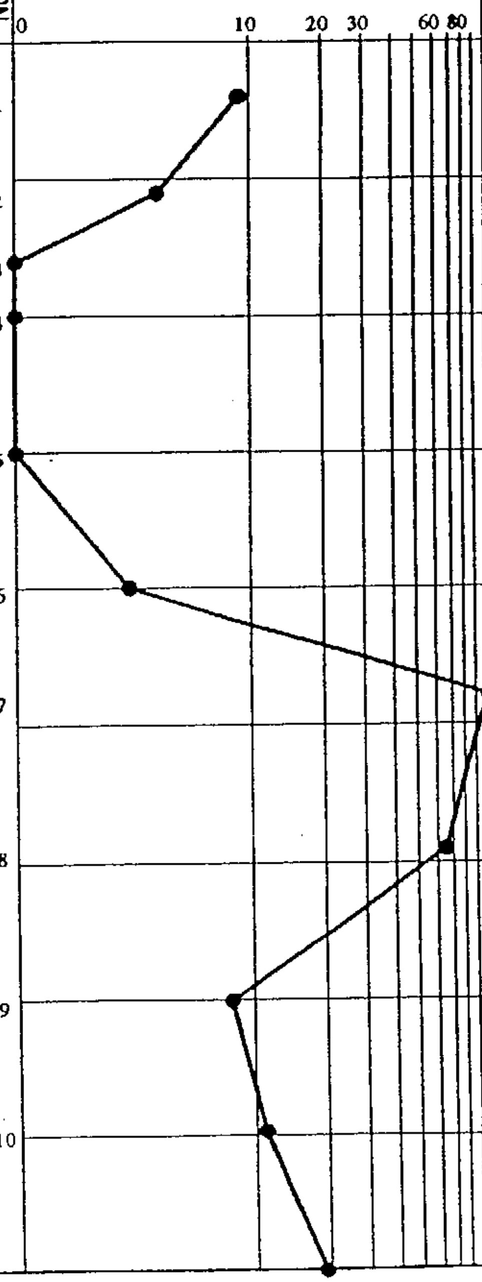
S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

PROJECT: US 501 PHASE VI MYRTLE BEACH, SOUTH CAROLINA 1131-93-431	**BORING LOG PB-24**

\TE STARTED: 11/19/93	DATE FINISHED: 11/19/93	NOTES: N 273,768.25 E 2,625,333.21 EL 14.5
DRILLING METHOD: MUD ROTARY		
CASING LENGTH:	DRILLER: NORWOOD	
WATER LEVEL: AT SURFACE AT TIME OF BORING		

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
	Top 6" dark brown, very soft, MUCK, leaves Bottom 12" brown, loose, medium to fine SAND, trace (+) silt (SP-SM)		SS-1	9
5	Same and wood	9.5	SS-2	4
	Gray, very soft, slightly organic silty CLAY (CL)		SS-3	WOH
10	Top 8" Same Bottom gray, loose, fine SAND and silty clay (SC)	4.5	SS-4	WOH
15	Brown, loose, medium to fine SAND, trace fine gravel, trace silt (SP)	-0.5	SS-5	1
20	Gray, very loose, calcareous, coarse to fine SAND and fine gravel, little shells (SP)	-5.5	SS-6	3
25	Dark gray, very dense, fine SAND, some (+) silt, cemented fine sand and silt at tip (SM)	-10.5	SS-7	50/5"
30	Dark gray, very dense, fine SAND, little clayey silt, some (+) cemented fine sand & silt (SM)	-15.5	SS-8	64
35	Dark gray, loose, fine SAND and clayey silt (SM)	-20.5	SS-9	8
40	Dark gray, medium stiff, clayey SILT and fine sand, cemented fine SAND at tip (ML)	-25.5	SS-10	11
	Dark gray, stiff, clayey SILT and fine sand (ML)			

STANDARD PENETRATION TEST DATA (blows/ft)



1. *BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
2. *PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

PROJECT: US 501 PHASE VI MYRTLE BEACH, SOUTH CAROLINA 1131-93-431	**BORING LOG PB-24**

DATE STARTED: 11/19/93 DATE FINISHED: 11/19/93

DRILLING METHOD: MUD ROTARY

CASING LENGTH: DRILLER: NORWOOD

WATER LEVEL: AT SURFACE AT TIME OF BORING

NOTES: N 273,768.25 E 2,625,333.21 EL 14.5

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
			SS-11	19
50	Dark gray, stiff, clayey SILT and fine sand, cemented fine sand at tip (pp=1.5 Tsf)(ML)	-35.5	SS-12	50/4"
55	Dark gray, very stiff, SILT & clay, some (+) fine sand (pp=2.0 Tsf)(ML)	-40.5	SS-13	24
60	Dark gray, very stiff, SILT & clay, some (+) fine sand (ML)	-45.5	SS-14	21
	BOTTOM OF HOLE AT 60 FEET			

STANDARD PENETRATION TEST DATA (blows/ft): 0, 10, 20, 30, 60, 80

1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

PROJECT: US 501 PHASE VI MYRTLE BEACH, SOUTH CAROLINA 1131-93-431	BORING LOG PB-25

DATE STARTED: 11/18/93 | DATE FINISHED: 11/19/93

NOTES: N 273,848.65 E 2,625,326.15 EL 15.6

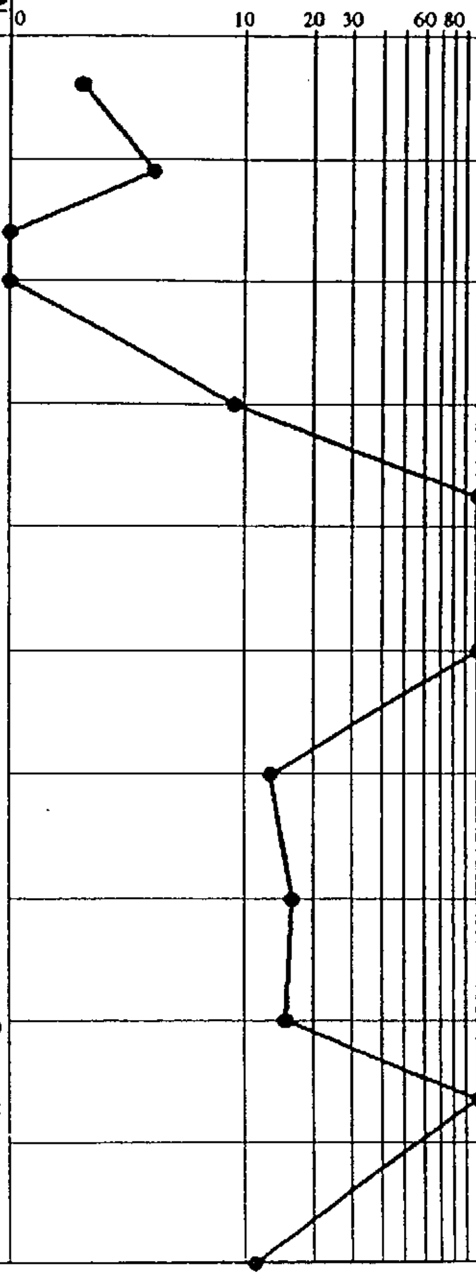
DRILLING METHOD: MUD ROTARY

CASING LENGTH: | DRILLER: NORWOOD

WATER LEVEL: AT SURFACE AT TIME OF BORING

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
0–5	Top 6" dark brown, very soft, MUCK and leaves Bottom 6" brown, very loose, medium to fine SAND, little (+) silt (SP-SM)		SS-1	2
	Same, some wood	10.6		
5	Gray, very soft, organic clayey SILT, trace fine sand, trace wood (PP=0.0 Tsf)(OH)		SS-2	4
	Gray, very soft, organic clayey SILT, some (-) fine sand, little wood (PP=0.0 Tsf)(OH)		SS-3	1/12"
10		5.6	SS-4	1
	Brown, loose, medium SAND, trace silt (SP)			
15		0.6	SS-5	9
	Gray, very dense, calcareous, medium SAND and fine gravel, little cemented fine sand, trace silt (SP)		SS-6	50/3"
20		-4.4		
	Dark gray, very dense, fine SAND, some (+) silt, little fine gravel (SM)			
25		-9.4	SS-7	50/1"
	Dark gray, medium dense, fine SAND, some (+) clayey silt (SM)			
30		-14.4	SS-8	13
	Dark gray, medium dense, fine SAND, some (+) clayey silt, little (+) fine gravel (SM)			
35		-19.4	SS-9	16
	Dark gray, medium dense, fine SAND and clayey silt (SM)			
40		-24.4	SS-10	15
	Gray, very dense, cemented fine SAND and silt (SM/ML)		SS-11	50/3"
45		-29.4		
	Dark gray, stiff, clayey SILT and fine sand (ML)			

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

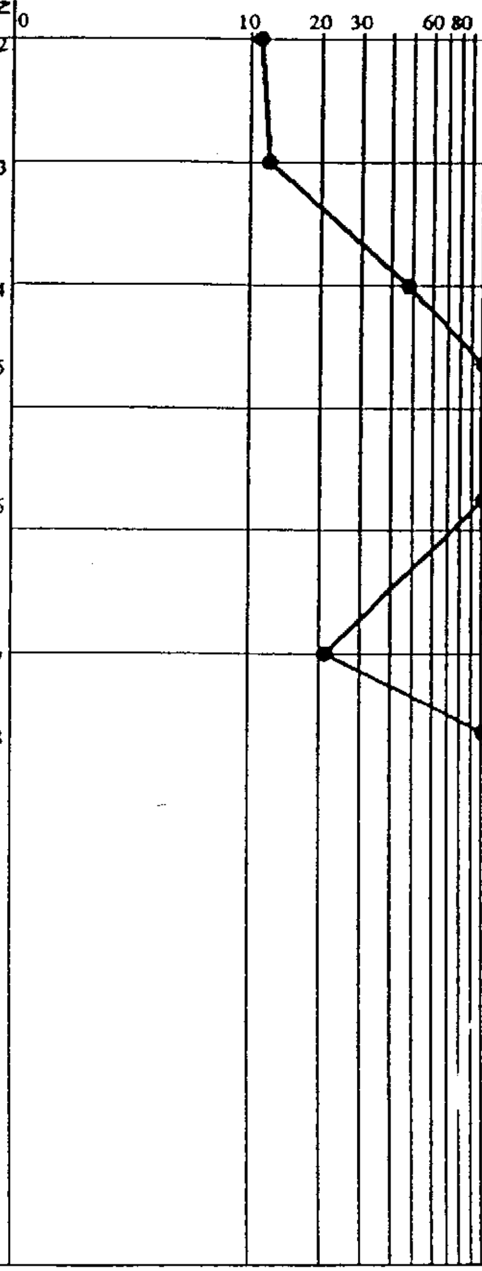
PROJECT: US 501 PHASE VI MYRTLE BEACH, SOUTH CAROLINA 1131-93-431	BORING LOG PB-25

DATE STARTED: 11/18/93	DATE FINISHED: 11/19/93
DRILLING METHOD: MUD ROTARY	
CASING LENGTH:	DRILLER: NORWOOD
WATER LEVEL: AT SURFACE AT TIME OF BORING	

NOTES: N 273,848.65 E 2,625,326.15
EL 15.6

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
	dark gray, stiff, SILT & clay, some fine sand (ML)		SS-12	11
55		-39.4	SS-13	12
	dark gray, very stiff, clayey SILT, some (+) fine sand (PP=2.0 Tsf)(ML)			
60		-44.4	SS-14	47
	Gray, very dense, cemented fine SAND & silt (SM/ML)		SS-15	50/2"
65		-49.4		
	Dark gray, hard clayey SILT, some (+) fine sand, gray cemented fine SAND at tip (ML)		SS-16	50/3"
70		-54.4		
	Dark gray, very stiff, SILT & clay, some fine sand (PP=2.0 Tsf)(ML)			
75		-59.4	SS-17	21
	Gray, very dense, cemented fine SAND and silt (SM/ML)		SS-18	50/2"
	BOTTOM OF HOLE AT 78.17 FEET			

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

BORING LOG PB-26

PROJECT:	US 501 PHASE VI MYRTLE BEACH, SOUTH CAROLINA 1131-93-431
DATE STARTED: 10/14/93	DATE FINISHED: 10/14/93
DRILLING METHOD: MUD ROTARY	
CASING LENGTH:	DRILLER: NORWOOD
WATER LEVEL: 5 FEET AT TIME OF BORING	

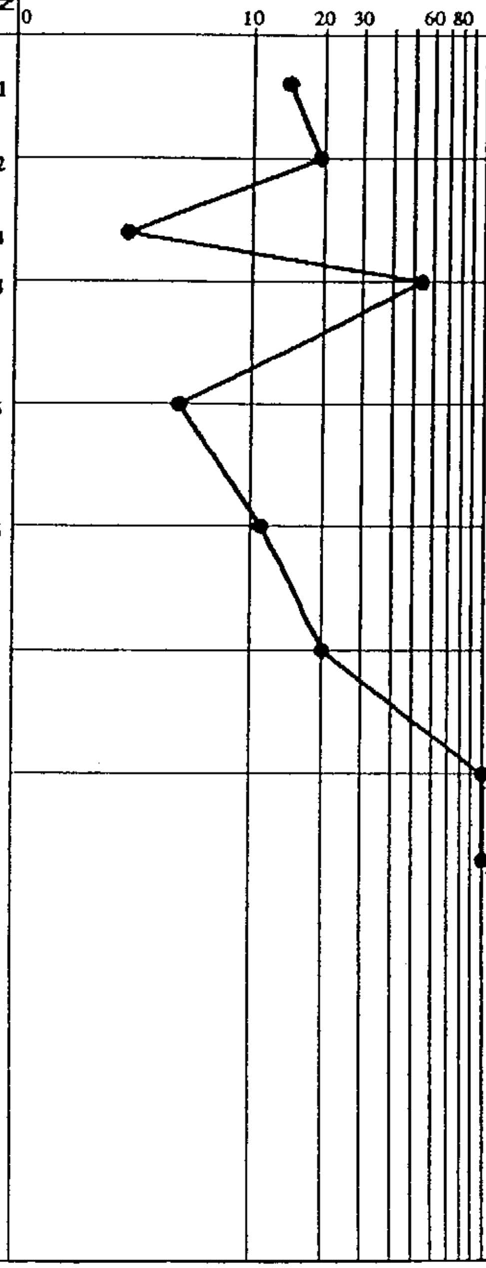
NOTES: N 273,198.68 E 2,625,626.05
EL 19.8

DEPTH (feet)	MATERIAL DESCRIPTION
0–8	Dark brown, medium dense, medium to fine SAND, little (-) silt, trace (-) roots, trace (+) coarse gravel (Topsoil)(FILL)(SM) Same + piece of wood (FILL)(SM) Dark brown, very loose, fine SAND, little (-) silt (FILL)(SP-SM)
8–12	Light brown, very dense, medium to fine SAND, trace (-) silt (SP)
12–17	Gray, soft, silty CLAY, highly plastic, trace (+) fine sand, trace (+) wood (CH)
17–21.5	Greenish gray, medium dense, calcareous coarse SAND, little (+) fine gravel,trace (-) silt, little (-) shells (SM)
21.5–27	Gray, medium dense, medium to fine SAND, little (+) silt, trace (+) shells (SM)
27–33.5	Gray, very dense, cemented fine SAND and silt, little (-) fine gravel (at tip)(SM/ML) Hard drilling at 29' Same

BOTTOM OF HOLE AT 33.5 FEET

SAMPLE NO/TYPE	ELEVATION (ft-MSL)	N VALUE
SS-1		14
SS-2	14.8	19
SS-3		3
SS-4	9.8	53
SS-5	4.8	5
SS-6	-0.2	11
SS-7	-5.2	20
SS-8	-10.2	50/1
SS-9		50/6

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

PROJECT: **US 501 PHASE VI**
MYRTLE BEACH, SOUTH CAROLINA
1131-93-431

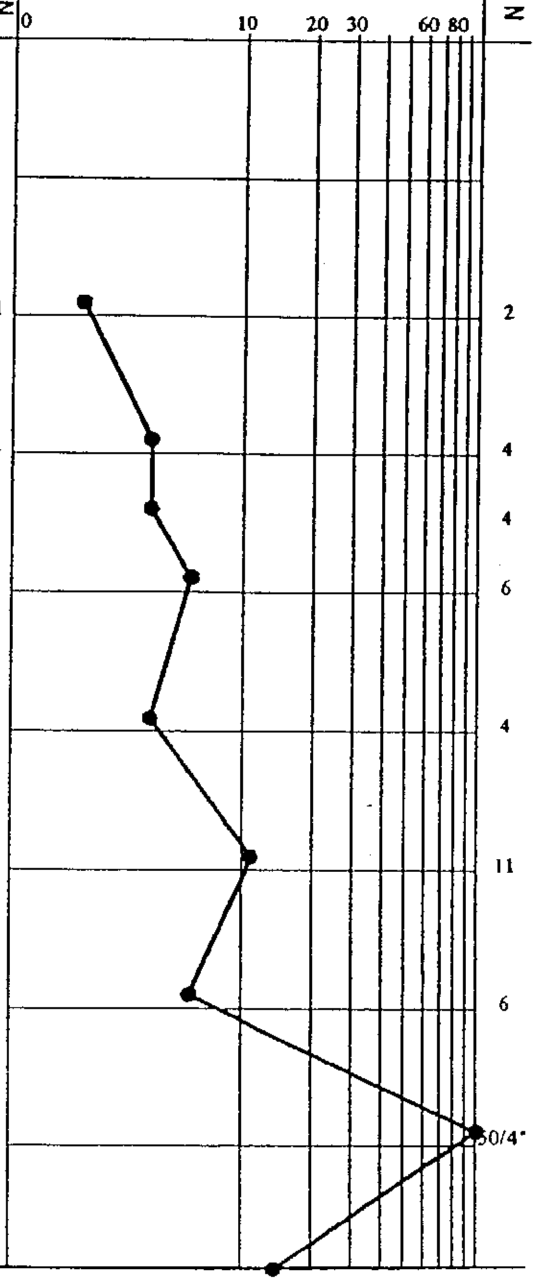
BORING LOG PB-27

ATE STARTED: **11/16/93**	DATE FINISHED: **11/16/93**
DRILLING METHOD: **MUD ROTARY**	
CASING LENGTH:	DRILLER: **SIMRIL**
WATER LEVEL:	

NOTES: **N 273,177.83 E 2,625,656.78**
EL 12.5

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
	TOP OF BRIDGE			
5	TOP OF WATER	15.0		
10	Brown, very loose, medium to fine SAND, trace (-) silt, little wood (SP)	10.0	SS-1	2
15	Top 6" brown, very loose, medium to fine SAND, little silt (SM) Bottom gray, very soft, organic clayey SILT, little fine sand (PP=0.25 tsf)(OH)	5.0	SS-2	4
	Top 12" Same (0.5 tsf pp)(OH) Bottom 12" gray, very loose, calcareous, coarse to medium SAND (SM), some fine gravel, little silt, little shells		SS-3	4
20	Same	0.0	SS-4	6
25	Same	-5.0	SS-5	4
30	Dark gray, medium dense, fine SAND, little (+) clayey silt, little cemented fine sand (SM)	-10.0	SS-6	11
35	Same	-15.0	SS-7	6
40	Dark gray, very dense, fine SAND and clayey silt, trace (-) cemented fine sand (SM/ML)	-20.0	SS-8	50/4"

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

PROJECT: US 501 PHASE VI
MYRTLE BEACH, SOUTH CAROLINA
1131-93-431

BORING LOG PB-27

DATE STARTED: 11/16/93 | DATE FINISHED: 11/16/93

NOTES: N 273,177.83 E 2,625,656.78
EL 12.5

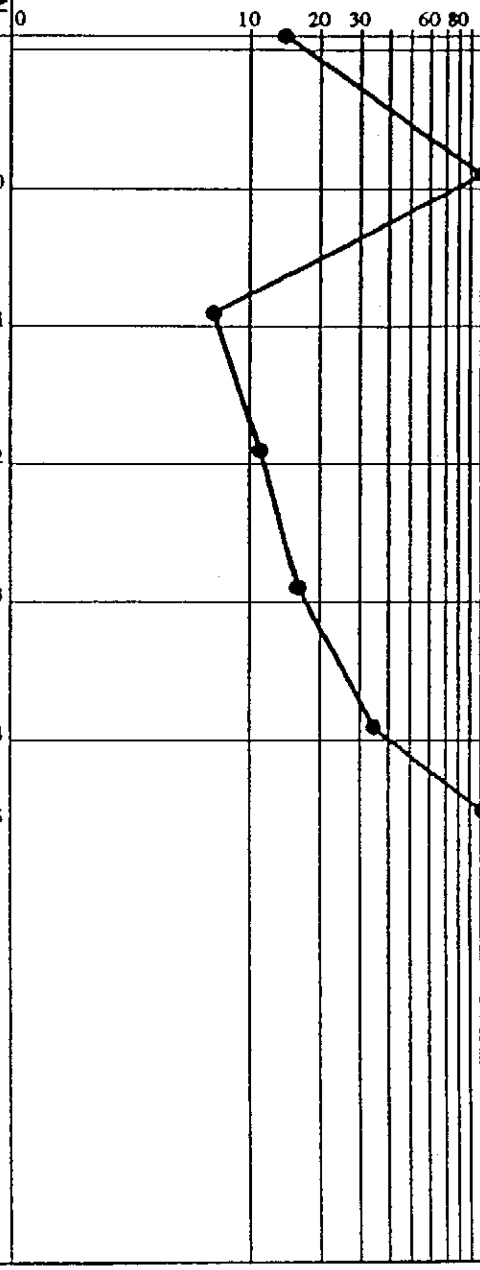
DRILLING METHOD: MUD ROTARY

CASING LENGTH: | DRILLER: SIMRIL

WATER LEVEL:

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
45	Same	-25.0	SS-9	14
50	Dark gray, very stiff, clayey SILT and fine sand, trace cemented fine sand and silt (ML)	-30.0	SS-10	50/5
55	Same(ML)	-35.0	SS-11	7
60	Same	-40.0	SS-12	11
65	Dark gray, very stiff, SILT and clay, some fine sand (pp=2.0 tsf)(ML)	-45.0	SS-13	16
70	Same (pp=1.5 tsf)	-50.0	SS-14	34
	Spoon bouncing, no recovery		SS-15	50/0'
	BOTTOM OF HOLE AT 72.5 FEET			

STANDARD PENETRATION TEST DATA (blows/ft)



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S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

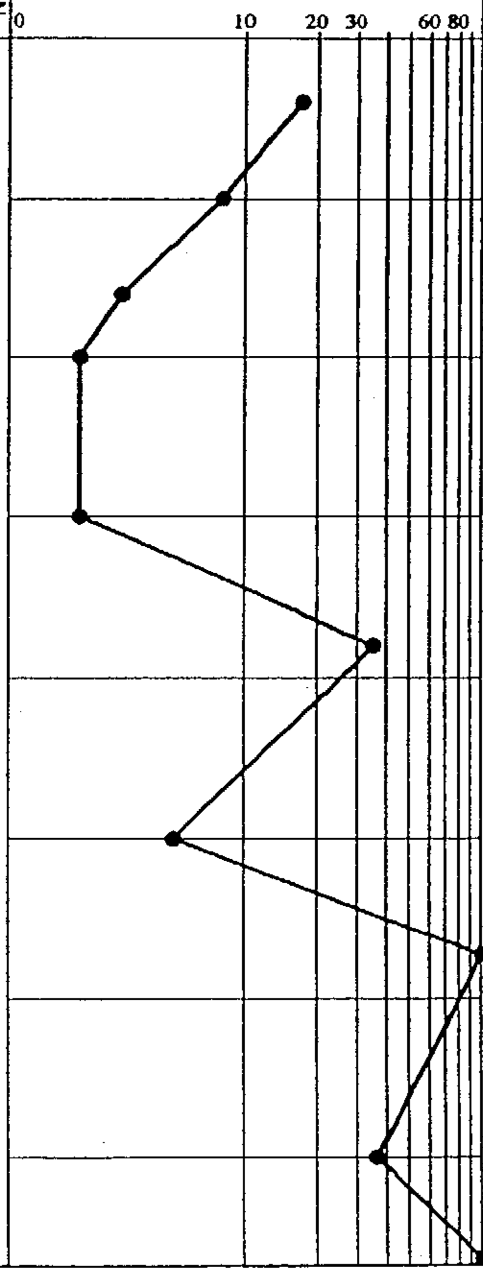
PROJECT: US 501 PHASE VI MYRTLE BEACH, SOUTH CAROLINA 1131-93-431	**BORING LOG PB-28**

ATE STARTED: 10/14/93	DATE FINISHED: 10/14/93
DRILLING METHOD: MUD ROTARY	
CASING LENGTH:	DRILLER: NORWOOD
WATER LEVEL: 5 FEET AT TIME OF BORING	

NOTES: N 273,171.90 E 2,625,688.11
EL 19.8

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
	Brown, medium dense, medium to fine SAND, little (-) roots, trace (+) medium gravel, trace (+) silt (TOPSOIL)(SP-SM)		SS-1	17
5	Dark brown, loose, medium to fine SAND, little (-) silt (FILL)(SP-SM)	14.8	SS-2	8
	Top 12" Same Bottom 12" dark brown, soft, PEAT and fine SAND, roots, slight organic odor		SS-3	3
10	Gray, very loose, fine SAND, some (+) clayey silt, wood, roots (SM)	9.8	SS-4	2
	Top 18" Same			
15	Bottom 4" gray, very soft, silty CLAY, trace (-) wood (CH)	4.8	SS-5	2
	Piston sample		U-1	
	Top 6" light brown, loose, coarse to medium SAND Bottom 18" greenish gray, dense, calcareous medium SAND, little (-) shells (SP)		SS-6	35
20		-0.2		
	Hard Drilling at 21' - 22.5'			
25	Gray, loose, calcareous fine SAND, little silt (SM)	-5.2	SS-7	5
	Gray, very dense, cemented fine SAND and silt, some fine gravel (SM)		SS-8	50/1"
30		-10.2		
35	Gray, dense, fine SAND, some (+) clayey silt, trace (+) coarse gravel (SM) Drilling at 36.5'	-15.2	SS-9	36

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

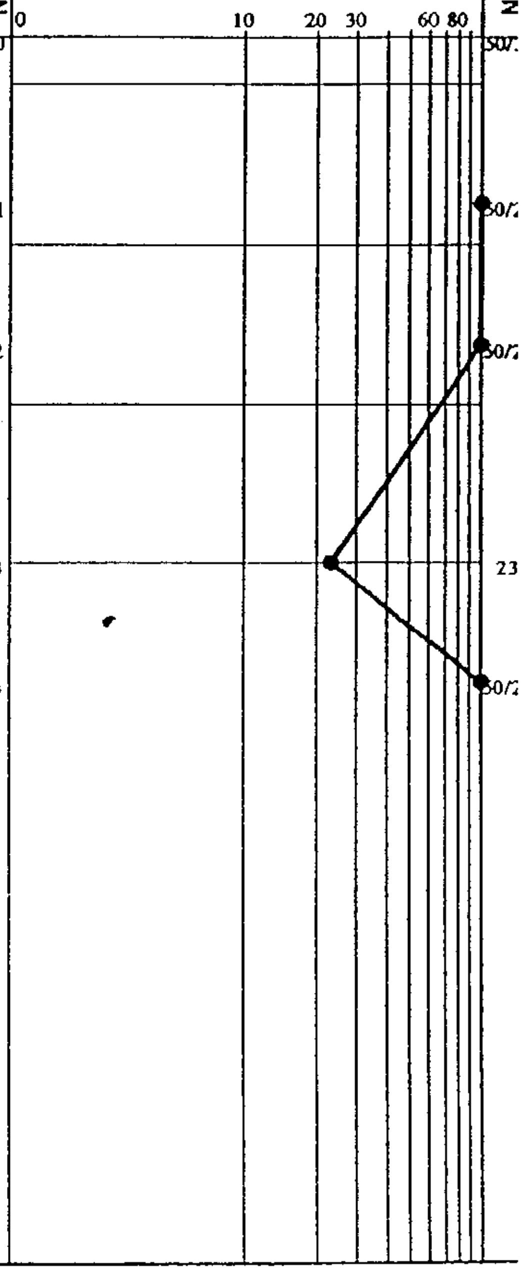
S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

PROJECT: US 501 PHASE VI MYRTLE BEACH, SOUTH CAROLINA 1131-93-431	**BORING LOG PB-28**

DATE STARTED: 10/14/93	DATE FINISHED: 10/14/93	NOTES: N 273,171.90 E 2,625,688.11 EL 19.8	
DRILLING METHOD: MUD ROTARY			
CASING LENGTH:	DRILLER: NORWOOD		
WATER LEVEL: 5 FEET AT TIME OF BORING			

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
		Gray, very dense, cemented fine SAND and silt (SM/ML)		SS-10	50/.
40			-20.2		
		Gray, very dense, cemented fine SAND and silt(SM/ML)		SS-11	50/.
45			-25.2		
		Same		SS-12	50/.
50			-30.2		
55		Gray, very stiff, clayey SILT and fine sand, trace (-) fine gravel (ML)	-35.2	SS-13	23
		Gray, very dense, cemented, fine SAND and silt, trace (+) fine gravel (SM/ML)		SS-14	50/.
		BOTTOM OF HOLE AT 58.7 FEET			

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

PROJECT: US 501 PHASE VI
MYRTLE BEACH, SOUTH CAROLINA
1131-93-431

BORING LOG B-29

DATE STARTED: 11/8/93 DATE FINISHED: 11/9/93

NOTES: N 271,993.30 E 2,624,800.40
EL 13.6

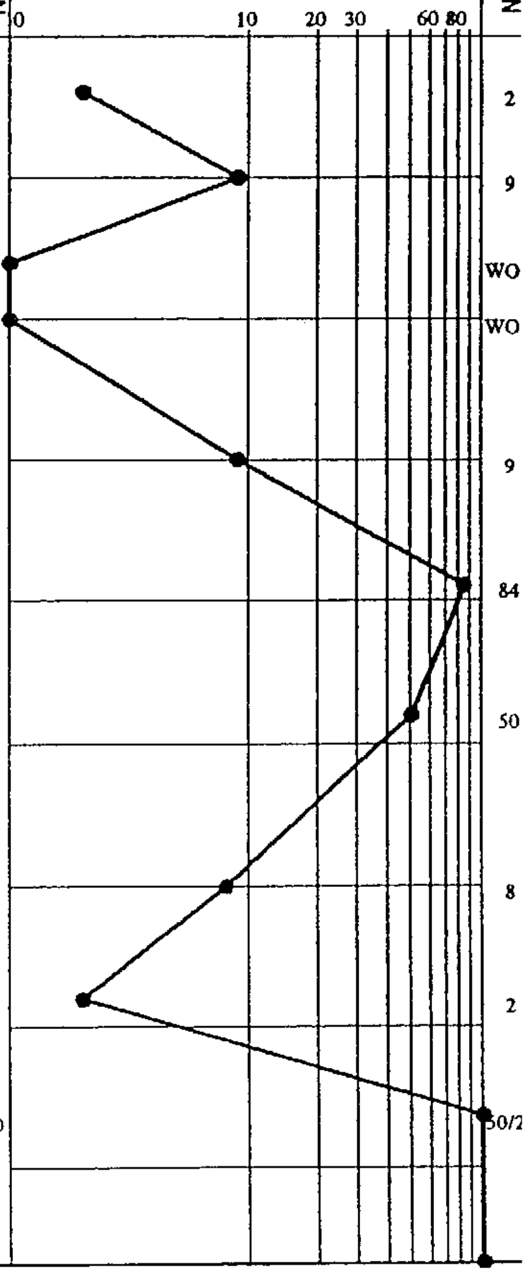
DRILLING METHOD: MUD ROTARY

CASING LENGTH: DRILLER: NORWOOD

WATER LEVEL: 0 FEET AT TIME OF BORING

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
	Dark brown, very soft, MUCK, little (+) fine sand (PT)		SS-1	2
5	Gray, loose, fine SAND, little clayey silt (SM)	8.6	SS-2	9
	Gray, very soft, silty CLAY, trace peat, trace fine sand, slightly organic (CH)		SS-3	WO
10	Same	3.6	SS-4	WO
	Top 12" gray, soft, silty CLAY, little peat, trace sand, wood (CH)			
15	Bottom 12" brown, medium dense, fine SAND (SP)	-1.4	SS-5	9
20	Tan, very dense, fine SAND, trace (-) silt (SP)	-6.4	SS-6	84
	Same		SS-7	50
25		-11.4		
	Top 12" Same			
30	Bottom 12" Gray, soft, organic clayey SILT (pp=0.25 tsf)(OH)	-16.4	SS-8	8
	NO RECOVERY			
	Dark gray, very soft, organic clayey SILT, some (-) fine sand (OH)		SS-9	2
35		-21.4		
	Gray, very dense, cemented fine SAND and silt (SM/ML)		SS-10	50/2
40		-26.4		

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

PROJECT: US 501 PHASE VI MYRTLE BEACH, SOUTH CAROLINA 1131-93-431	**BORING LOG B-29**

ATE STARTED: 11/8/93	DATE FINISHED: 11/9/93
DRILLING METHOD: MUD ROTARY	
CASING LENGTH:	DRILLER: NORWOOD
WATER LEVEL: 0 FEET AT TIME OF BORING	

NOTES: N 271,993.30 E 2,624,800.40
EL 13.6

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	STANDARD PENETRATION TEST DATA (blows/ft)	N VALUE
		Same		SS-11		50/5"
45			-31.4			
50		Dark gray, stiff, clayey SILT and fine sand (pp=0.5 tsf)(ML)	-36.4	SS-12		14
		Dark gray, stiff, clayey SILT and cemented fine sand and silt (ML)		SS-13		50/2"
55			-41.4			
		Same		SS-14		50
		BOTTOM OF HOLE AT 59 FEET				

SPT scale: 0, 10, 20, 30, 60, 80

1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

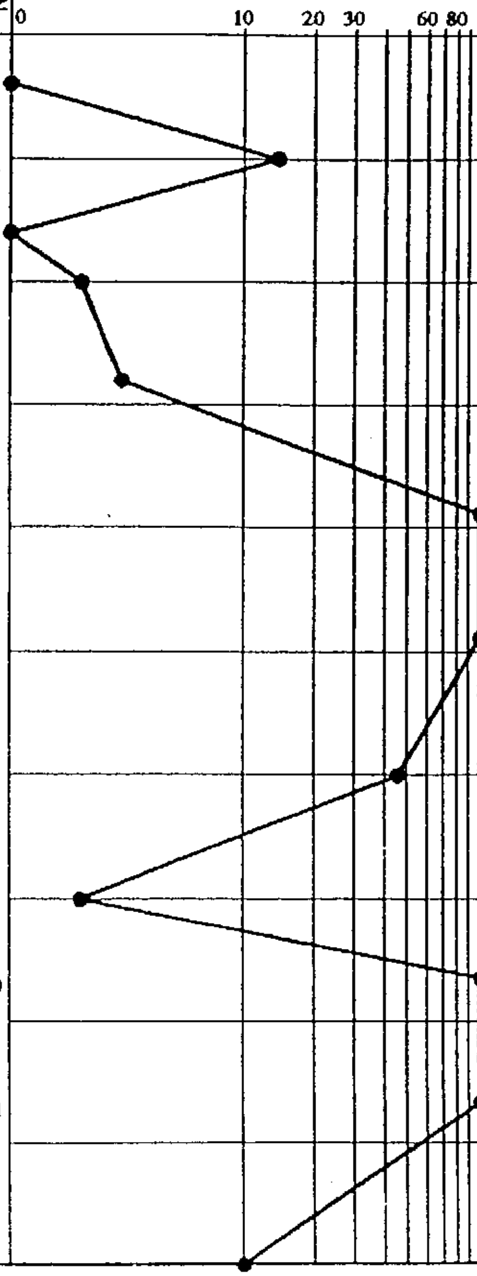
BORING LOG B-30

PROJECT: US 501 PHASE VI, MYRTLE BEACH, SOUTH CAROLINA, 1131-93-431	
DATE STARTED: 11/7/93	DATE FINISHED: 11/8/93
DRILLING METHOD: MUD ROTARY	
CASING LENGTH:	DRILLER: NORWOOD
WATER LEVEL: 0 FEET AFTER 24 HOURS	

NOTES: N 271,948.14 E 2,624,768.04
EL 13.4

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
	Dark brown, very soft, MUCK, roots (PT)		SS-1	WOI
5	Top 4" Same Bottom 10" brown, medium dense, medium to fine SAND, little (-) silt (SM)	8.4	SS-2	14
	Top 6" Same Bottom 10" very soft, gray, silty CLAY, little (+) fine sand (CH)		SS-3	1
10	Gray, very soft, silty CLAY and peat (CH)	3.4	SS-4	2
	Push down easy, Piston Sample		U-1	
15	Dark brown & gray, silty CLAY, little peat, slightly organic, trace sand (CH)	-1.6	SS-5	3
20	Tan, dense, fine SAND (SP)	-6.6	SS-6	50/5"
25	Same	-11.6	SS-7	50/5"
30	Gray, dense, fine SAND, trace silt (SP)	-16.6	SS-8	45
35	Dark gray, very soft, organic clayey SILT, trace (-) fine sand (OH) *slightly light unit weight *no smell	-21.6	SS-9	2
	Gray, very dense, cemented fine SAND and silt (SM/ML)		SS-10	50/3"
40		-26.6		
	Same Very Hard Drilling 44.3 to 45.3		SS-11	50/4"
45		-31.6		
	Gray, medium stiff, clayey SILT, some (+) fine sand (ML)			

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

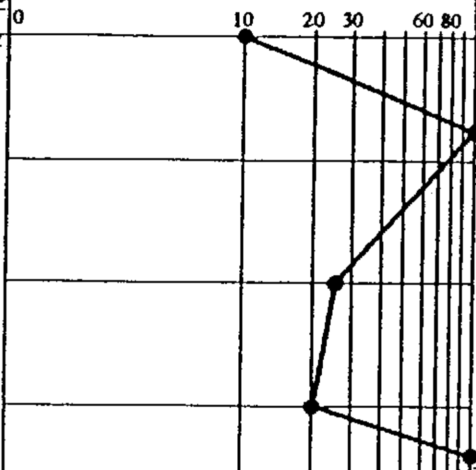
PROJECT: US 501 PHASE VI
MYRTLE BEACH, SOUTH CAROLINA
1131-93-431

BORING LOG B-30

DATE STARTED: 11/7/93	DATE FINISHED: 11/8/93
DRILLING METHOD: MUD ROTARY	
CASING LENGTH:	DRILLER: NORWOOD
WATER LEVEL: 0 FEET AFTER 24 HOURS	

NOTES: N 271,948.14 E 2,624,768.04
EL 13.4

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	STANDARD PENETRATION TEST DATA (blows/ft)	N VALUE
				SS-12		10
55		Gray, hard, clayey SILT, some (+) fine sand, gray, very dense, cemented fine SAND and silt (ML)	-41.6	SS-13		50/4"
60		Gray, very stiff, clayey SILT, some (+) fine sand (ML)	-46.6	SS-14		25
65		Same (PP=1.25 tsf)	-51.6	SS-15		20
		Spoon bouncing, no recovery		SS-16		50/0"
		BOTTOM OF HOLE AT 67 FEET				



1. *BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.*
2. *PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.*

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

PROJECT: US 501 PHASE VI
MYRTLE BEACH, SOUTH CAROLINA
1131-93-431

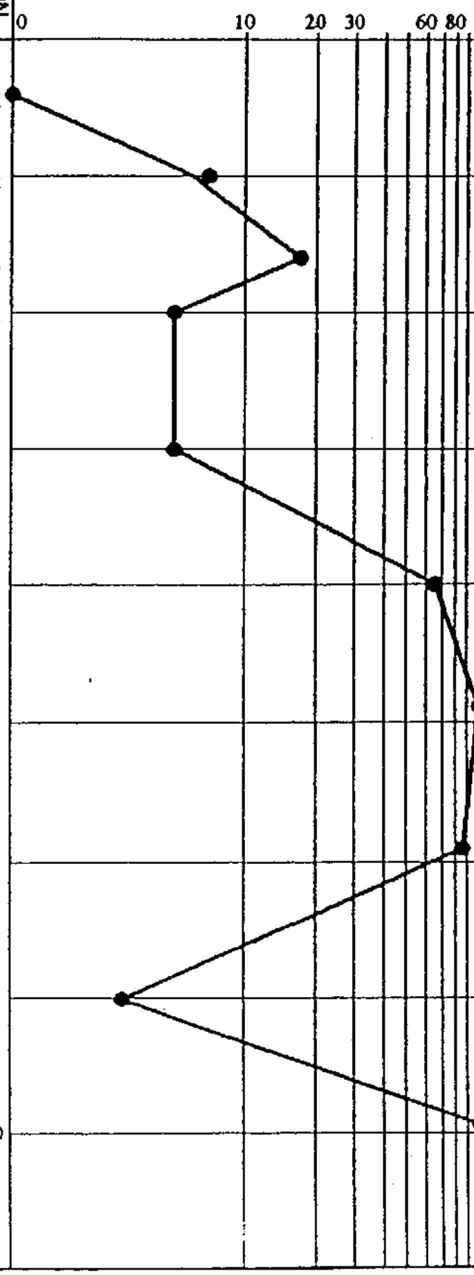
BORING LOG B-32

ATE STARTED: 11/6/93	DATE FINISHED: 11/7/93
DRILLING METHOD: MUD ROTARY	
CASING LENGTH:	DRILLER: RAY NORWOOD
WATER LEVEL: 1 FOOT ABOVE GROUND AT TIME OF BORING	

NOTES: N 271,940.15 E 2,624,872.65
EL 13.7

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
	Dark brown, very soft, MUCK, roots, grass (PT)		SS-1	WOI
	Top 6" Same			
5	Bottom brown, loose, fine SAND, trace silt (SM)	8.7	SS-2	7
	Top 6" Same 4"			
	Top 8" Same		SS-3	17
10	Bottom 16" dark brown, very soft, silty CLAY, little organic silt, fibrous, trace peat (CH)	3.7	SS-4	5
	Top 8" Same, wood			
15	Bottom gray, loose, fine SAND, trace clayey silt (SP)	-1.3	SS-5	5
	Tan, dense, fine, SAND (SP)			
20		-6.3	SS-6	65
25	Same	-11.3	SS-7	50/5"
30	Tan dense, fine, SAND, trace (-) silt (SP)	-16.3	SS-8	85
	Dark gray, soft organic, clayey SILT and fine sand (OH)			
35		-21.3	SS-9	3
	Top 6" gray, very dense, calcareous, fine, SAND and shells			
40	Bottom 2" cemented fine SAND and silt (SM/ML)	-26.3	SS-10	50/2"
	Spoon Bouncing			

STANDARD PENETRATION TEST DATA (blows/ft)



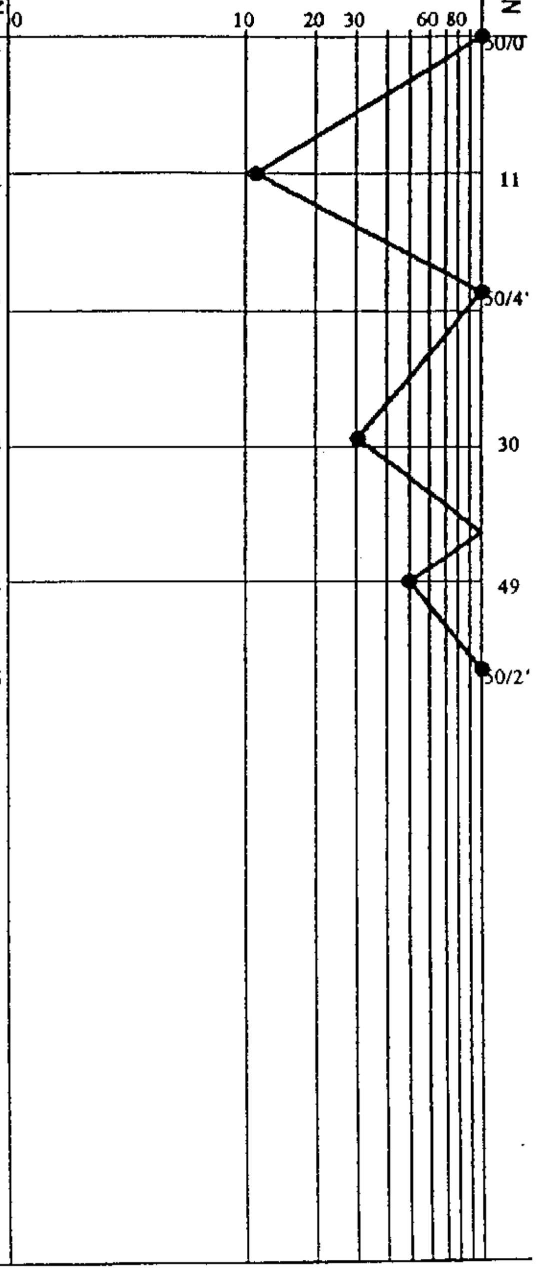
1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

PROJECT: US 501 PHASE VI MYRTLE BEACH, SOUTH CAROLINA 1131-93-431		**BORING LOG B-32**
DATE STARTED: 11/6/93	DATE FINISHED: 11/7/93	NOTES: N 271,940.15 E 2,624,872.65 EL 13.7
DRILLING METHOD: MUD ROTARY		
CASING LENGTH:	DRILLER: RAY NORWOOD	
WATER LEVEL: 1 FOOT ABOVE GROUND AT TIME OF BORING		

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
		Hard Drilling at 45'-46'		SS-11	50/0
		Dark gray, stiff, clayey SILT, some (+) fine sand (ML)			
50			-36.3	SS-12	11
		Dark gray, hard clayey SILT, some (+) fine sand, gray cemented fine sand and silt at tip (ML)			
55			-41.3	SS-13	50/4'
		Same(ML)			
60			-46.3	SS-14	30
		Gray, hard, clayey SILT and fine sand (ML)			
65			-51.3	SS-15	49
		Gray, cemented fine SAND and silt (SM/ML)		SS-16	50/2'
		BOTTOM OF HOLE AT 68.17 FEET			

STANDARD PENETRATION TEST DATA (blows/ft)



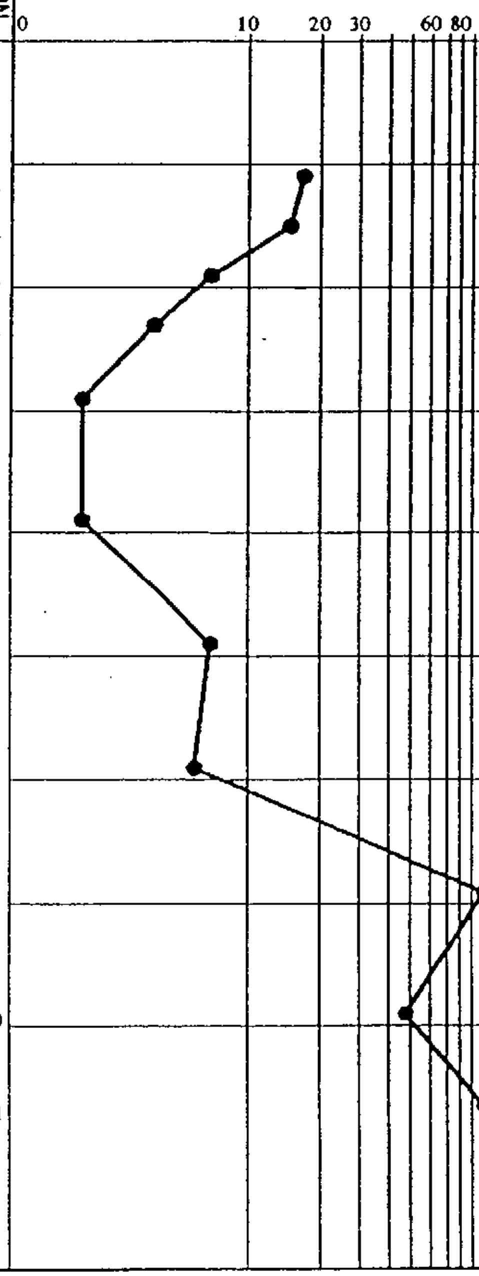
1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

PROJECT: US 501 PHASE VI, MYRTLE BEACH, SOUTH CAROLINA, 1131-93-431	**BORING LOG B-43**
DATE STARTED: 11/7/93 — DATE FINISHED: 11/7/93	NOTES: N 272,400.22 E 2,625,107.85 EL 22.8
DRILLING METHOD: MUD ROTARY	
CASING LENGTH: — DRILLER: SIMRIL	
WATER LEVEL: 8 FEET AFTER 24 HOURS	

DEPTH (feet)	MATERIAL DESCRIPTION	ELEVATION (ft-MSL)	SAMPLE NO/TYPE	N VALUE
	Hand Auger to 3.5'			
5	Brown, medium dense, fine SAND, some silty clay (FILL)(SM)	17.8	SS-1	17
	Brown, medium dense, fine SAND, little (-) silt (FILL)(SM)		SS-2	15
10	Brown, loose, fine SAND, little (-) silt (SP-SM)	12.8	SS-3	7
	Same		SS-4	4
15	Top 2" Same Bottom dark brown, very soft, silty CLAY and wood, slightly organic, trace peat (CH)	7.8	SS-5	2
20	Top 2" Same Bottom dark gray, very soft, silty CLAY, slightly organic, some (-) fine sand, wood (CH)	2.8	SS-6	2
25	Top 16" gray, loose, fine SAND, trace clayey silt (SM) Bottom 4" gray, loose, fine calcareous SAND, little shells, trace silt (SP-SM)	-2.2	SS-7	7
30	Same	-7.2	SS-8	6
35	HARD DRILLING FROM 32.5-37.5 SPOON BOUNCING, NO RECOVERY	-12.2	SS-9	50/0
40	Top 12" gray, soft, clayey SILT, some (+) fine sand (ML) Bottom 12" gray, very dense, cemented fine SAND and SILT	-17.2	SS-10	47
	Gray, very dense, cemented fine SAND and silt		SS-11	50/3
	BOTTOM OF HOLE AT 43.25 FEET			

STANDARD PENETRATION TEST DATA (blows/ft)



1. BORING AND SAMPLING IS IN ACCORDANCE WITH ASTM D-1586.
2. PENETRATION (N-VALUE) IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

S & ME, INC.
840 LOW COUNTRY BOULEVARD
MT. PLEASANT, SOUTH CAROLINA

Appendix C

Laboratory Data

S&ME

Soil Data Summary

Highway 501 Flat Bridges

S&ME Job No. 1611-99-401

Boring (tube)	Sample Depth (feet)	Classification	Natural Moisture (%)	% Fines <#200	γ (pcf)	γ_d (pcf)	Void Ratio (%)	ϕ (^{o})	C (psf)	ϕ' (^{o})	C' (psf)	Atterberg Limits LL	Atterberg Limits PI	CBR (%)	C_c	M	Unconfined stress (psf)
B-201	8.5 - 10	Clayey Sand (SC)	31	16.9								35	19				
B-201	33.5 - 35	Silty Clayey Sand (SC-SM)	26.3	33.7								27	6				
B-203	33.5 - 35	Clayey Sand (SC)	22.7	37.3								26	8				
B-205	6 - 7.5	Poorly Graded Sand with Silt (SP-SM)	22	6.2													
B-205	33.5 - 35	Clayey Sand (SC)	27.8	32.1								26	9				
B-208	8 - 10.5	Elastic Silt (MH)	115.4	98.5								55	25				
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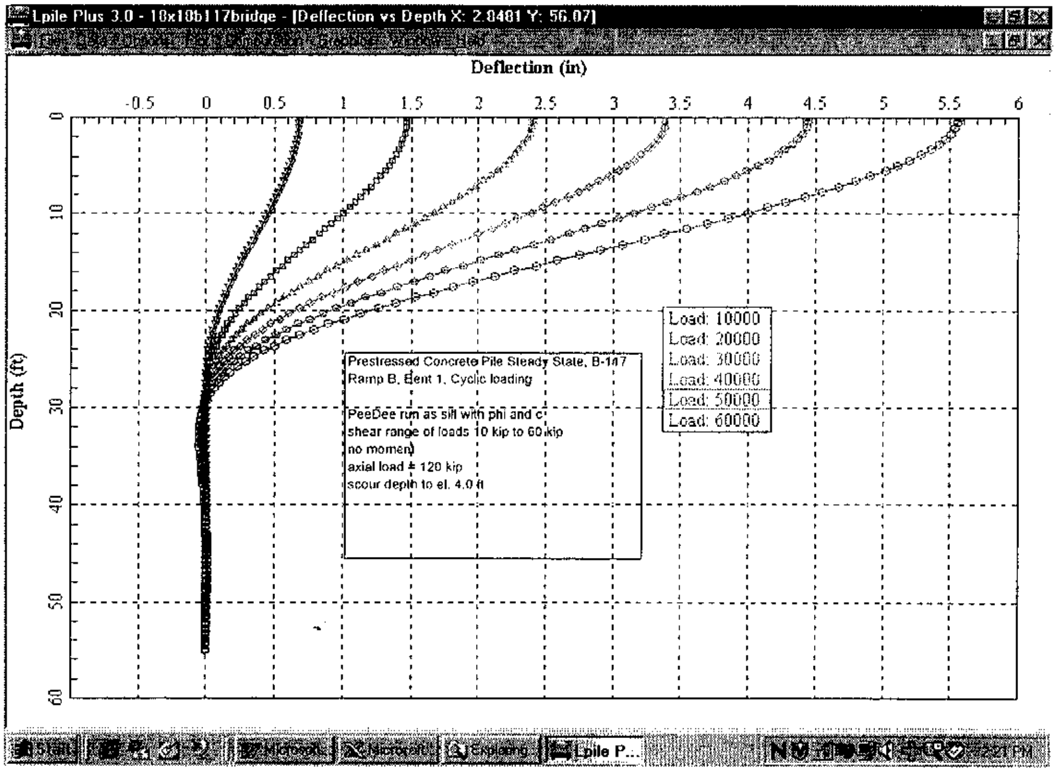
Appendix D

Lateral PSC Deflection Diagrams

Ramp B, Cyclic Loading, Scour Depth El. 4.0 ft.



Ramp B, Cyclic Loading, No Scour

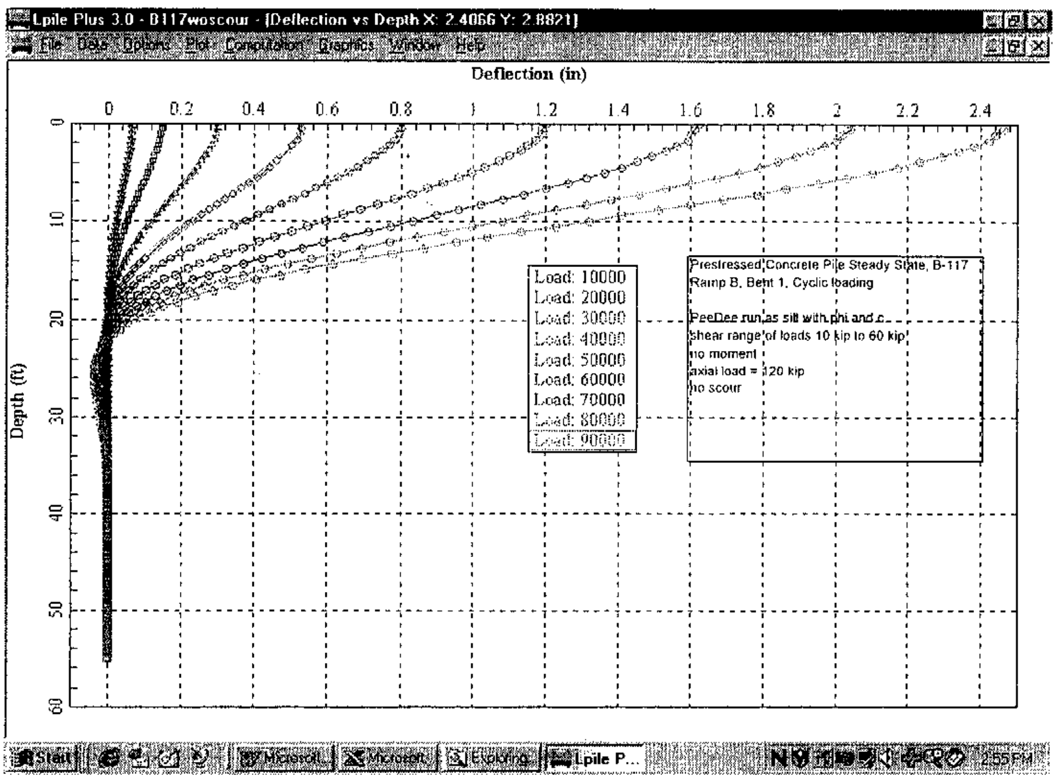
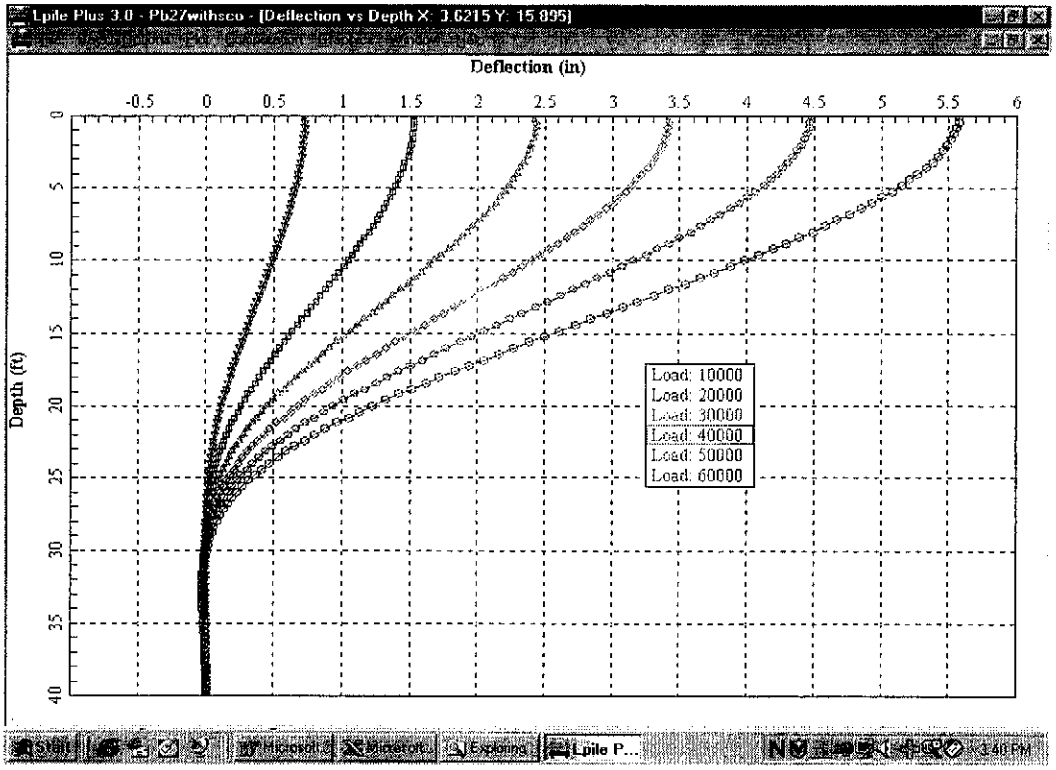


Figure D-1

Dick Scobee Drive with scour El. 1.9 ft., Cyclic loading, PB-27



Dick Scobee Drive with out scour, Cyclic loading, PB-27

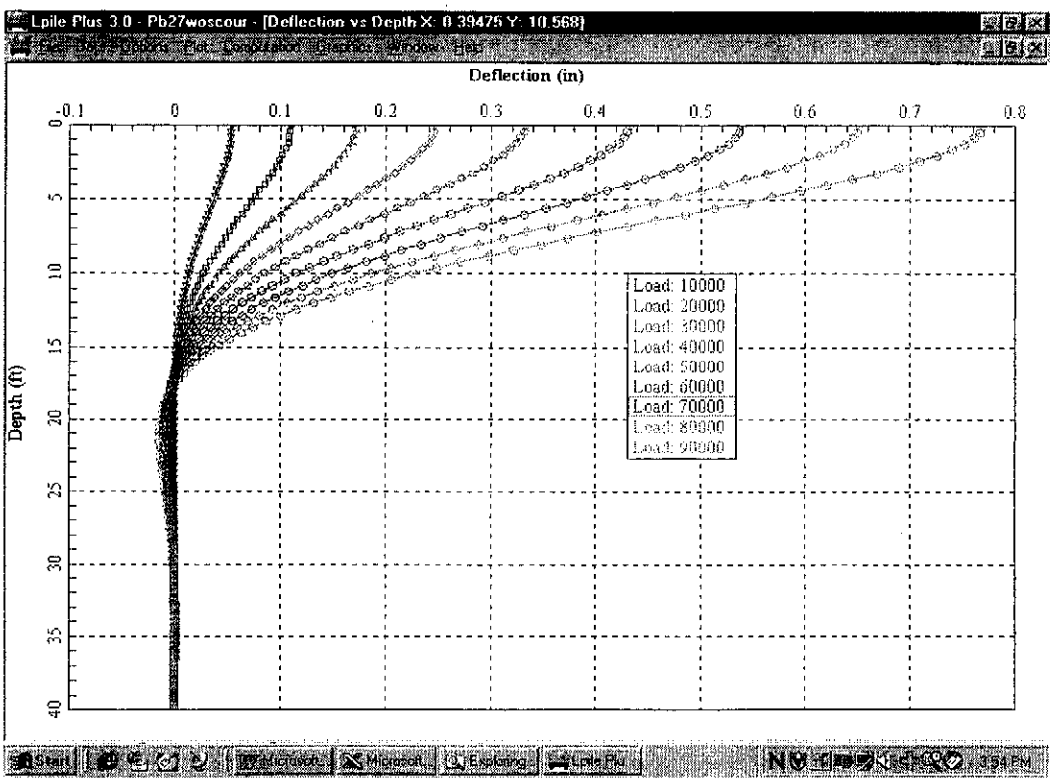
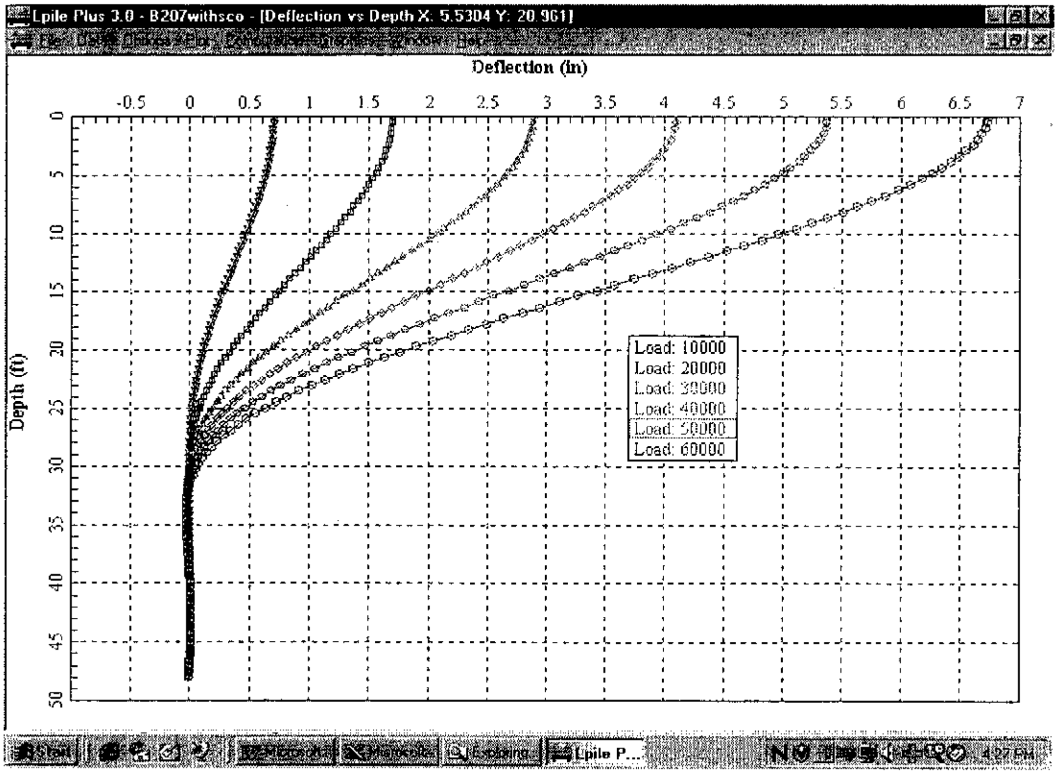


Figure D-2

Frontage Road 6F, Cyclic Loading with scour to El. 4.7 ft.



Frontage Road 6F, Cyclic Loading without scour

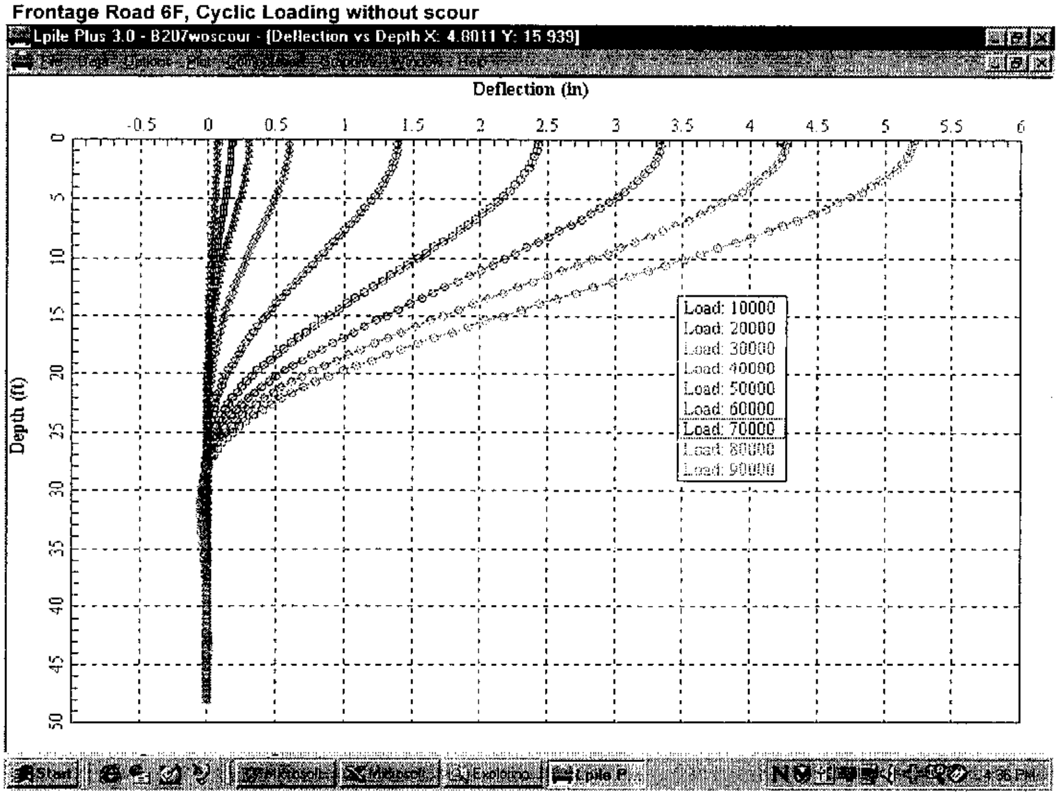


Figure D-3

Waccamaw River,
Horry County, South Carolina
Flood Risk Management Study
Draft Integrated Feasibility Report and
Environmental Assessment
Appendix A5. Structural

US Army Corps
of Engineers

September 2024

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TABLE OF CONTENTS

LIST OF FIGURES

1.0 INTRODUCTION

The structural engineering scope of this feasibility study is to evaluate various flood wall barriers, the installation of relief bridges or culverts on existing roadways, and the installation of a storm surge gate. The types of barriers considered for this study consist of Earthen Berms, I-Walls, and T-Walls; each barrier type has its own requirements, limitations, and footprint requirements. In addition, the relief bridges and culverts were evaluated based on the amount of additional flow needed. The report will discuss in more detail the use of the different flood wall barriers, and relief bridges or culverts.

The study was broken up into four main flood impact areas in Horry County. Below is the summary of structural requirements for each flood impact area. For more details and assumptions of the structural requirements for the flood impact areas, please refer to remainder of this appendix.

1.1 BUCKSPORT

In the Bucksport flood impact area, the study considered and evaluated the installation of a storm surge gate that would be installed across Cowford Swamp on the south side of the bridge along Old Pee Dee Road. The gate would be open to allow normal flow of the creek and would be closed for major storm events.

1.2 CONWAY

In the Conway flood impact area, the study considered and evaluated the installation of relief bridges and/or culverts along HWYs 501, 501 Business, and 905. Conceptual size and locations of the relief bridges and culverts were determined to assist with modeling efforts.

1.3 SOCASTEE

In the Socastee flood impact area, the study considered and evaluated the installation of a flood barrier along both banks of the Socastee Creek. Due to known soil conditions in the vicinity, an I-Wall was proposed for this structure. The I-Wall consisted of a reinforced concrete wall supported by steel sheet piles.

1.4 LONGS AND RED BLUFF

In the Longs flood impact area, the study considered and evaluated the installation of a floodwall along Buck Creek adjacent to the Aberdeen community and continuing north to Rolling Ridge Drive. Due to the anticipated soil conditions and height requirements of the wall, an I-Wall was proposed for this structure. The I-Wall would consist of a reinforced concrete wall supported by steel sheet piles.

2.0 ASSUMPTIONS AND LIMITATIONS

Due to the conceptual stage of this study, assumptions had to be made and there were limitations that existed. One major limitation was not having geotechnical reports in the footprint of the structures proposed. Therefore, soil conditions had to be assumed using known geotechnical data from projects in the vicinity of the proposed and evaluated structures. Also, the heights above existing grade were estimated using the best data obtained at the time.

However, given this is at a conceptual stage, conservative assumptions were made for the purposes of this report. These assumptions and limitations can be fine tuned during the

Preconstruction Engineering and Design (PED) Phase.

2.1 EARTHEN BERM

Earthen Berms were ruled out as a viable option primarily due to their large footprint requirement (i.e. 10 ft wide top, 3 to 1 slope or 4 to 1 slope, vegetative free zone on each side, etc.). The locations where flood barriers were considered were in residential areas where the construction of the earthen berm would require acquisition of the residences the study is trying to protect. There may be opportunities to use earthen berms where the flood barrier crosses the golf course in Aberdeen, located in the Longs and Red Bluff flood impact area, in conjunction with the I-Walls proposed. However, due to the conceptual nature of where this project currently is, the team decided to use I-Walls and optimize the flood barrier if a flood barrier was selected as part of the TSP. Since the I-Wall has not been selected as part of the TSP due to the very low BCR that was calculated, further evaluation of earthen berms will not be considered as this study progresses. For berm footprint dimension, refer to the table below for Total Width requirements for earthen berms.

Berm Height (ft) Above Existing Grade	10 ft Top Width		8 ft Top Width	
	3H : 1V	4H : 1V	3H : 1V	4H : 1V
	Total Width (ft)	Total Width (ft)	Total Width (ft)	Total Width (ft)
1	46	48	44	46
2	52	56	50	54
3	58	64	56	62
4	64	72	62	70
5	70	80	68	78
6	76	88	74	86
7	82	96	80	94
8	88	104	86	102
9	94	112	92	110
10	100	120	98	118
11	106	128	104	126
12	112	136	110	134
13	118	144	116	142
14	124	152	122	150

* Total Widths include a Vegetation Free Zone (VFZ) of 15 ft on each side of the berm

Figure 1: Earthen Berm Footprint Dimensions

2.2 I-WALL

For the purposes of this study, I-Walls were considered for the floodwalls in the Longs and Red Bluff, and Socastee study areas. Working closely with the geotechnical engineer, the team was able to gather geotechnical reports for various projects within the vicinity of the proposed flood walls and determined that an I-Wall constructed with a reinforced concrete wall supported on steel sheet piles was adequate for the required heights. The I-Wall would have a concrete pad installed on the dry side for scour protection in the event the wall was overtopped during a major storm event. The location and height of the I-Wall would need to be determined. Set back from existing structures would also need to be determined for the construction footprint and permanent easement.

However, after the benefits were calculated and compared to the cost of construction of the I-Wall, the I-Wall had a BCR of less than 1.0. Therefore, the I-Wall was not selected as part of the TSP and will not be carried forward in this study.

2.3 T-WALL

For the purposes of this study, T-Walls were explored early in the study but were screened out after close coordination with the geotechnical engineer. Due to known soil conditions in the area, the T-Wall was considered to be more robust than what is needed for the study areas. In addition, the flood wall locations for this study are more inland and not coastal, so the wave loading is much smaller. Also, the seismic loading in Horry County is very small. Therefore, the major loading condition for the flood barrier is the flood loading.

2.4 RELIEF BRIDGES AND CULVERTS

Relief bridges and culverts were considered and evaluated along HWY's 501, 501 Business, and 905. H&H modeling was performed to determine the size needed to allow additional flow of flood waters under the roadways. The positioning of the relief bridges and culverts would comply with SCDOT requirements to ensure they do not interfere with existing bridges along the roadways. In addition, existing utilities may need to be relocated or renovated to allow for installation of the relief bridges and culverts.

3.0 LOADS

All flood barriers will be designed to meet the requirements and guidance of the EM 1110-2-2502, and all relief bridges will comply with SCDOT and AASHTO criteria. More detailed information on the loads for the associated structures and load combinations are listed below.

3.1 FLOOD BARRIERS

The load cases considered for this study were in accordance with Inland Flood Wall requirements in EM 1110-2-2502. To date, analysis has not been completed, but engineering judgement and close coordination with the geotechnical engineer were used at this stage. More detailed analysis and site-specific geotechnical investigations would need to completed during project optimization. In addition, site specific geotechnical investigations would need to be completed to characterize the soils, as well as determine the foundation system and if any site improvements would be required.

Case I1: Design Flood Loading
Case I2: Water to Top of Wall
Case I3: Earthquake Loading
Case I4: Construction Short-Duration Loading

3.2 RELIEF BRIDGES

All bridges will be designed to SCDOT requirements, as well as culverts that will be placed under the roads. All culverts will be rated to support vehicular traffic, including the HS-20 truck loading. As the study progresses and optimization occurs, additional information and detailing would be completed. Working closely with H&H and the rest of the engineering team, the sizes of the culverts and bridges would be optimized, and some preliminary detailing would be done to prevent erosion. This could be accomplished using rip rap or large stone to prevent scour or erosion. In addition, working closely with Geotech would help determine the foundation system needed and if site improvements would be required.

4.0 GATES

A storm surge gate was considered and evaluated in the Bucksport study area. The gate would primarily be open to allow for normal flow of water and boat recreation that occurs. However, during major storm events the gate would be closed to hold back the surge. It was anticipated that the gate structure would be pile supported since it would be constructed in the creeks waterway and would need a seepage wall to prevent water seeping underneath the wall. The walls that would tie the structure into high ground still needed to be determined, but due to current analysis, these walls were expected to be I-Walls, which were discussed earlier in this Appendix.

However, after modeling was completed, and the cost of construction was compared to the benefits, the storm surge gates produced a BCR of less than 1.0. Therefore, this will not be selected as part of the TSP.

Figure 2: Storm Surge Gate Example

5.0 FUTURE DETAILING AND RESILIENCY

Due to climate change and future changes to the surrounding land where the barriers were proposed to be constructed, measures would be taken to ensure the barrier can adapt to our changing environment, as well as reduce required maintenance and ensure longevity. All the items listed below would be considered during optimization of this study.

5.1 INCREASING BARRIER HEIGHT

Since the I-Wall does not have any battered piles or major lateral resisting elements, the I-Wall would be the most difficult to increase in height, if that needed to be done in the future. For the purposes of this study, it is assumed that an I-Wall cannot be raised without significant structural upgrades to the system.

In addition, the T-Wall has battered piles which are currently assumed to be deeply driven to a suitable stratum providing significantly more lateral resistance than the I-Wall. This would allow for easier retrofitting of the barrier to provide an increased level of protection without requiring significant structural or foundation upgrades. However, future retrofit design would need to analyze the structure to determine how high the existing structure could adequately provide protection, and be designed to updated codes, criteria, and standards.

5.2 CORROSION PREVENTION AND COMPOSITE MATERIALS

The barriers for this study were being built in areas where they would constantly be exposed to weather and in very corrosive environments. Therefore, corrosion prevention measures should be taken into consideration to reduce required maintenance and ensure longevity. These measures would consist of use of galvanized or epoxy coated rebar, use of fiberglass rebar, and the use of composite materials for major structural components. In addition, where material strengths are sufficient, vinyl sheet piles could be considered.

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References

EM 1110-2-2502 Retaining and Flood Walls
EM 1110-2-2906 Design of Pile Foundations
EM 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures
ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects
American Association of State Highway and Transportation Officials (AASHTO)
SCDOT Criteria and Standards