US Army Corps
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APPENDIX A1: HYDROLOGY AND HYDRAULICS

WACCAMAW RIVER,

HORRY COUNTY, SOUTH CAROLINA

FLOOD RISK MANAGEMENT STUDY INTEGRATED FEASIBILITY REPORT AND ENVIRONMENTAL ASSESSMENT

MAY 2026

MAIN REPORT SUMMARY

The Integrated Feasibility Report and Environmental Assessment (FR/EA), that this appendix addresses, details a collaborative study by the U.S. Army Corps of Engineers (USACE) and Horry County, South Carolina. It is aimed at reducing existing and future flood risks to communities and transportation infrastructure within the Waccamaw River Basin, with a focus on Horry County. The study identifies four key flood impact areas: Longs & Red Bluff, Conway, Bucksport, and Socastee.

The flood impacts in each of these areas were independent of each other, so solutions could be evaluated self-reliantly, making any proposed alternative plans separable. The study considered a range of structural, non-structural, and nature-based solutions while incorporating public feedback gathered during meetings. An environmental analysis was completed, and a Finding of No Significant Impact is included within the main report. The document completed a public review and comment period while also undergoing internal agency reviews and adapted to those concerns and suggestions. In addition to historical flooding, the report acknowledges the flooding event caused by Hurricane Debby in August 2024 during this study, and its impact was assessed to further inform the study's conclusions.

The Recommended Plan, based on an evaluation of alternatives, includes two separable elements that are incrementally justified: Relief Bridges (cross drains) in the Conway flood impact area and Barrier Removal in the Socastee flood impact area. The Recommended Plan is classified as the National Economic Development Plan and is also the plan that maximizes net comprehensive benefits. No alternatives were justified for Federal investment for the Longs & Red Bluff and Bucksport flood impact areas. This Appendix provides detailed hydrology and hydraulics information to support these recommendations.

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A 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, SC Feasibility 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 the study area and potential effects long-term hydrometeorological and coastal change as specified by Engineering and Construction Bulletin (ECB) 2018-14 Revision 1 (rev. 2, 2022)(USACE, 2018, 2022) and ER 1100-2-8162 (USACE, 2019).

A.1 Vertical Datum

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

B BASIN OVERVIEW

B.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, depicted in Figure B-1.

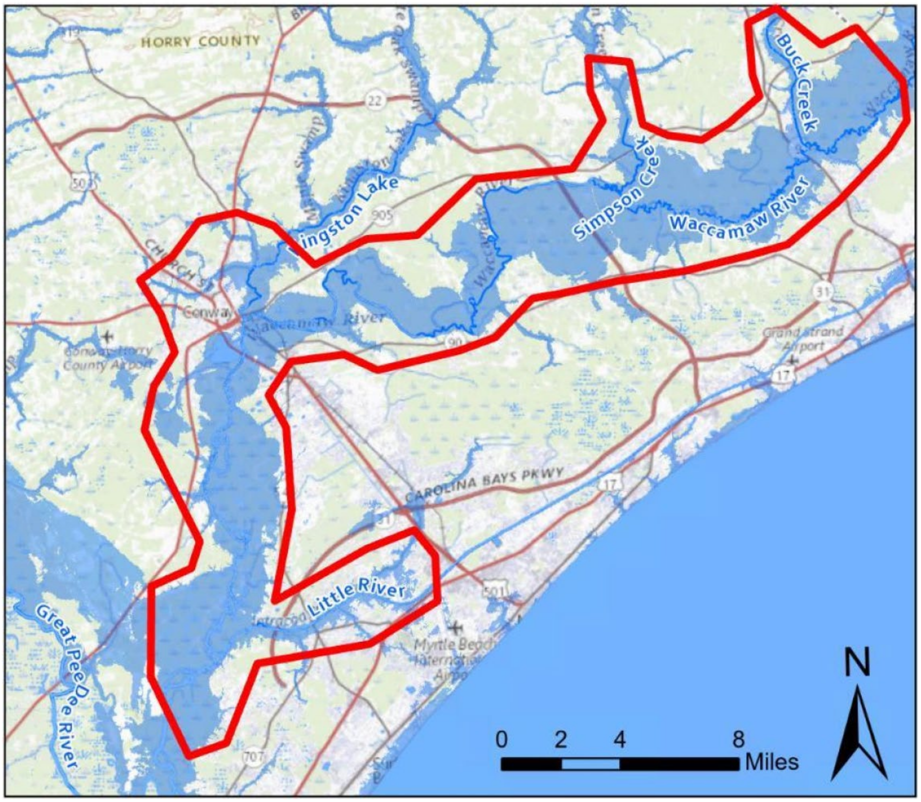


Figure B-1. Waccamaw River FRM study area

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 B-2 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.

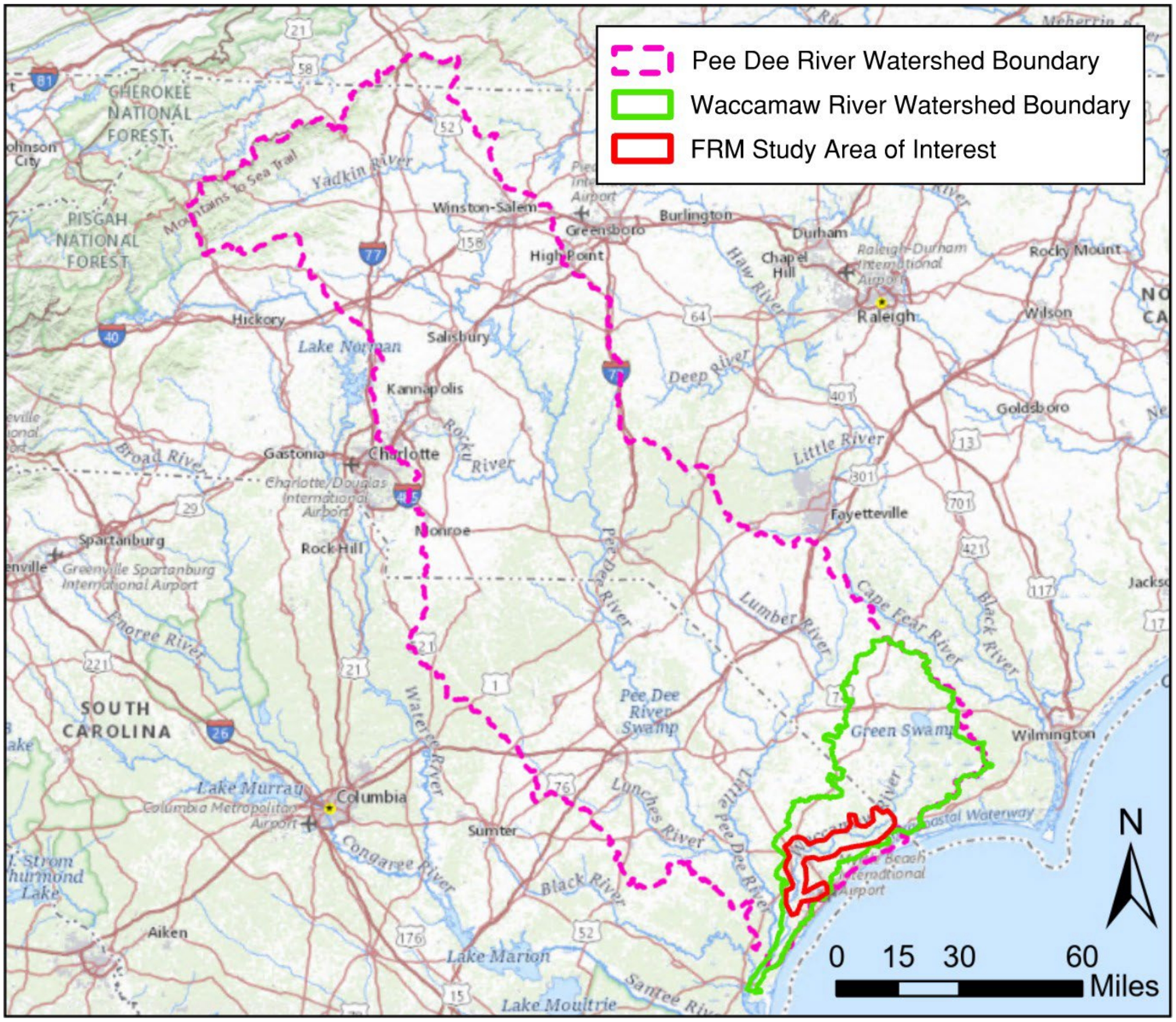


Figure B-2. Waccamaw River Basin Watershed and Study Area

Figure B-2 shows the HUC 8 and 10 outlines for the Waccamaw River Watershed. This image provides an overview of the watershed and sub watersheds that spans across North Carolina and South Carolina.

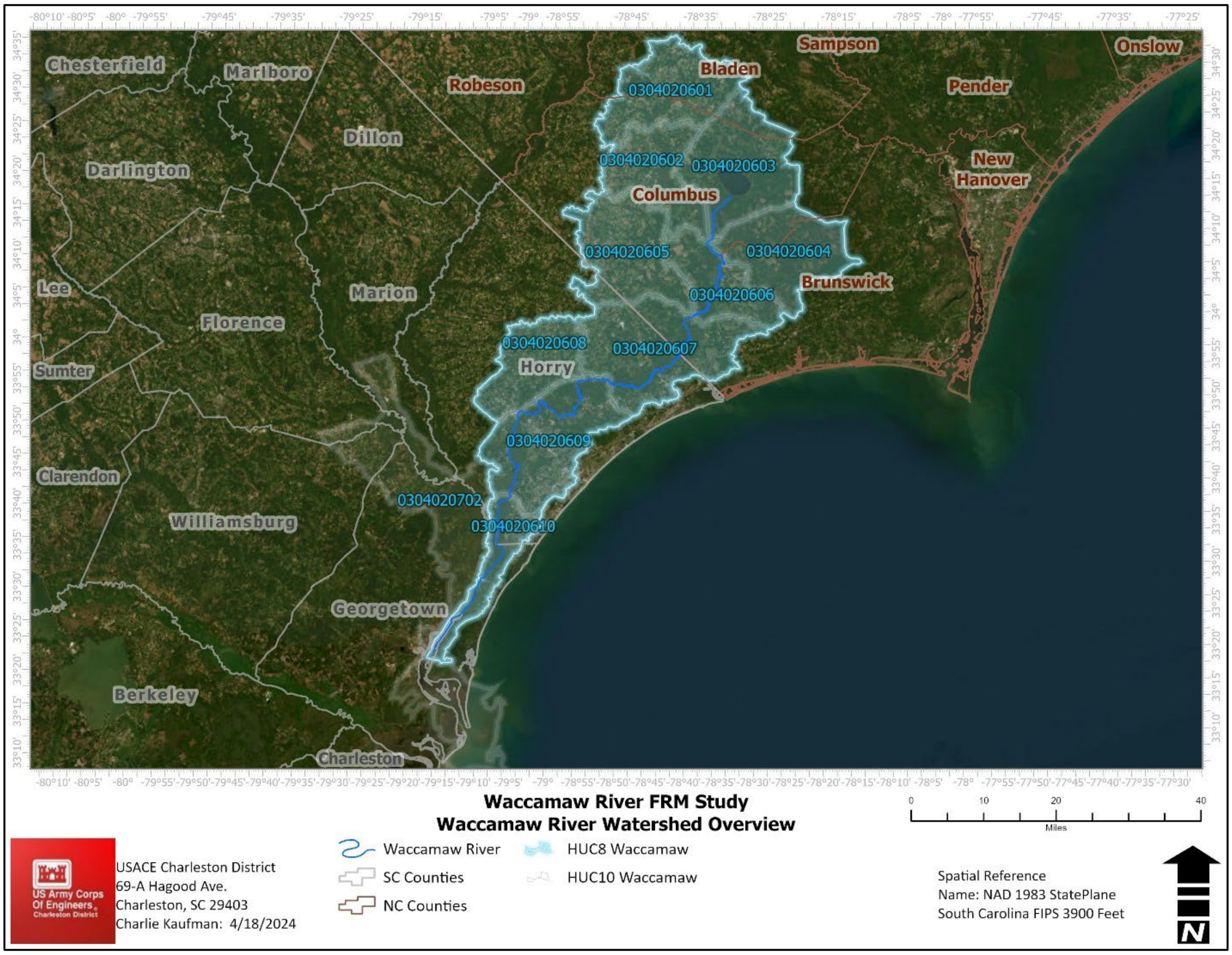


Figure B-3. Waccamaw River watershed and sub watersheds

B.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 (NID) (https://nid.sec.usace.army.mil/) are shown in **Figure B-4**. 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.

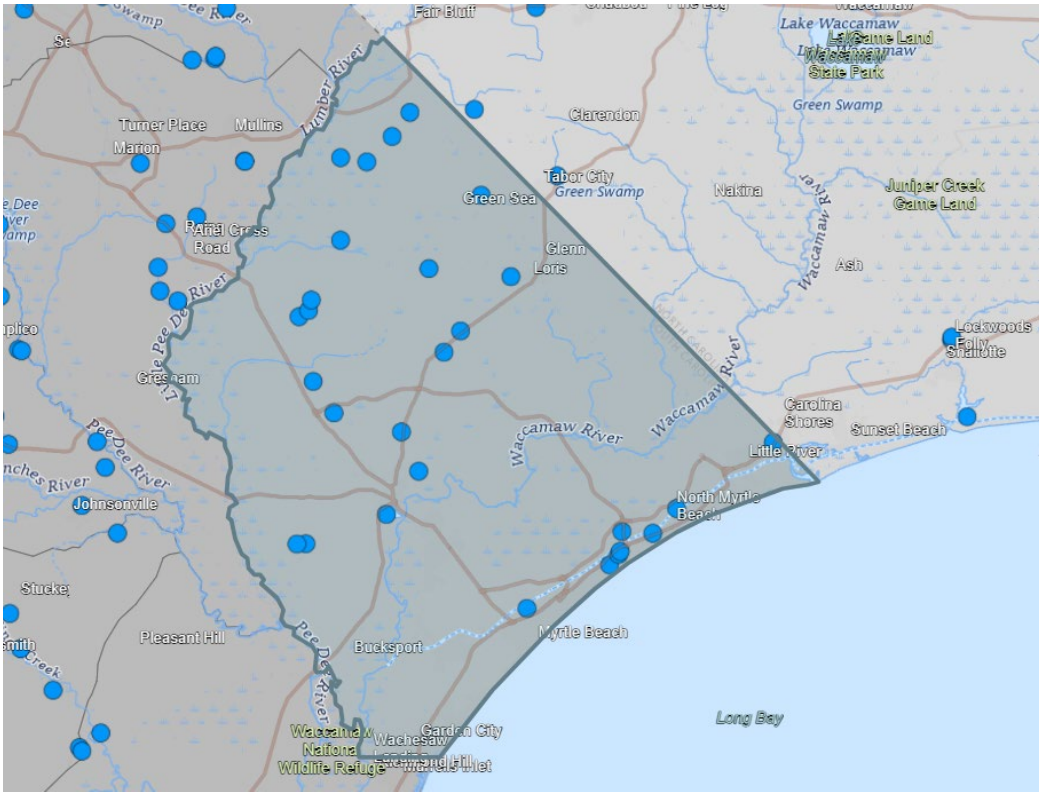


Figure B-4. National Inventory of Dams locations within Horry County

B.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 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%. Watershed storage is 48.4%, which includes lakes, ponds, rivers and wetlands.

Horry County contains a portion of the Atlantic Intracoastal Waterway (AIWW), which was constructed by USACEin 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. **Figure B-5** shows the Waccamaw River Watershed with wetlands and water quality locations identified. 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, depicted in **Figure B-5**.

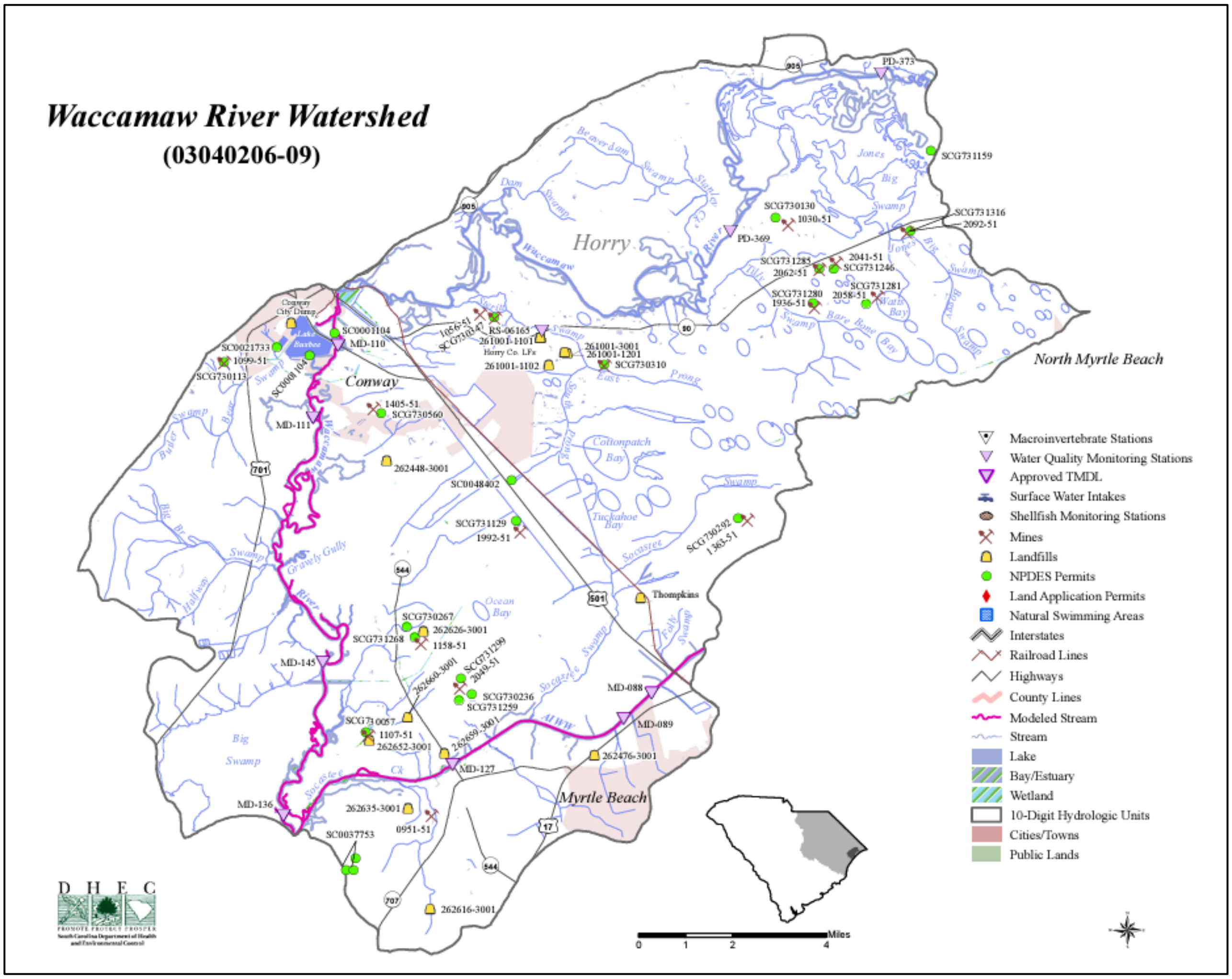


Figure B-5. Waccamaw River Watershed location major highways, lakes/bays and wetlands (SCDES 2023)

Table B-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

Stream	Drainage Area (sq mi)
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

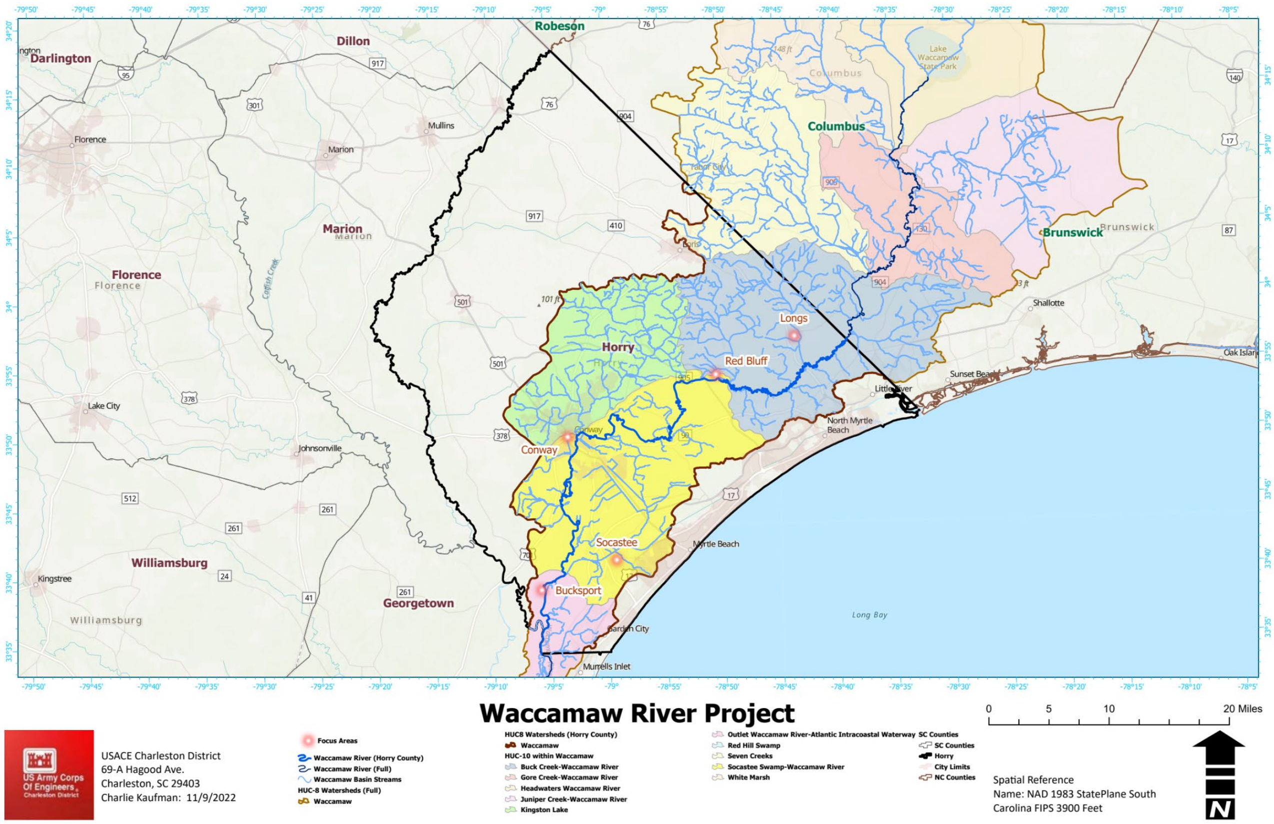


Figure B-6. HUC 10 Watershed map for Waccamaw River with the study areas identified

B.4 Land Cover

The most current (2019) National Land Cover Database (NLCD) for the Waccamaw River basin is shown in **Figure B-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.

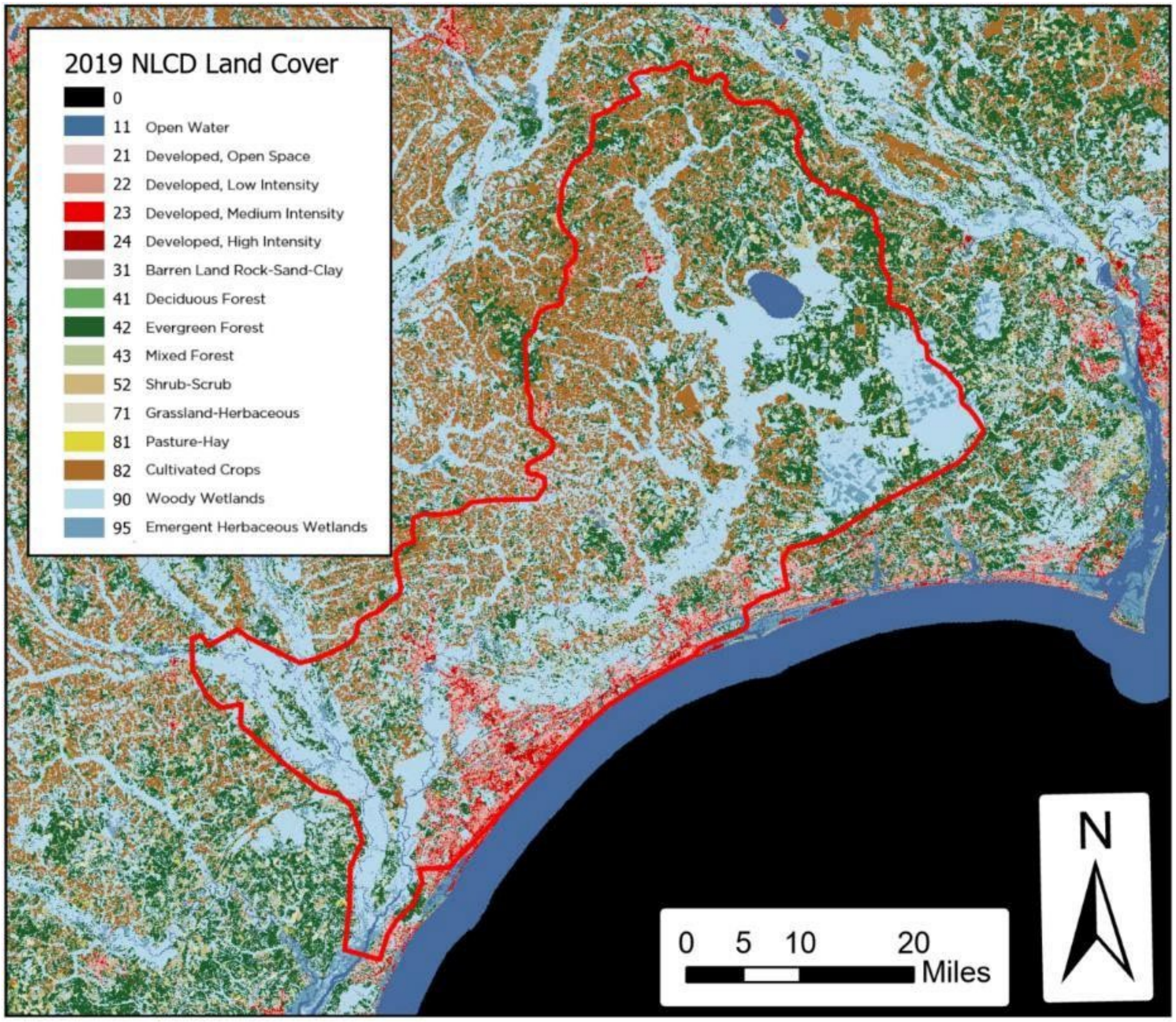


Figure B-7. NLCD 2019 map of the Waccamaw River Watershed

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 B-2**.

Table B-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%

Land Cover Type	Percentage of Total River Basin Area
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%

B.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 km^2) 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.

B.6 Topography

The Waccamaw River Basin is located entirely within the plains 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).

B.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 (million years ago) 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 B-7**) and the SSURGO soil data (**Figure B-8**) 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 E.1.2. 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.

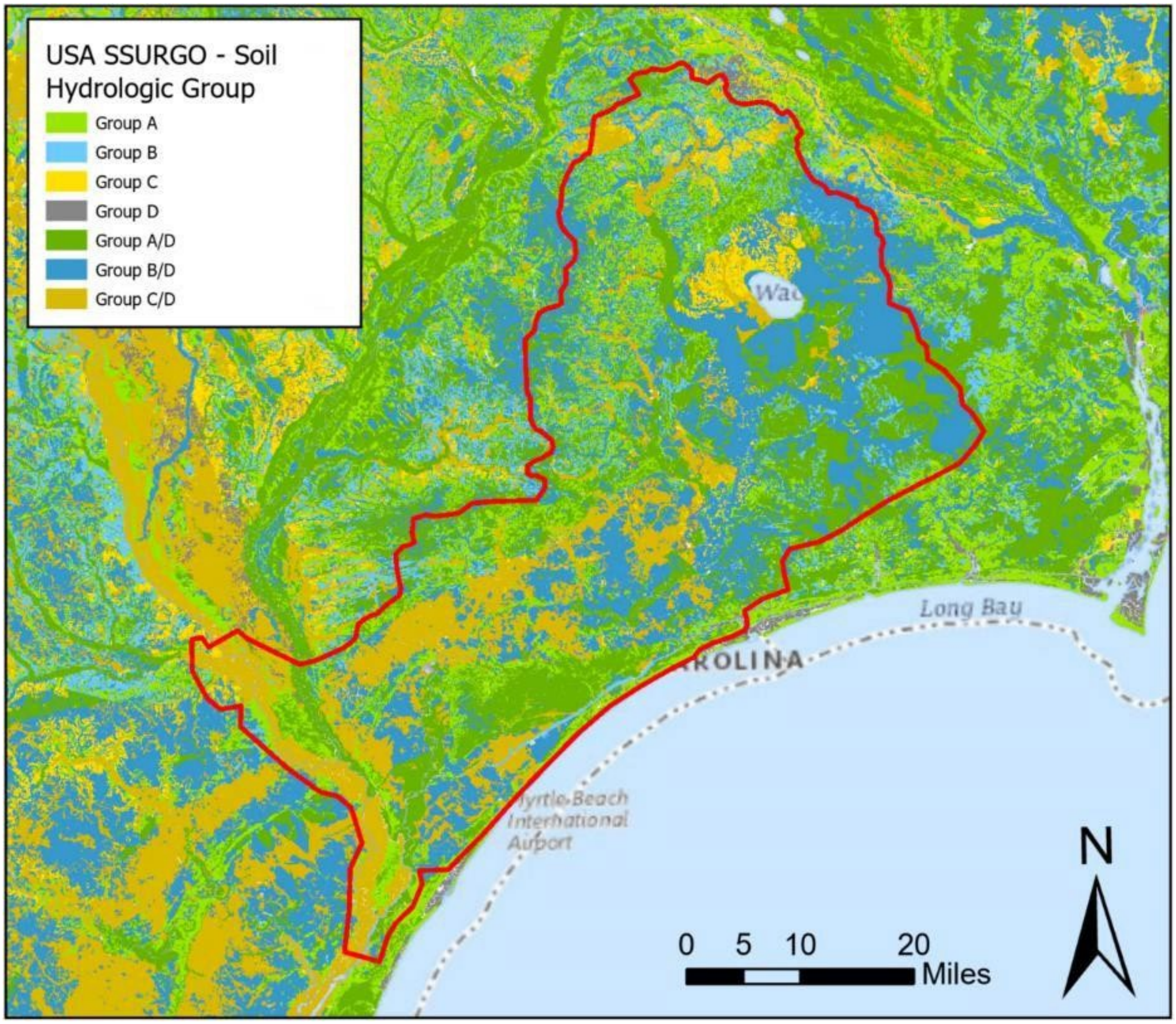


Figure B-8. USA SSURGO data of the soil types in the Waccamaw River Watershed

B.8 Previous Studies

B.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 many 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.

B.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.

B.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.

B.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.

B.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 to understand the benefit and cost benefit ratio of the study.

B.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. 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).

B.8.3 State Studies

The state studies listed below were selected based on their broad scope within the basin and are 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.

B.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.

B.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.

B.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.

B.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 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.

B.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.

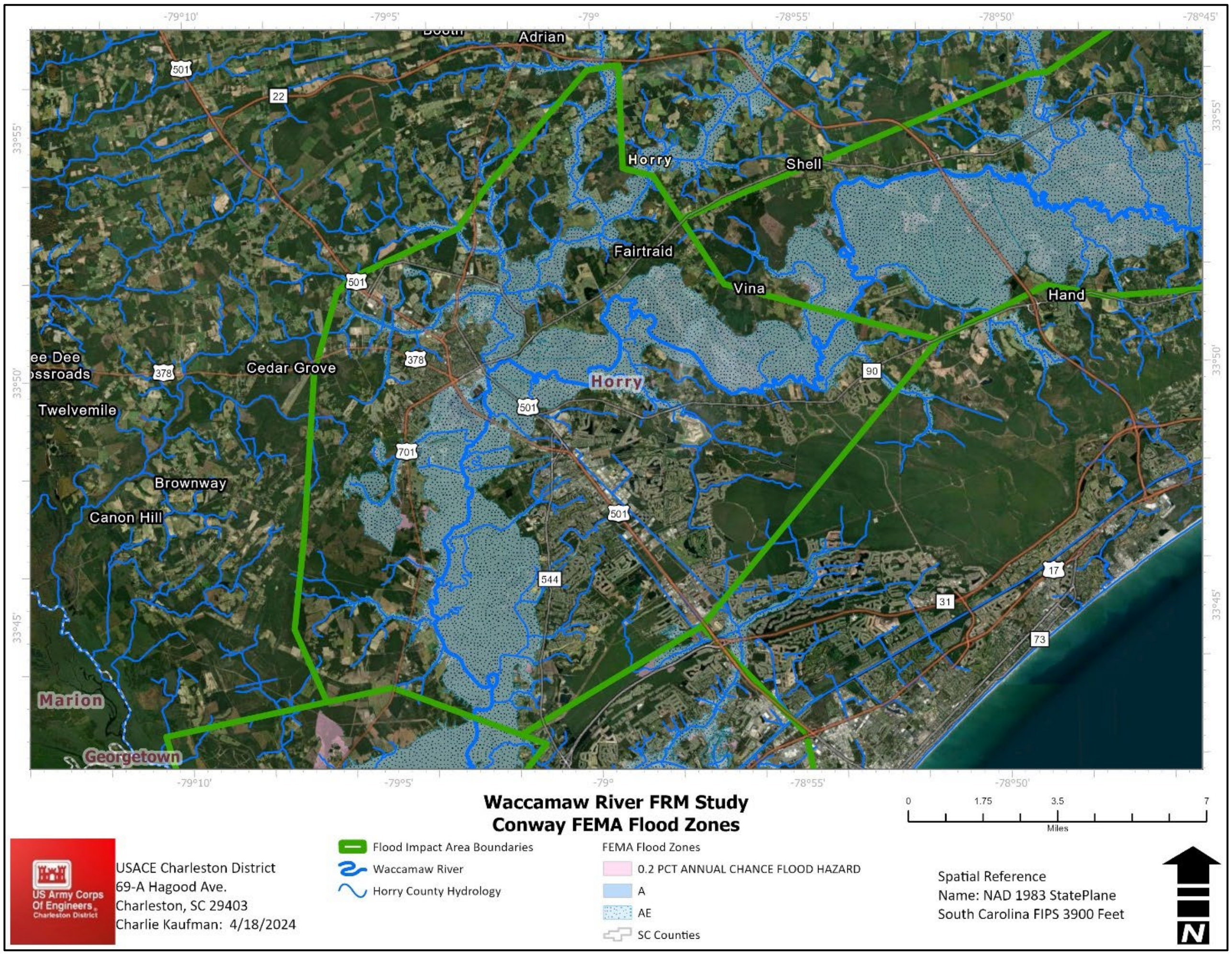


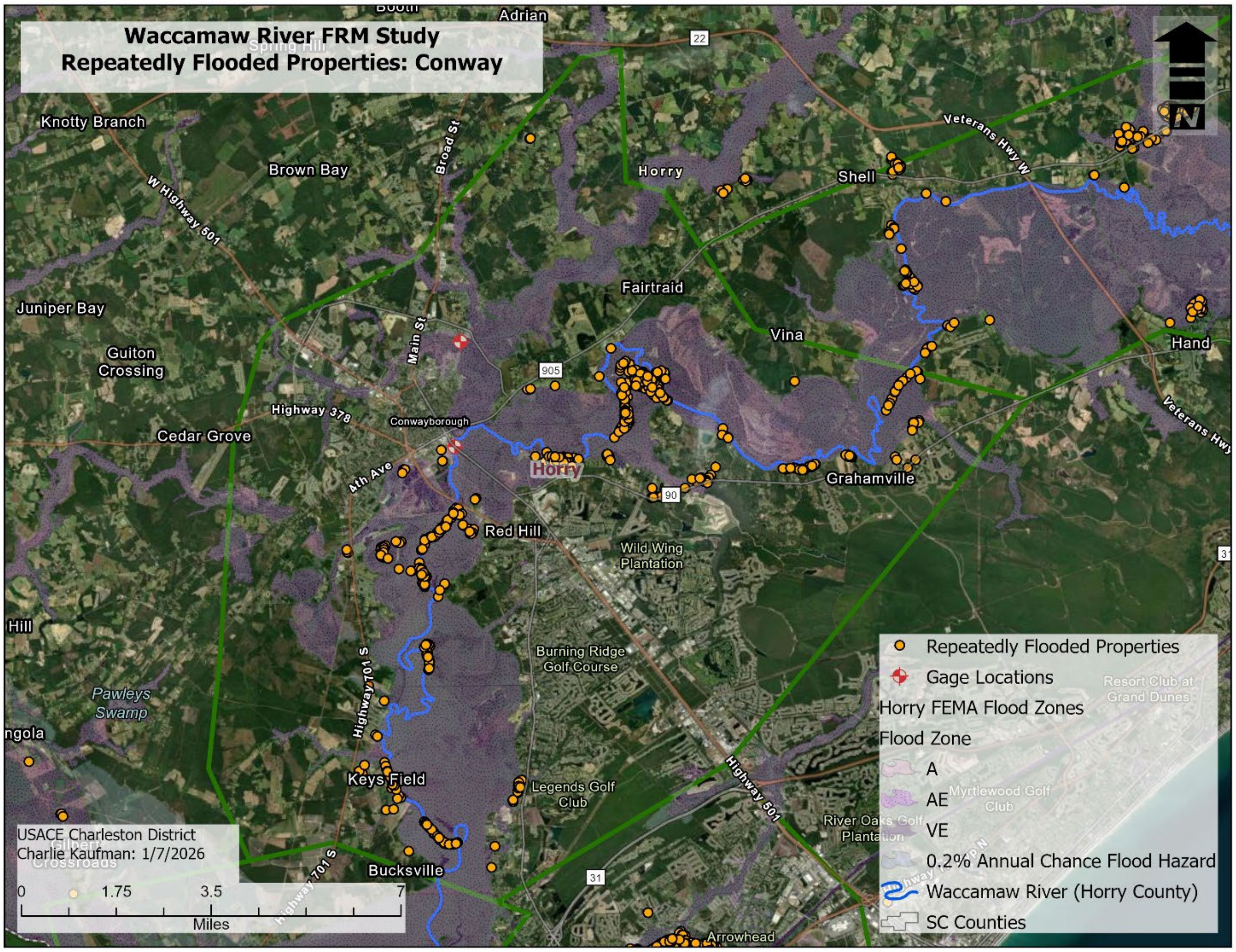
Figure B-9. Conway, SC Focus Area within the Waccamaw River Watershed

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 B-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.



Waccamaw River FRM Study
Repeatedly Flooded Properties: Conway

USACE Charleston District
Charlie Kaufman: 1/7/2026

Figure B-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.

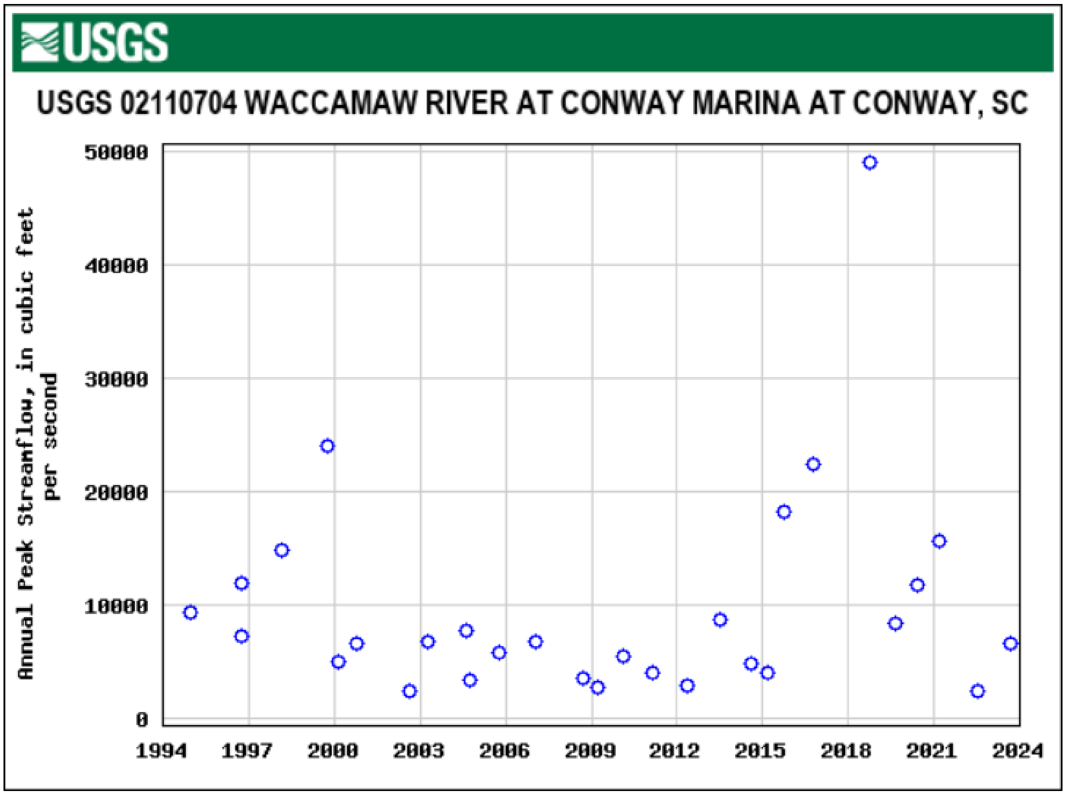


Figure B-12. USGS Gage 02110704 Waccamaw River at Conway Marina Annual Peak Streamflow since 1994

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. As seen in **Table B-3**, six of the top ten highest peaks occurred within the past ten years, indicated by an asterisk (*). On September 26, 2018 the Waccamaw River peaked at 21.16' – the highest crest ever occurring in a non-hurricane rain event. Due to the City's resiliency efforts, minimal damage was experienced.

Table B-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

Date	Gage Height (ft)
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

A figure of peak streamflow for USGS gage 02110550 Waccamaw River Above Conway can be viewed in **Figure B-13** and **Table B-4.** As seen in the table, six of the top ten highest peaks occurred within the past ten years, indicated by an asterisk (*). Annual Peak streamflow peaked at 43,500 cfs on September 23, 2018 during Hurricane Florence. The streamflow gage exceeded its flow capacity and stopped recording, so there is a likelihood that flows were significantly higher. Due to the City's resiliency efforts, minimal damage was experienced.

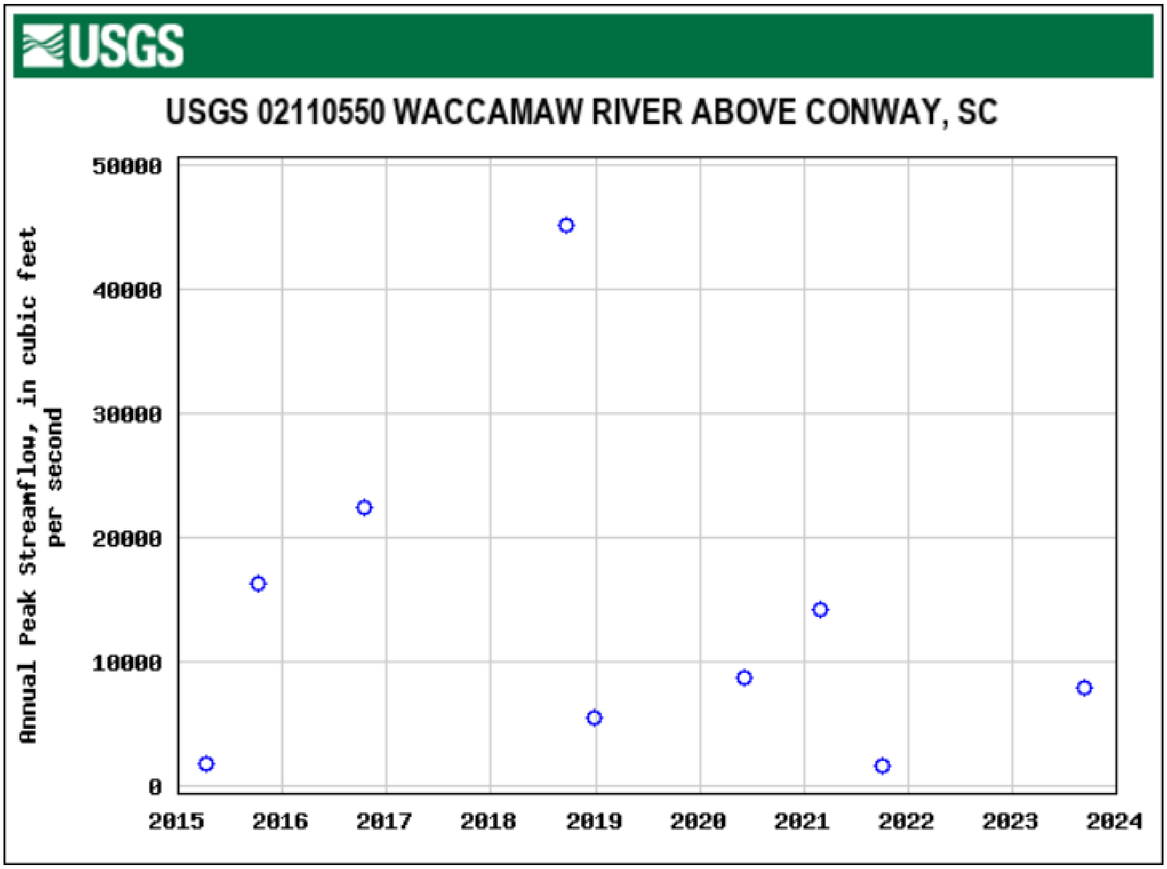


Figure B-13. USGS Gage 02110550 Waccamaw River Above Conway, Annual Peak Streamflow

Table B-4: Peak stream flow for USGS Gage 02110550

Date	Peak Streamflow (cfs)
9/23/2018*	45200
10/16/2016*	22400
10/8/2015*	16300
2/24/2021*	14200
8/17/2024*	13900
6/5/2020*	8740
9/9/2023*	7980
12/25/2018*	5540
4/8/2015*	1730
10/4/2021*	1710

B.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 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.

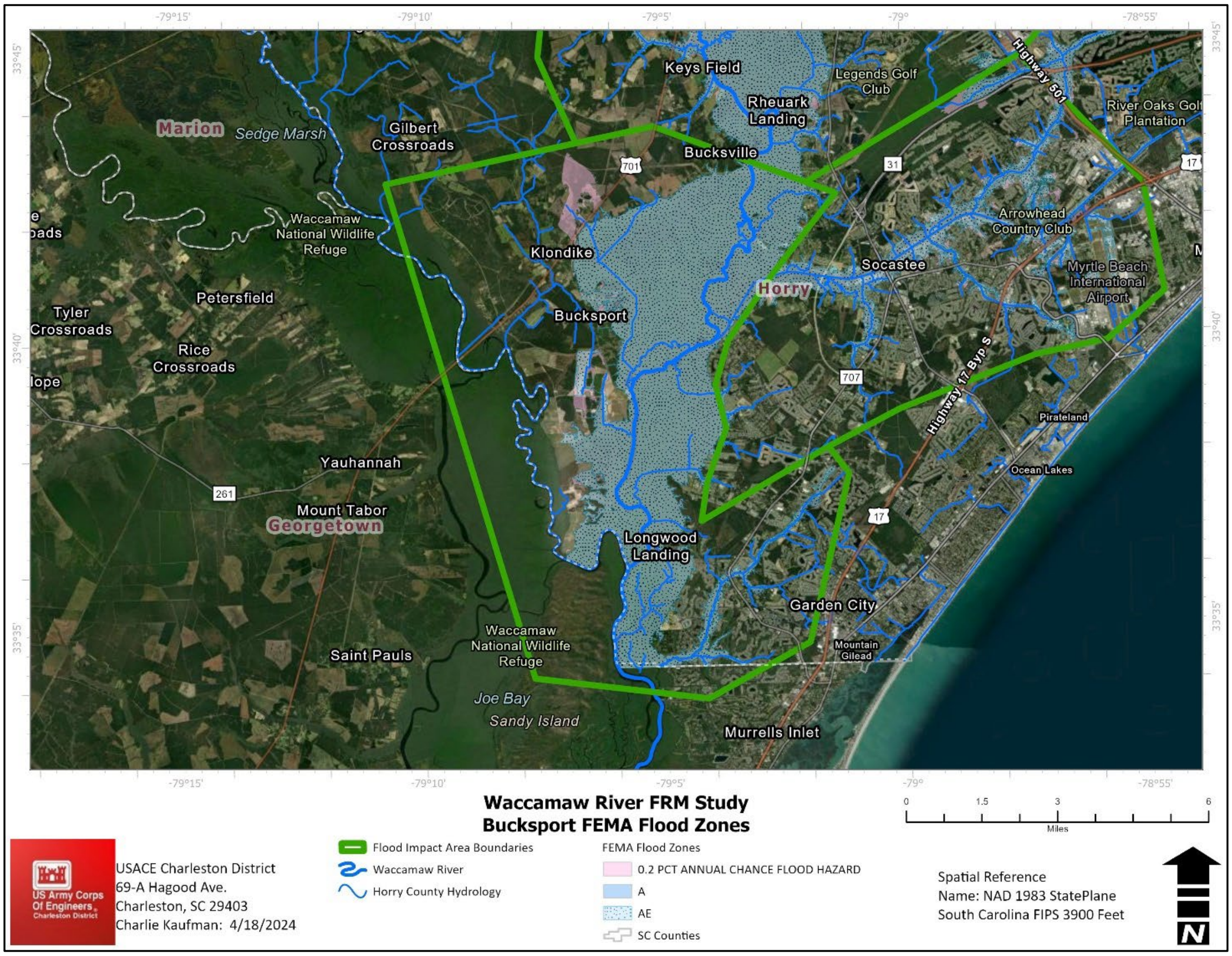


Figure B-14. Focus Area of Bucksport, SC within the Waccamaw River Watershed

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, or to 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 B-15. Flooded Road during Hurricane Florence, in Bucksport, SC (Horry County, 2021)

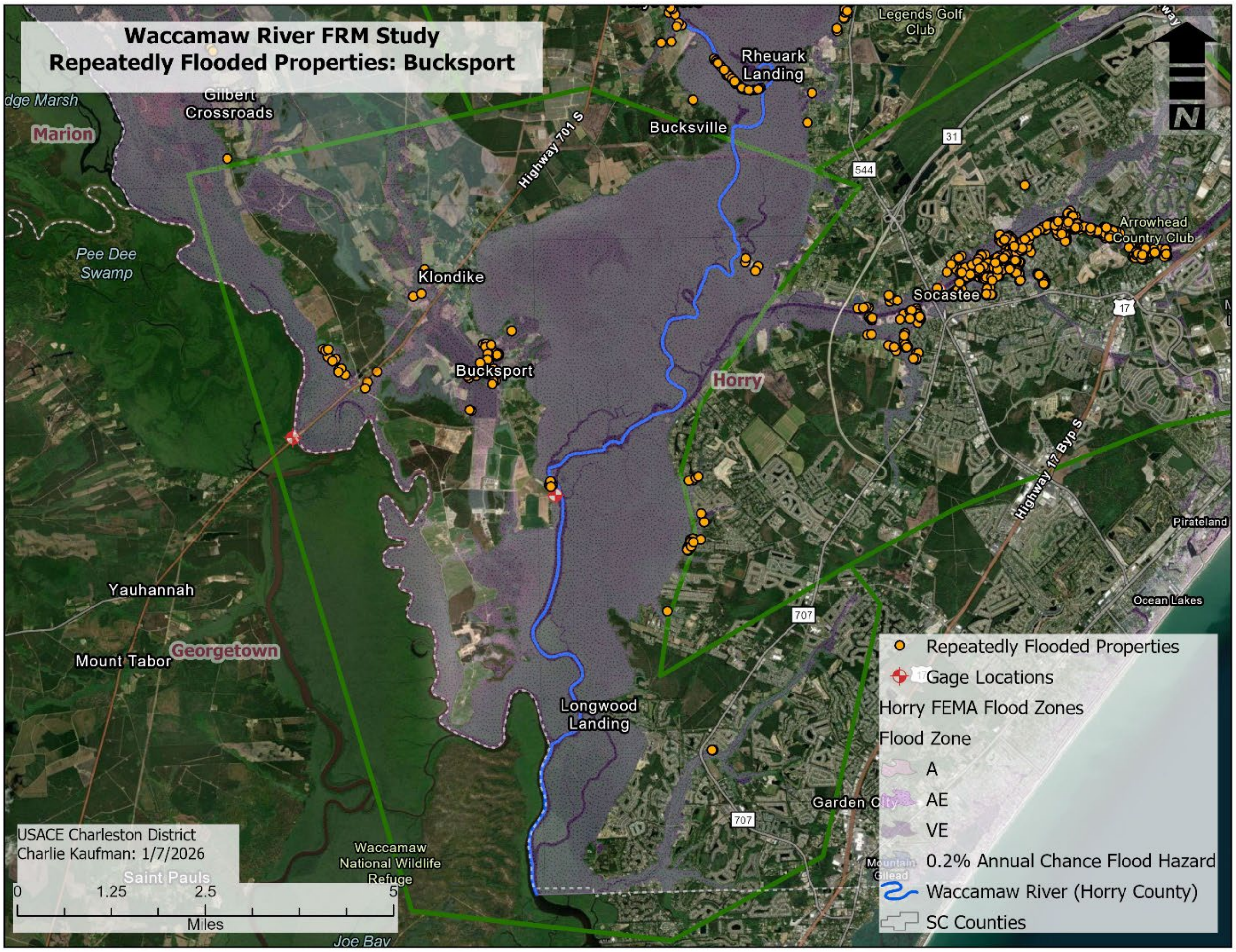


Figure B-16. Properties repeatedly flooded by major Hurricanes and flooding events

Figure B-16 identifies the location of the gage along the Waccamaw River at Bucksport, repetitive flooded properties and the corresponding **Figure B-17** is the peak gage heights measured.

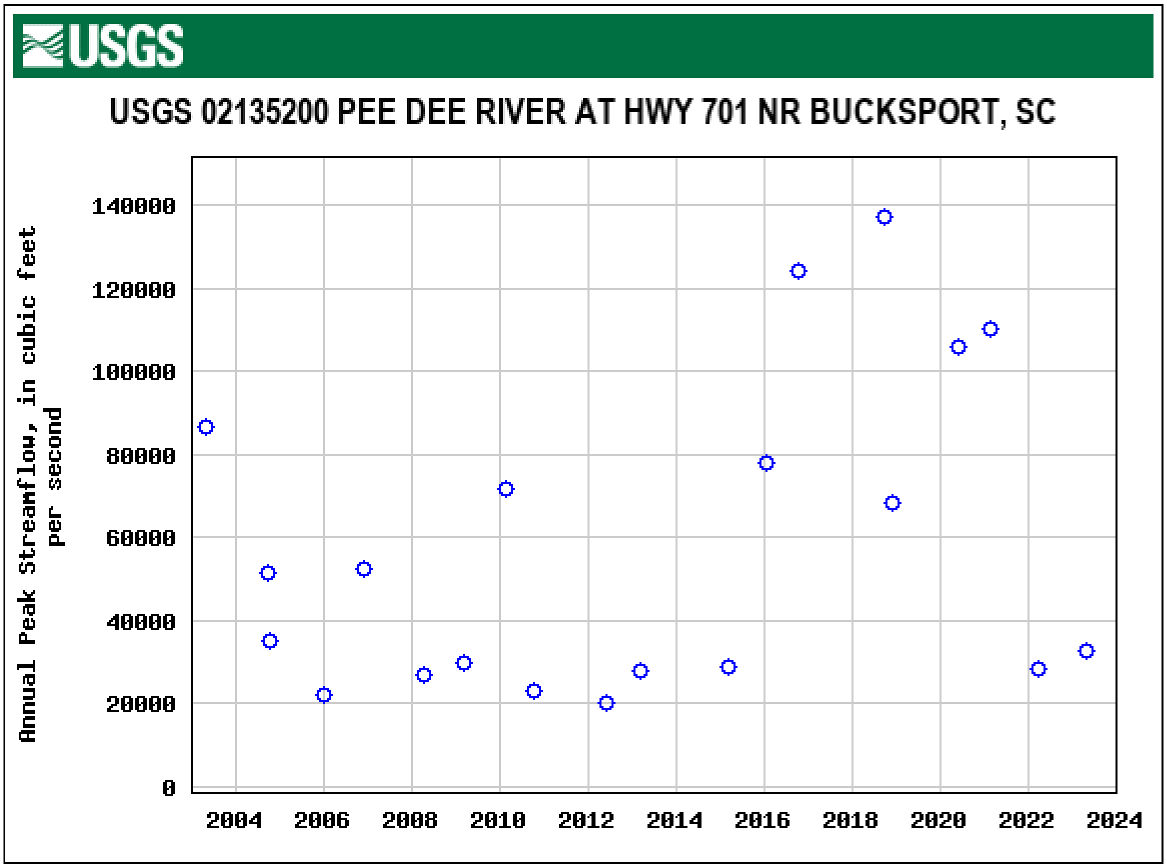


Figure B-17. Floods of Record of the Pee Dee River near Bucksport, SC

As seen in **Table B-5**, six of the top ten highest peaks occurred within the past ten years, indicated by an asterisk (*). On September 27, 2018, the Waccamaw River near Bucksport peaked at 26.67’ – the highest crest ever occurring in a non-hurricane rain event. Due to the City’s resiliency efforts, minimal damage was experienced. This peak is one day after the peak for the upstream gage.

Table B-5. 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

Date	Gage Height (ft)
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

B.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 B-18**). 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.

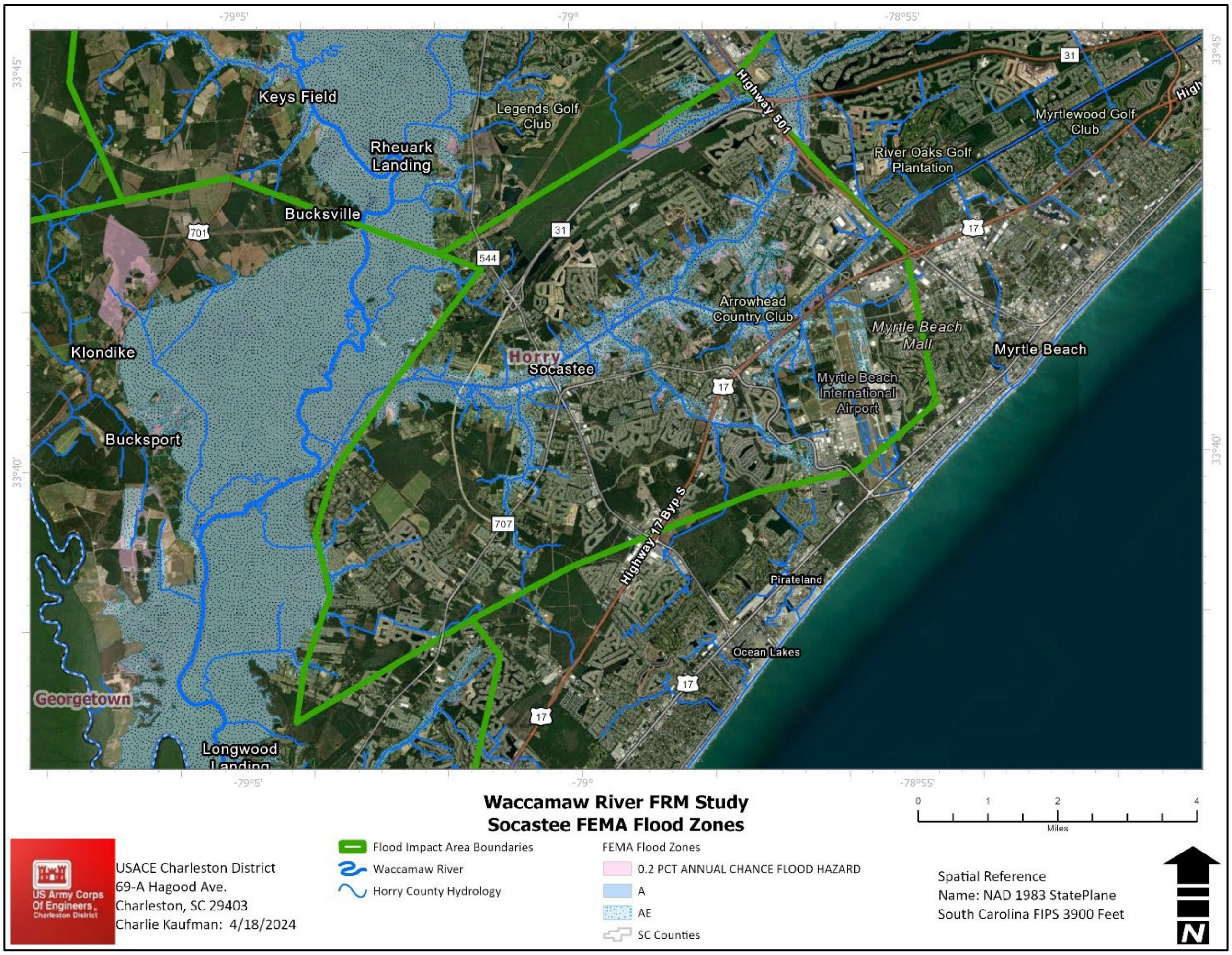


Figure B-18. Socastee Focus Area within the Waccamaw River Watershed

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 B-19**).

Figure B-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 (T**able B-6**).

Table B-6. 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

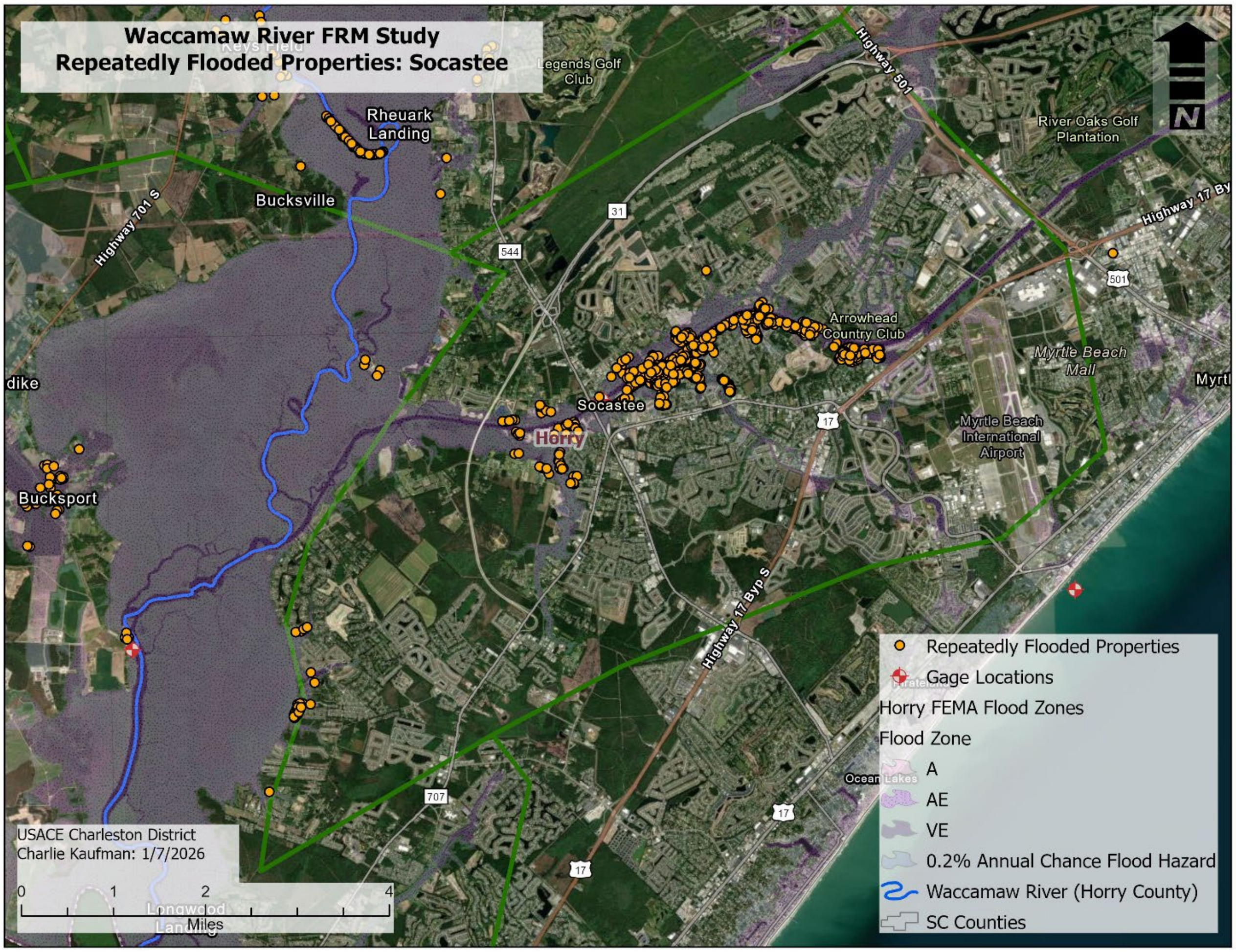


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Figure B-20. Properties in Socastee repeatedly flooded during major events

B.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.

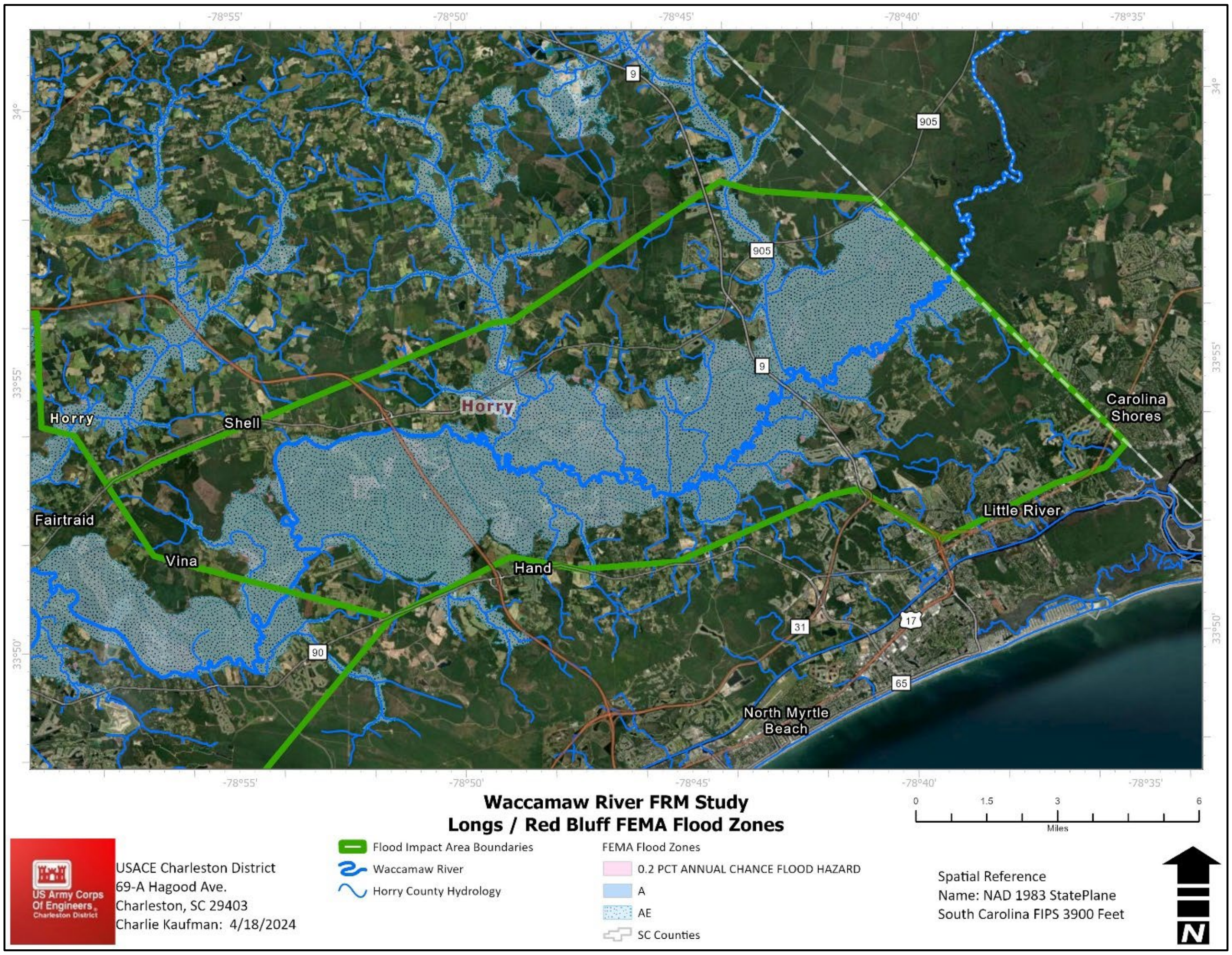


Figure B-20. Focus Area of Longs/ Red Bluff.

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 B-21**). During Florence, the Waccamaw River gage near Longs recorded a peak stage on September 21, 2018, of approximately 18 feet above normal (**Figure B-22**). 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 B-7.**

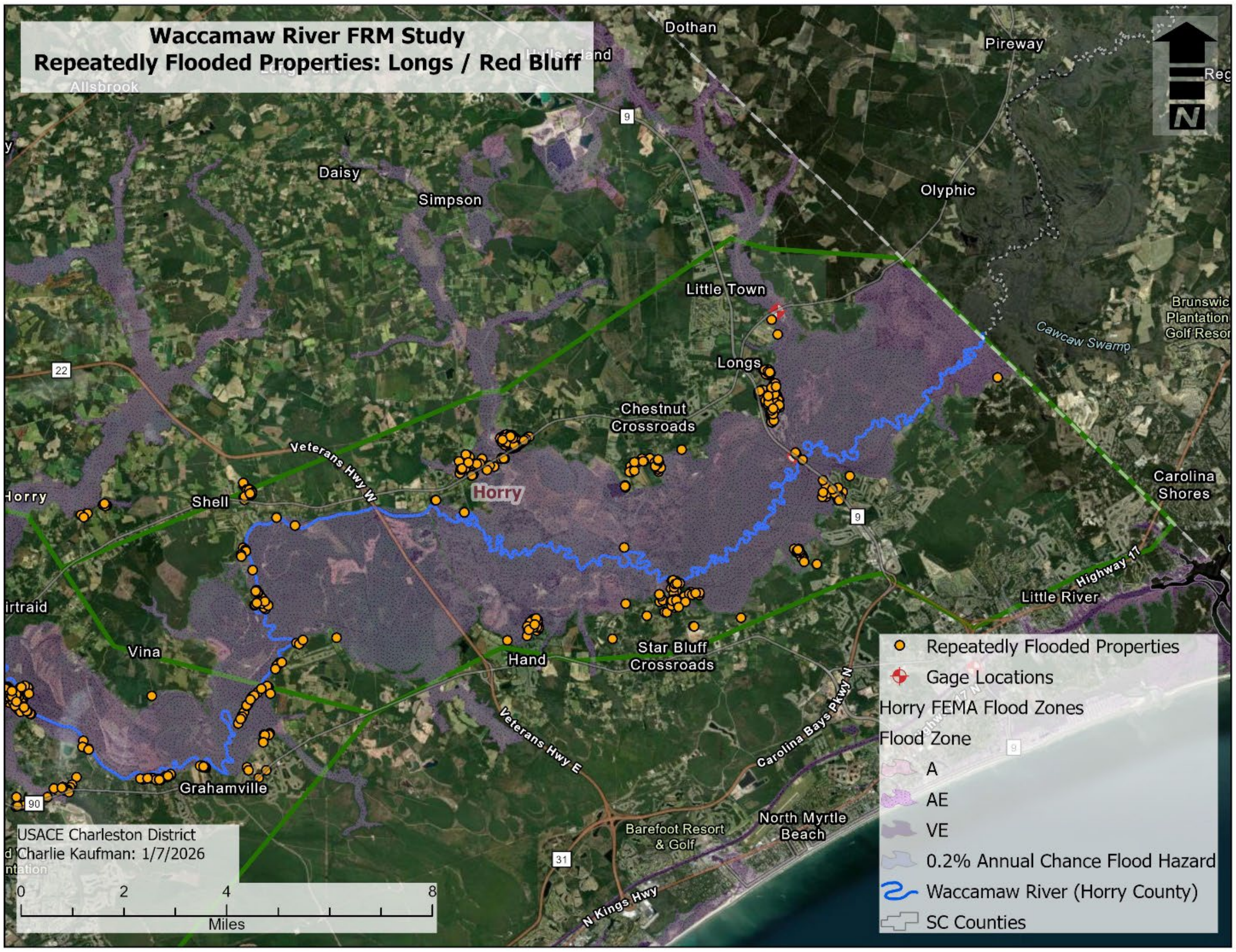


Figure B-21. Repeatedly Flooded Properties in Longs/Red Bluff

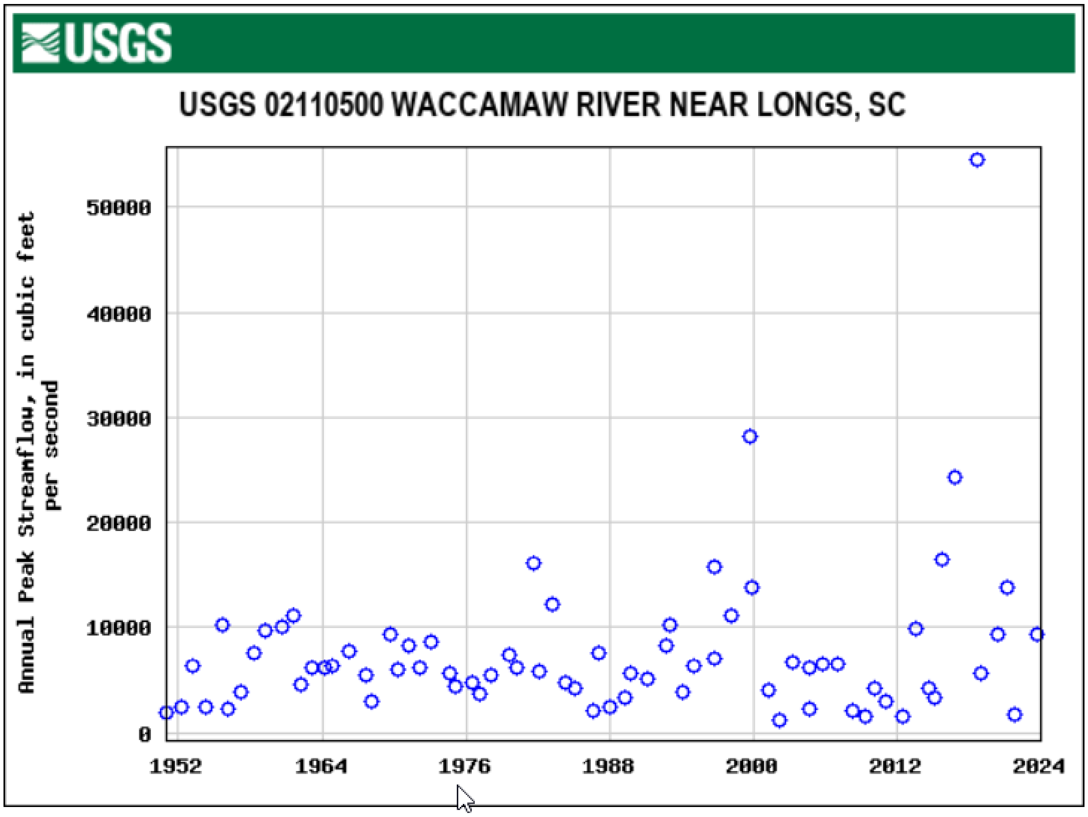


Figure B-22. Annual Peak Streamflow for Waccamaw River Near Longs, SC USGS Gage 02110500.

Table B-7. 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

Date	Gage Height (ft)
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

B.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 B-23**). 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.

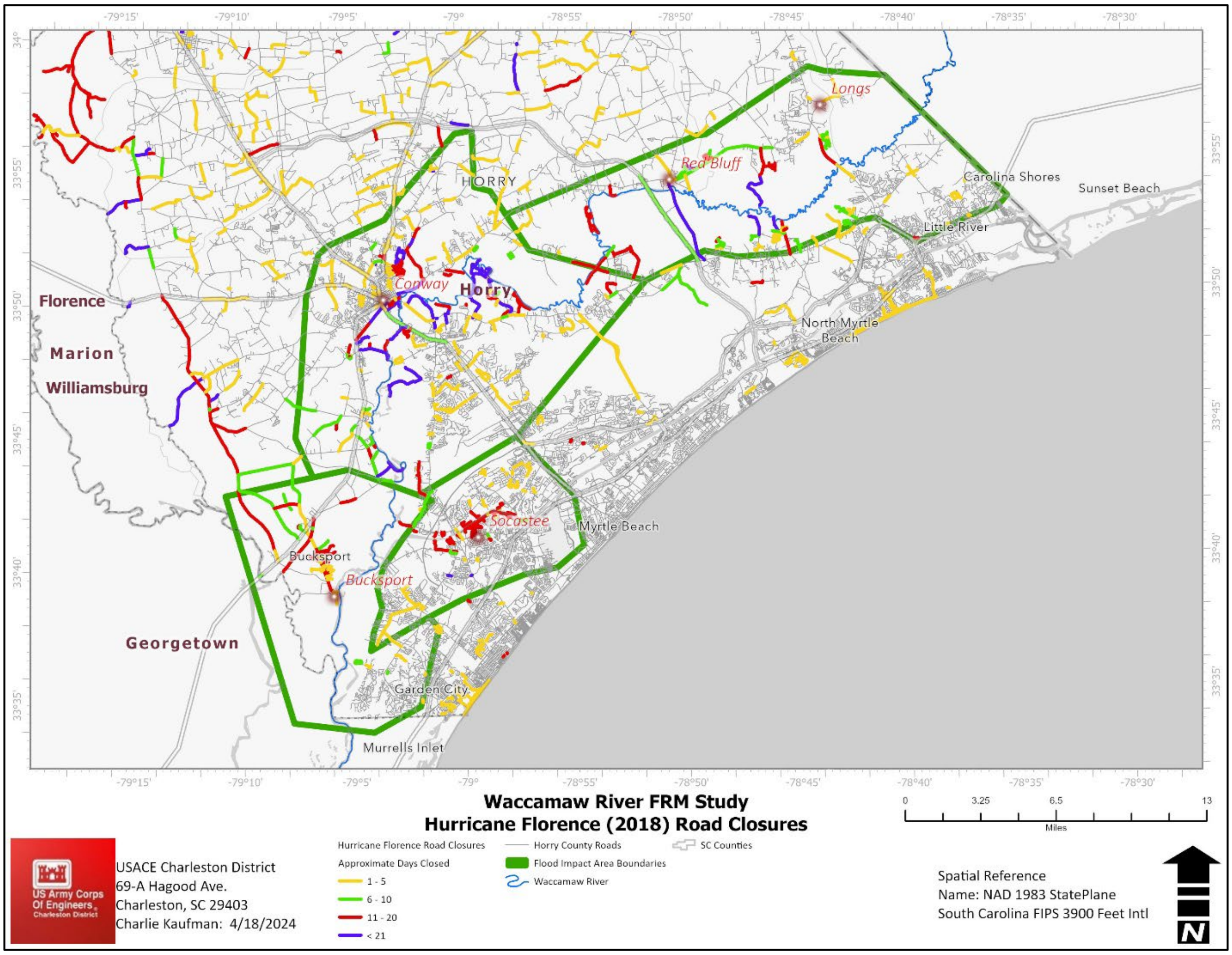


Figure B-23. Hurricane Florence Road Closures.

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 HWY 9 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.

C DATA COLLECTION

C.1 Hydrologic Data

C.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 C-1** provides a summary of available data for select USGS sites that were utilized for the purposes of this study.

Table C-1. 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	2019-2024*	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

C.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 C-2** provides the curve numbers for each land cover and soil type combination.

Table C-2. 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

C.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 Ta**ble C-3**.

Table C-3. 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 C-1. Precipitation Gage Station locations.

C.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 C-2** 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 meandering river. Restricting the natural channel meander, would inhibit natural stream flow.

Figure C-2. 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 C-4** lists the data provided by USACE Charleston District (USACE SAC) and data gathered from outside sources used to build the model geometry and its associated reference layers.

Table C-4. 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

Data Name	Data Type	Source	Notes
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

C.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).

C.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 C-3** 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.

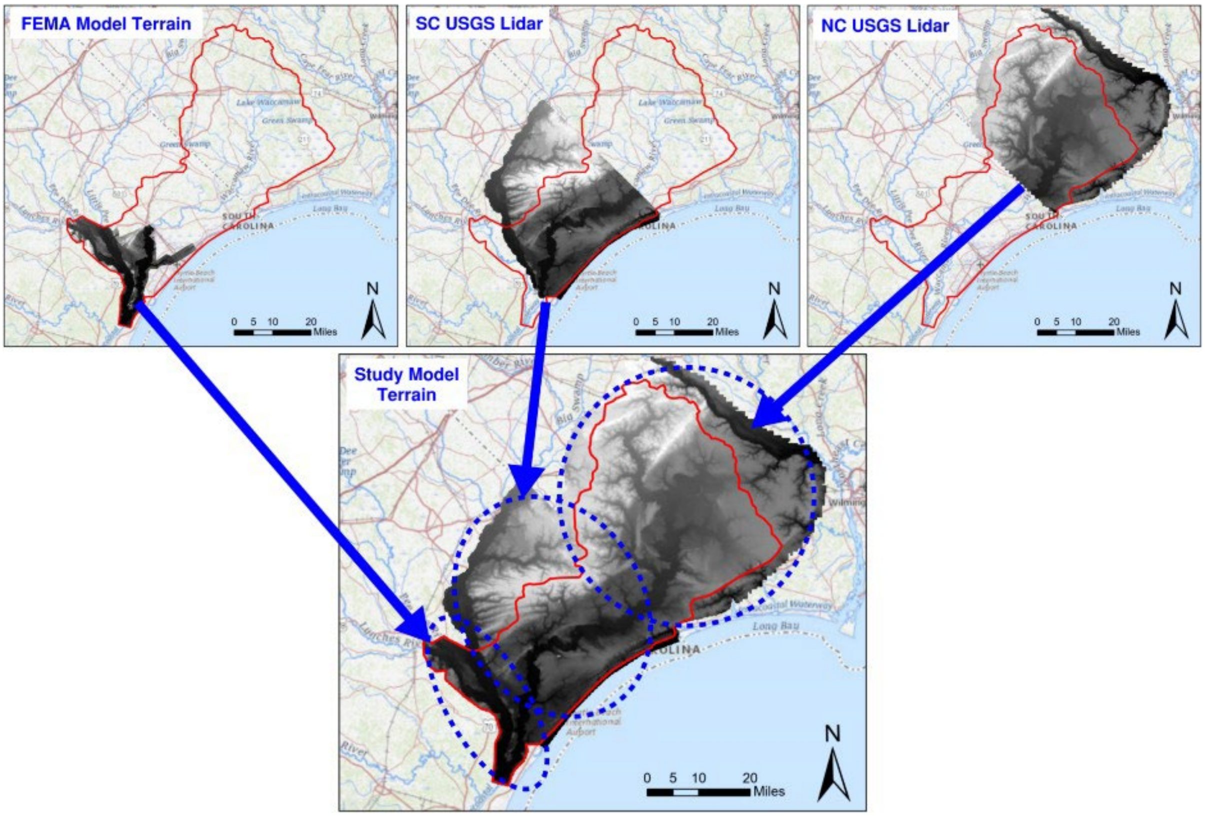


Figure C-3. Lidar Data Used to Create the Model Terrain File.

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 C-4** shows the extent of each bathymetry type incorporated into the model terrain.

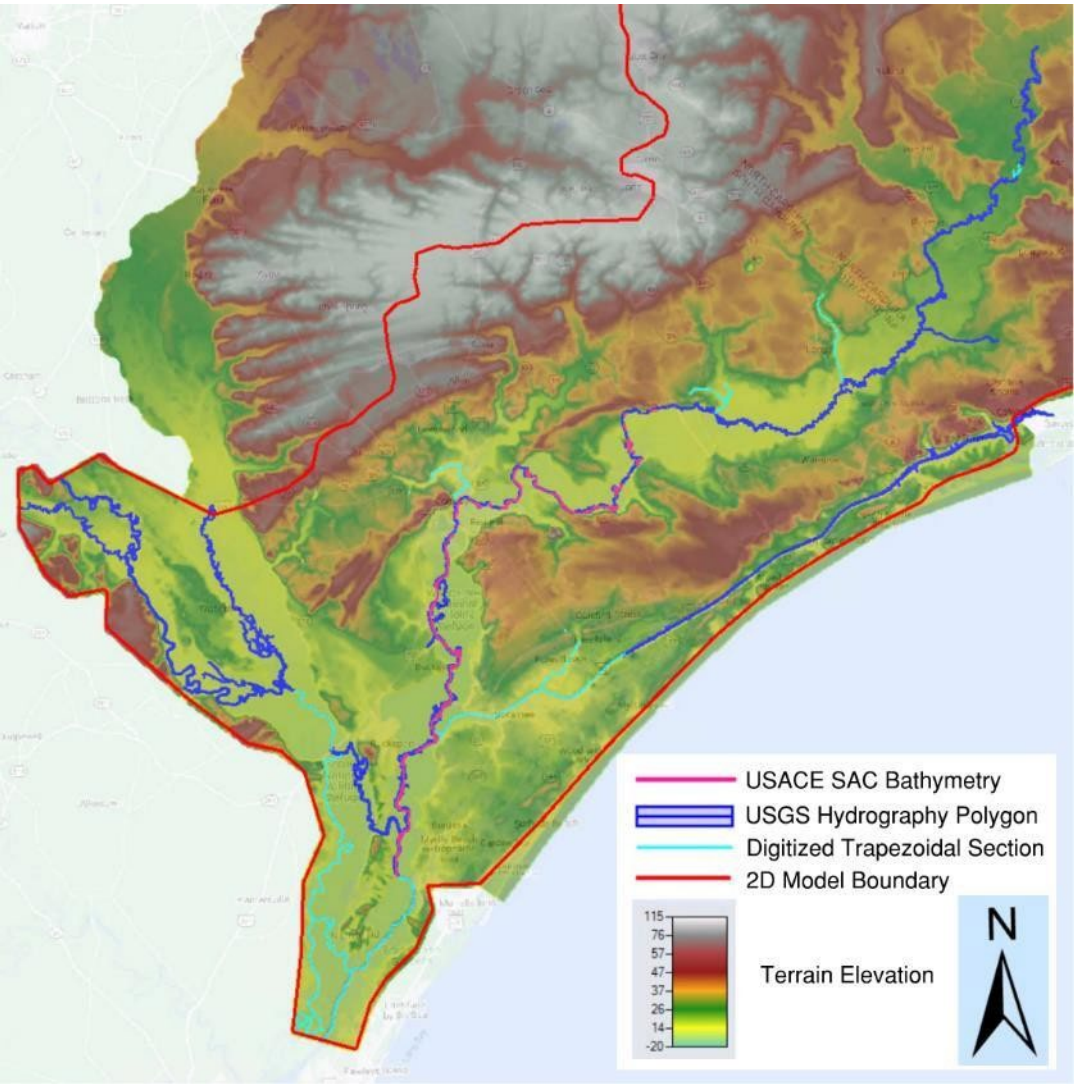


Figure C-4. Bathymetric Data Extents.

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.

C.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 SAC.

USACE SAC was provided the bridge as-built information for select bridges within the area of interest from SCDOT. **Table C-5** lists the floodplain crossings for the data provided by SCDOT, which includes multiple bridges in some cases. The PDF file names associated with each bridge in the crossing are provided. All of the bridge crossings within Horry County of the seven bridges Waccamaw River are included in the model domain. There are also two bridge crossings of tributaries that are included in the model domain. Additionally, the bridges that were modeled and used for the bridge sensitivity check, discussed in the Model Sensitivity and Calibration section E.1 are marked with an asterisk.

Table C-5. 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*

D HISTORIC EVENTS

D.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.

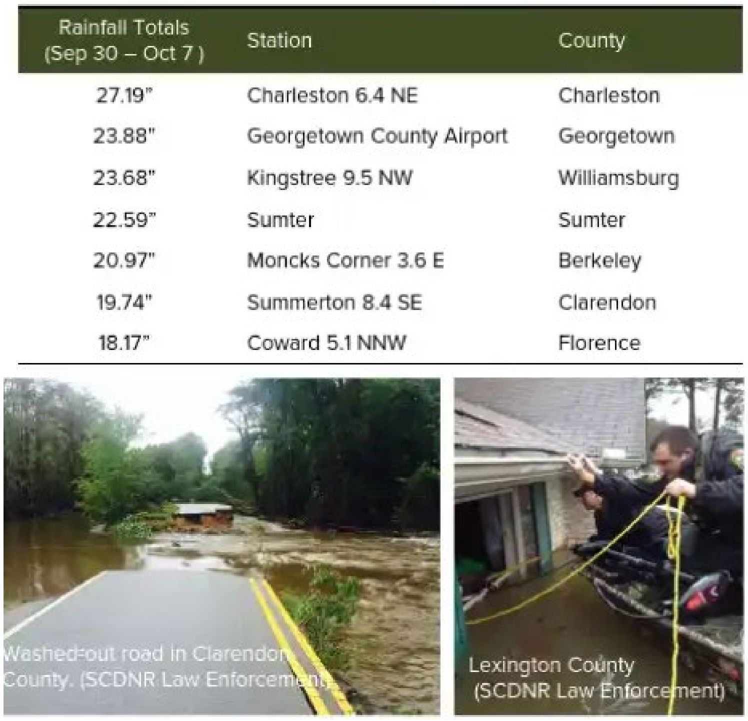
Table D-1 provides a list of historic flooding events prior to 2015 in the Waccamaw River basin adapted from a recent SCDNR publication: South Carolina Extreme Events Timeline.

Table D-1. 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

D.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 D-1**). Streams and creeks swelled out of their banks with 17 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 D-1. 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 D-2**).

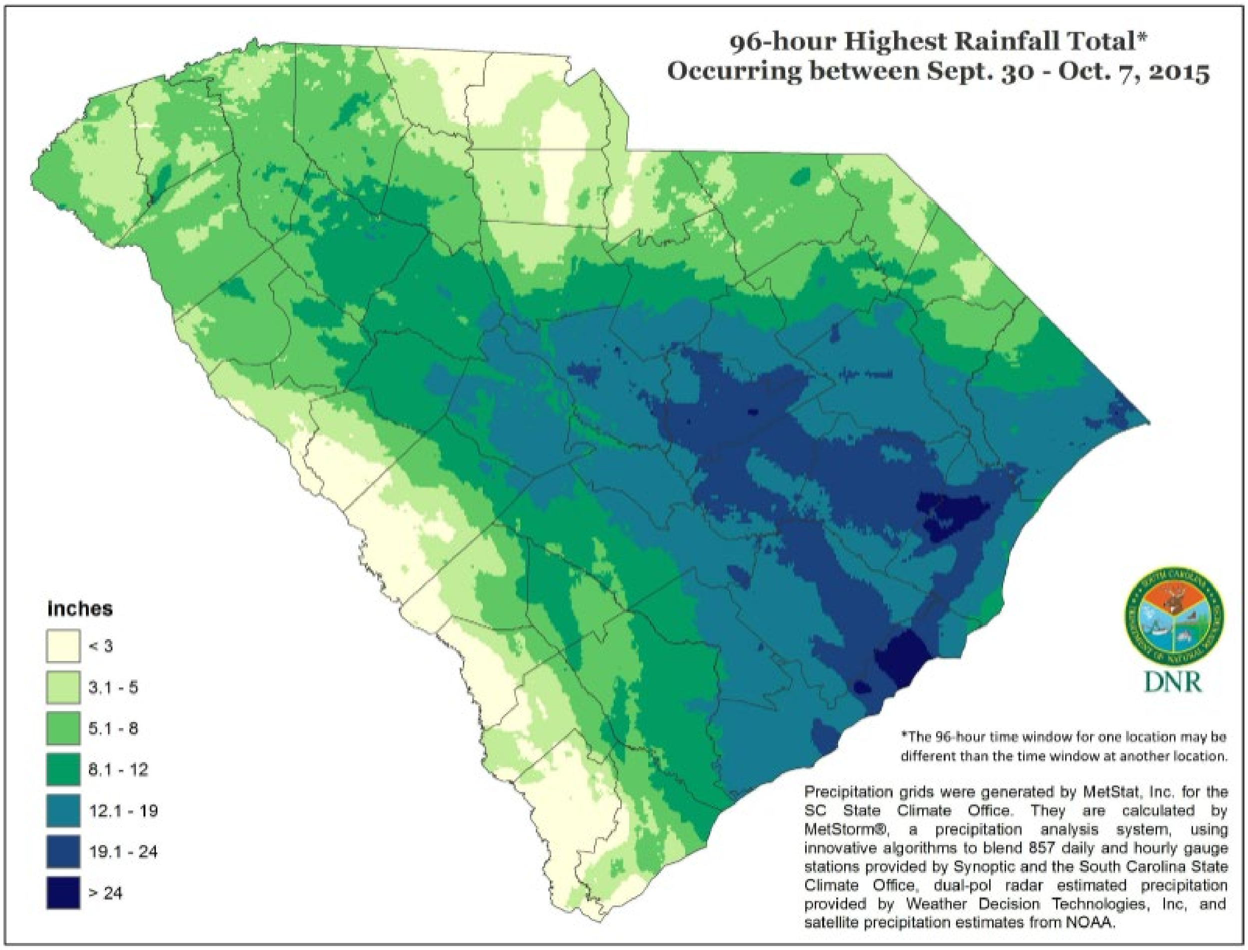


Figure D-2. 96-hour Highest Rainfall Totals, Sept. 30 - Oct. 7, 2015 (SCDNR).

Streams and rivers swelled out of their banks and 17 USGS gages reached record peaks including the Black River at Kingstree (Tab**le D-2**) and Conway Marina (**Figure D-3**).

Table D-2. 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)

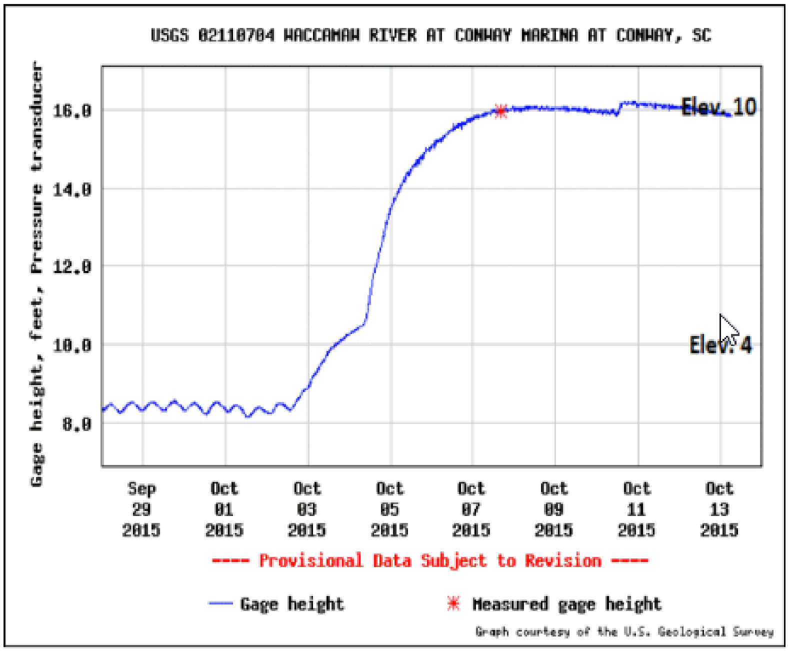
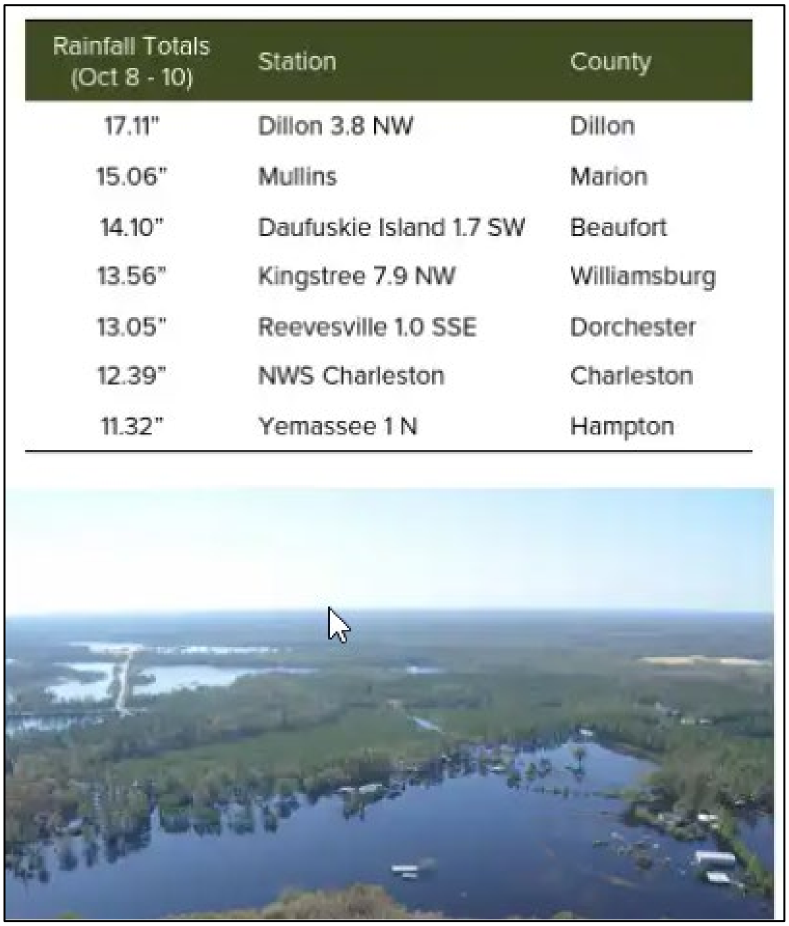


Figure D-3. Gage number 02110704 during Hurricane Joaquin

D.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 D-4**).



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 D-4. 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 D-5**).

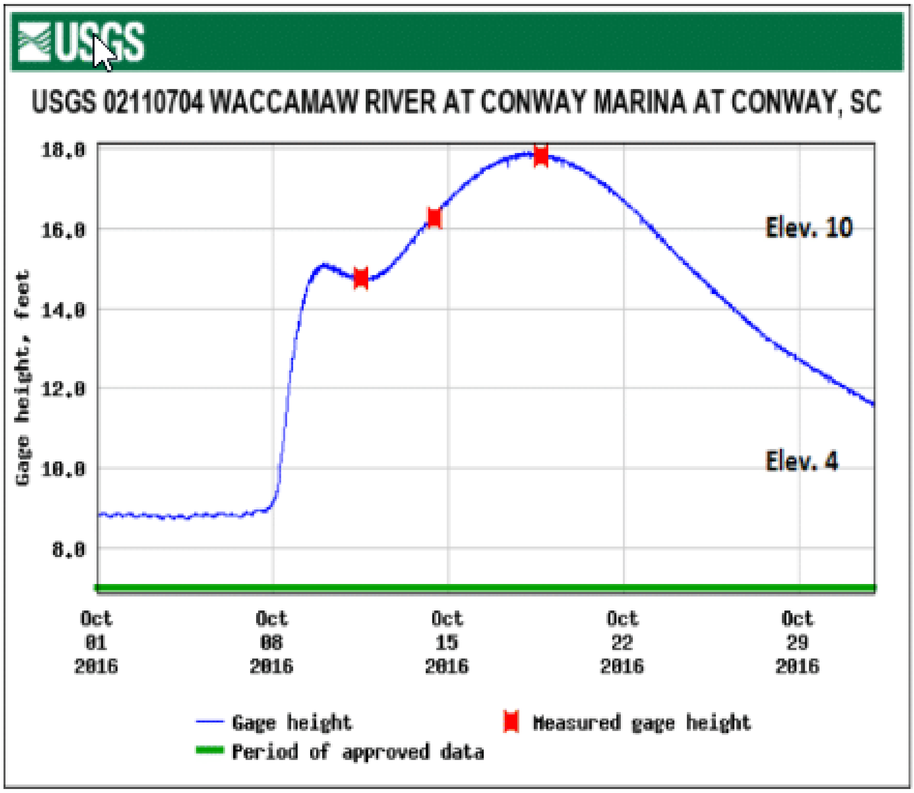


Figure D-5. USGS gage 02110704 during Hurricane Matthew.

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 D-6** shows the rainfall accumulation from Hurricane Matthew along the southeastern portion of the United States.

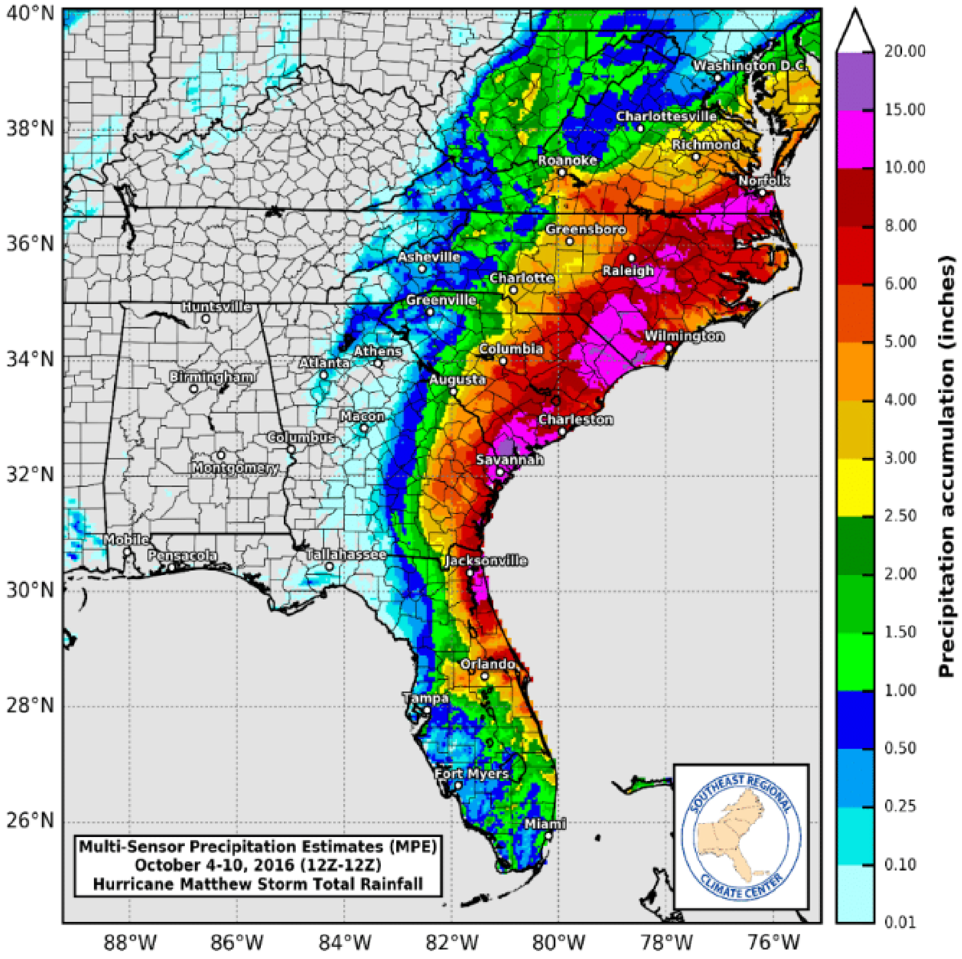


Figure D-6. Precipitation Estimates during Hurricane Matthew.

D.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 were 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 to 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 D-7**.

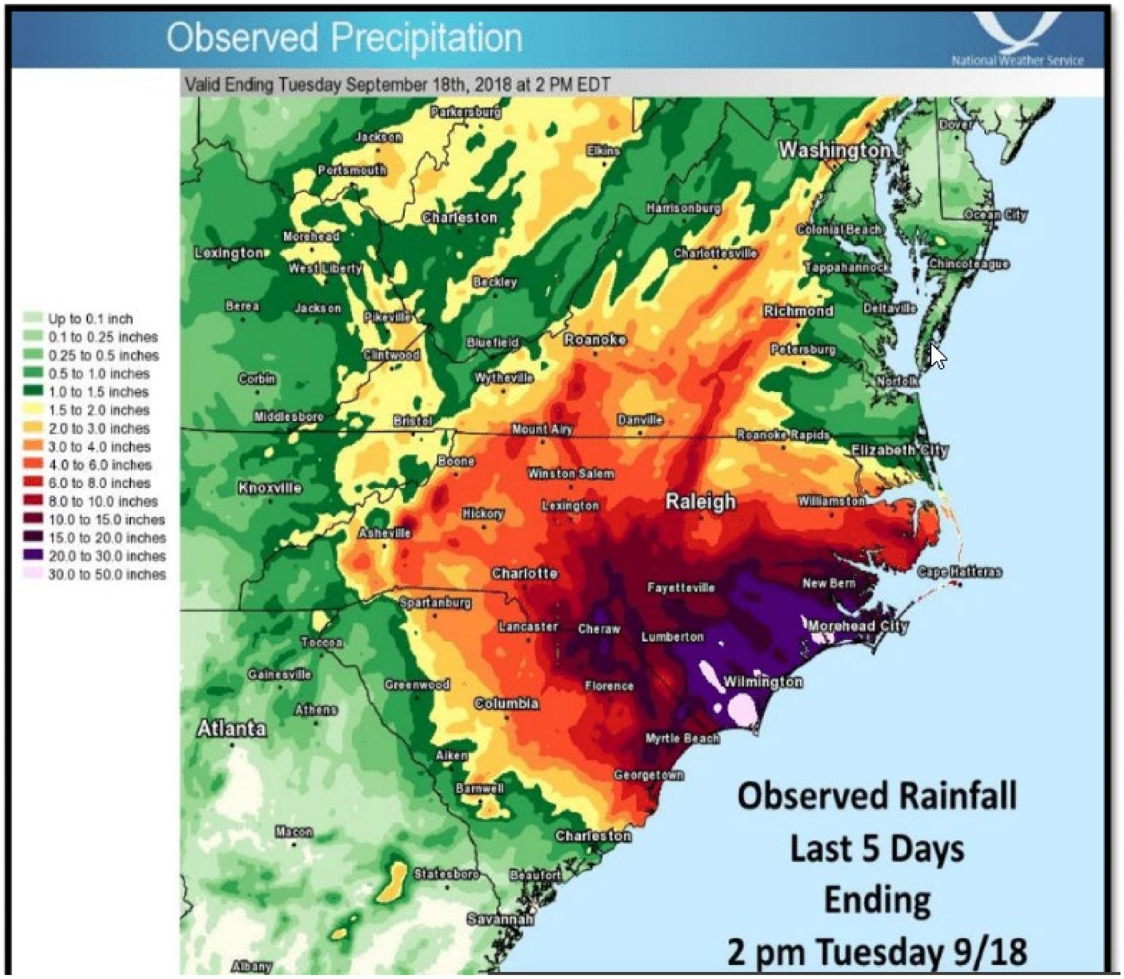


Figure D-7. National Weather Service - Hurricane Florence Observed Precipitation.

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 gage near Longs on September 21, near Conway on September 26, and at its downstream gage 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 D-3** lists flood crests from Florence compared to previous flood crests (SCDNR).

Table D-3. Florence vs Historical Crests at Selected USGS Gages (Horry, 2019)

River Gage	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 gages 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 gage 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), gages 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 gages 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. Gages along the Waccamaw exceeded previous record crests by three or more feet during this event. **Figure D-8** 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.

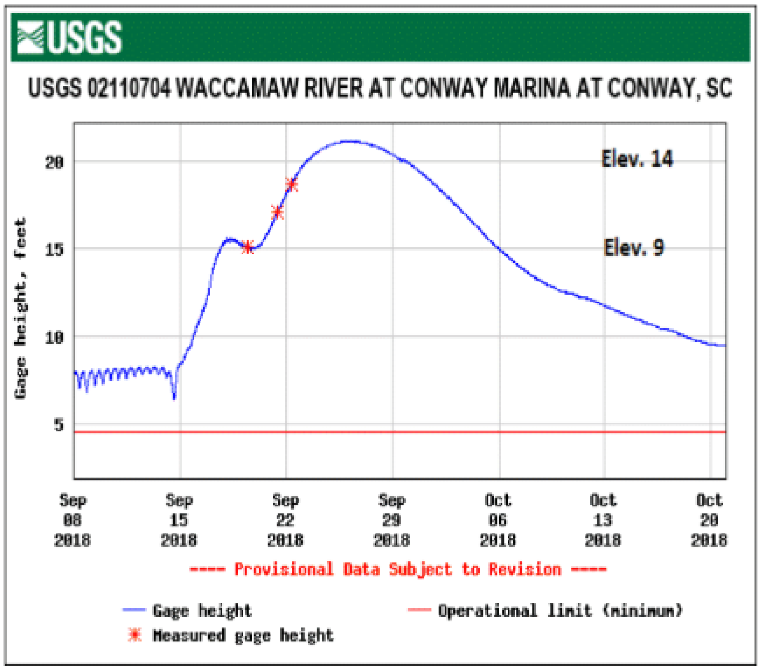


Figure D-8. USGS Gage 02110704 Waccamaw River at Conway gage height during Hurricane Florence.

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 D-4**.

Table D-4. Rainfall Totals for Hurricane/ TS Debby in Horry County

Location	Rainfall Totals (in)
Loris	15.89
Waccamaw River	14.89
North Myrtle Beach	14.25
Finklea Fire Station	13.1
Little River	13.06
Bucksport	12.64
Horry County Police	12.57
Aynor	10.85
Garden City	10.35
Conway	10.21
Allsbrook	9.52
Briarcliffe Ac	9.12
Galivants Ferry Land	9.07
Nichols	7.9
Red Hill	8.65
Crab Tree Swamp	8.51
North Conway	8.19
Waterford Plantation	7.99
Central Horry County	7.3
Surfside Beach	7.22
Socastee	6.02
Hunting Swamp SC	5.7
Longs	4.72

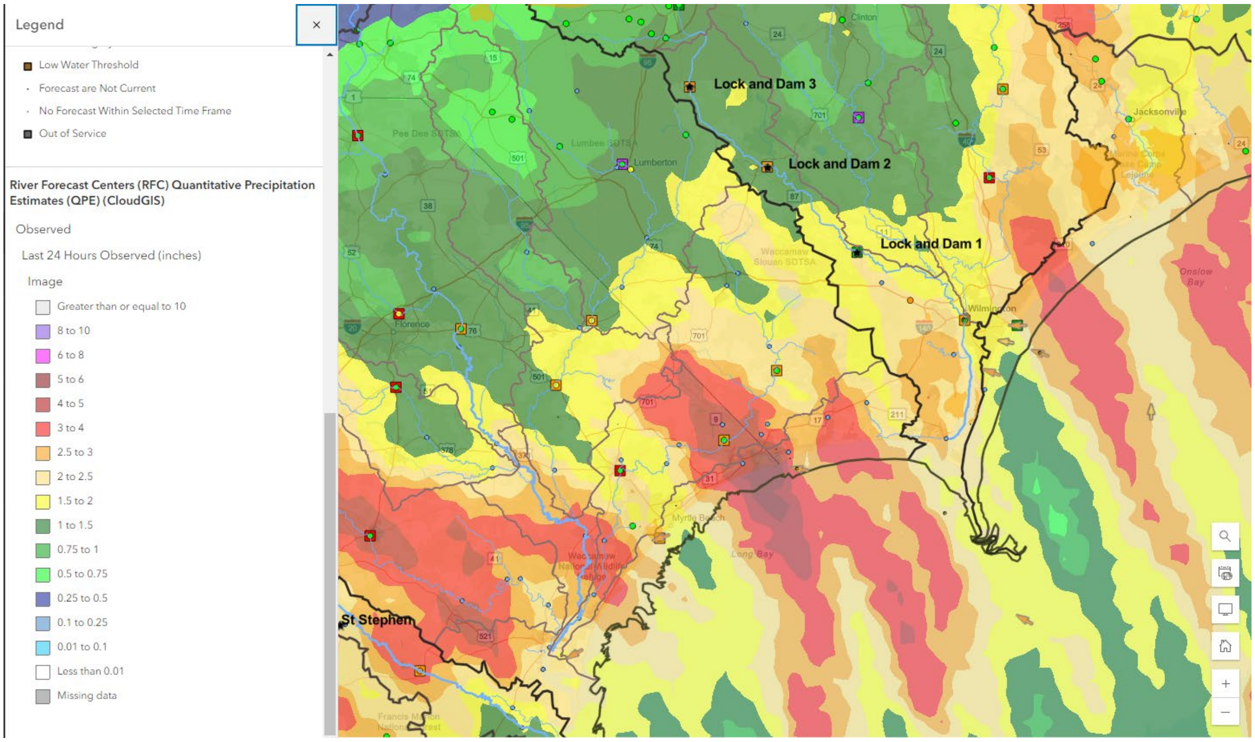


Figure D-9. 24 Hour Quantitative Precipitation Estimate (Aug. 6th, 2024).

As of August 20, 2024 the Waccamaw River near Conway showed a peak of 14.90 ft, which is a Major flood stage as shown in **Figure D-10**.

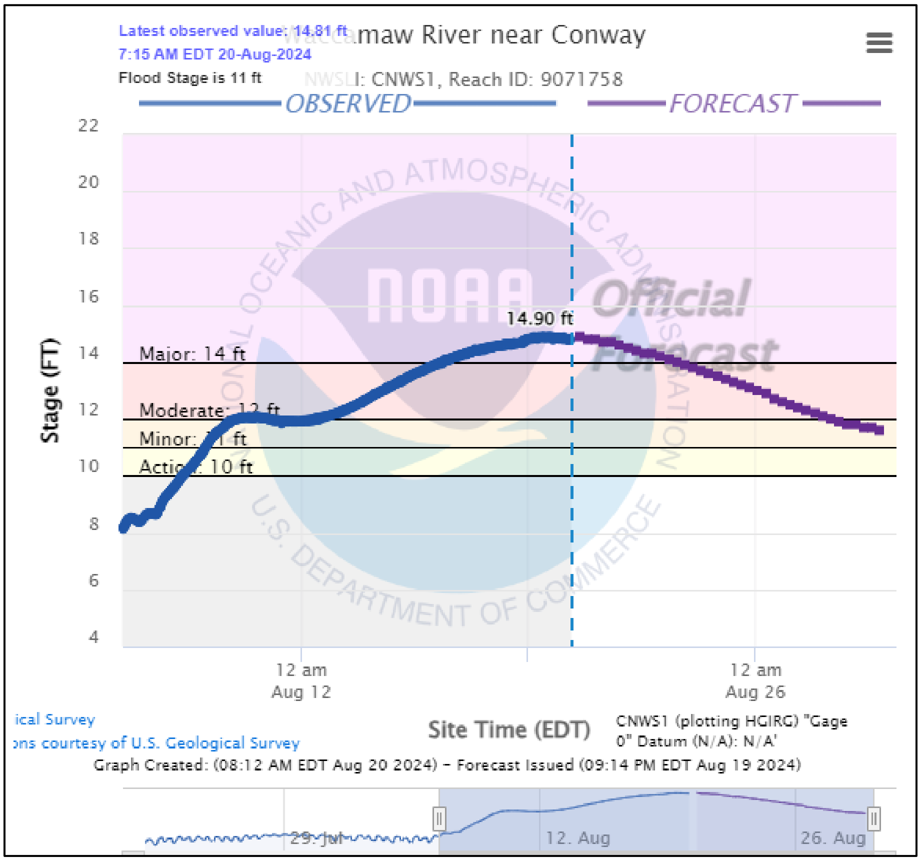


Figure D-10. River Forecasting Center report for USGS Gage Waccamaw River near Conway as of August 20, 2024.

Similar to Hurricane Florence and Matthew, an additional and larger peak was observed on August 10, at 12.06 ft and then receded and another peak was measured at 14.90 ft on August 19th. The gage information is shown in **Figure D-10**.

D.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 D-5, Figure D-8** and **Figure D-10**. 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 E.2 of this appendix.

E EXISTING CONDITIONS

E.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 AIWW 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 period of record (2003-2007). Streamflow data from two 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 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. 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 important area.

E.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 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. In regard to land use changes and increase in development, we lacked certainty that any particular development would happen for FWOP, therefore, basically no change between existing conditions (EC) and FWOP for the study area.

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.

E.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.

The decision to use Diffusive Wave (DWE) instead of Shallow Water Equation (SWE) was based on the specific characteristics of the Waccamaw River system. As the river is slow-moving, backwater-dominated, and characterized by a low Froude number with limited flow variability, it fits well within the parameters where Diffusive Wave is more appropriate. For conditions like these, using SWE can often introduce unnecessary numerical instability without improving accuracy, and in fact, it may degrade the model's stability and reliability due to the inclusion of inertial terms that are negligible in this context. While SWE is more accurate for rapidly varying flows, steep slopes, or areas where inertia plays a significant role (e.g., high Froude numbers, rapid transitions between flow regimes), we don’t have these conditions in our case. The Diffusive Wave approach, which focuses on gravity and friction-dominated flows, provides more stability and computational efficiency for slow-moving, backwater-dominated systems. Given the size of the model and the type of flow being modeled, SWE would likely increase computation time without adding value and could even introduce more instability.

To further justify the usage of DWE, we conducted a comparative analysis for the calibration case of Hurricane Florence, focusing on several USGS gage height locations (02110400, 02109500, and 02110500) in both tributary and main channel settings of the Waccamaw River.

Our findings revealed that while the SWE model consistently overpredicted the measured values, the DWE model demonstrated a more accurate alignment with observed data. Specifically, the increase in predictions from the SWE model was not significant—less than 2%—indicating that although both models showed some discrepancies, the DWE provided a closer approximation to actual measurements.

This comparison underscores the suitability of the DWE for this study, as it appears to capture the hydrodynamic behavior of the system more effectively than the SWE. We considered both approaches and determined that Diffusive Wave was the best fit for this application.

Synthetic rainfall events were developed to assess the 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 E-1** in Section B.5. Early coordination with the Flood Risk Management Planning Center of Expertise (FRM PCX), guided the PDT to analyze and determine the duration of storms within the region. The charts of accumulated precipitation, shown in **Figure E-1** and **Figure E-2**, 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. Preliminary results for the existing conditions model were highly sensitive to the initial flow assumptions, which indicates a 24-hour duration is not sufficient. Therefore a 96-hour storm was selected for this region to accurately depict the events.

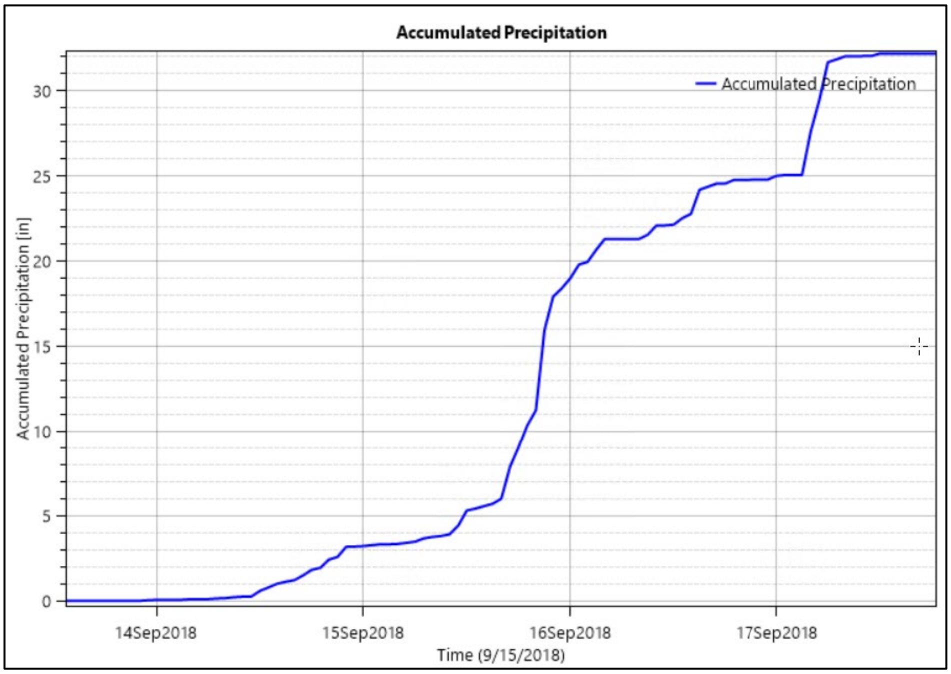


Figure E-1. Approximate Maximum Accumulated Precipitation Point for Hurricane Florence.

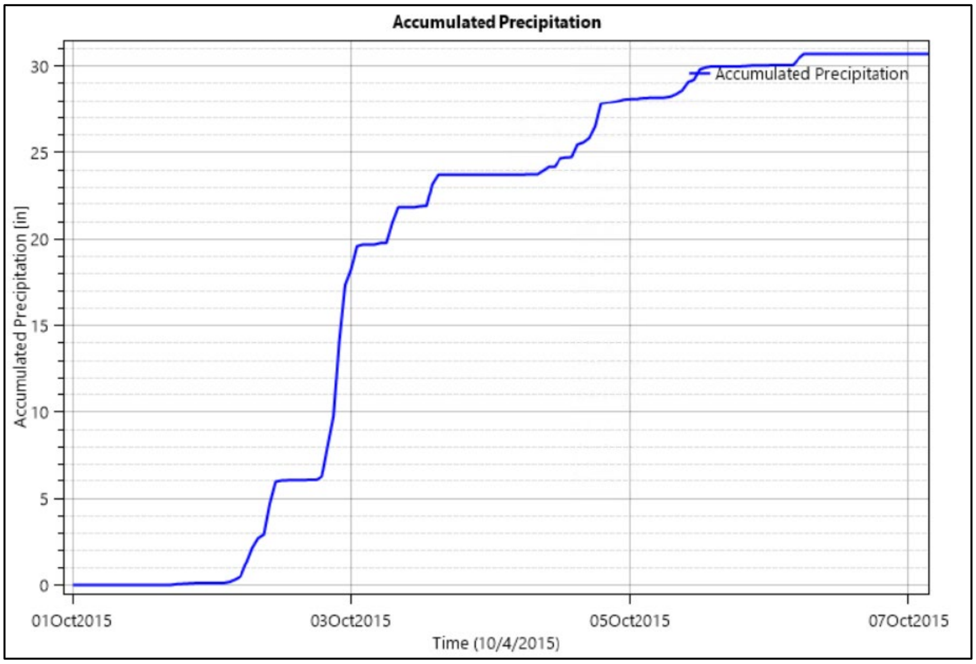


Figure E-2. Approximate Maximum Accumulated Precipitation Point for Hurricane Joaquin.

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 E-3**.

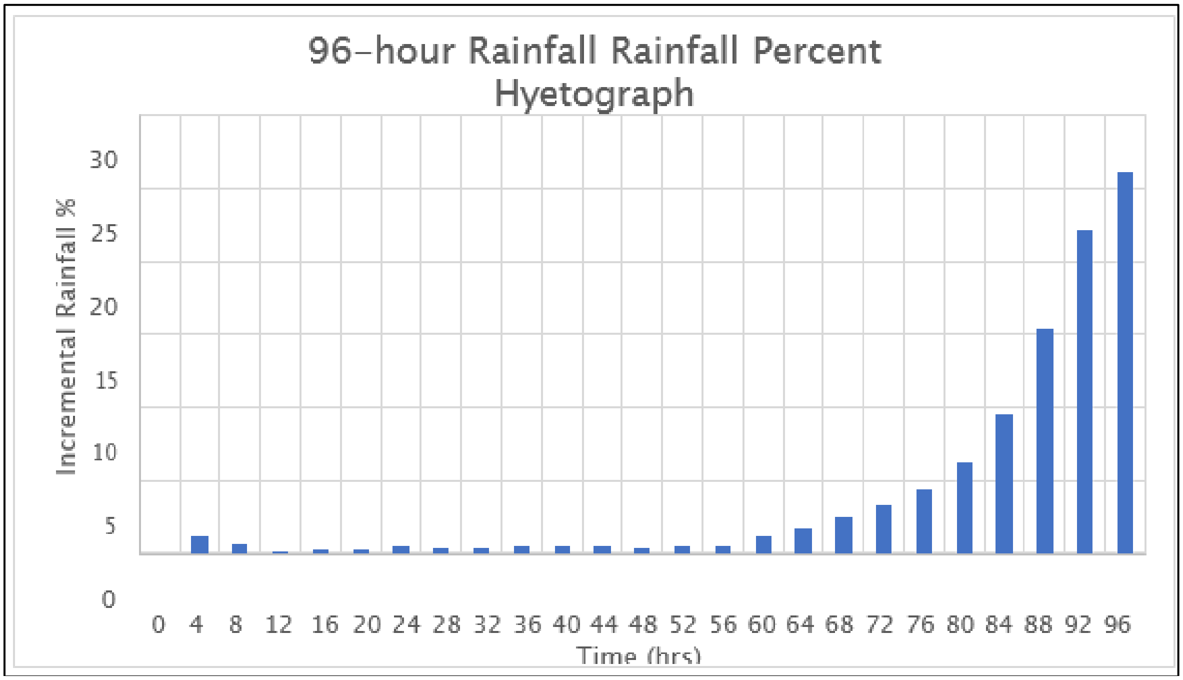


Figure E-3. Rainfall Percent Hyetograph for 96 Hour NOAA Atlas 14, Quartile 4 90% Decile Event.

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. To ensure that rain-on-grid could be accurately modeled across the entire watershed, initial runs were conducted with terrain modifications to remove culverts, bridges, and other obstacles in the small tributaries, allowing for realistic flow paths. These initial simulations helped identify areas where flow behavior could be improved. The model was then refined through a calibration process, adjusting the terrain and model parameters to better match observed flow conditions and improve the accuracy of rainfall distribution over the grid. This iterative approach ensured that both the main channels and smaller tributaries were accurately represented, and the rain-on-grid method effectively simulated the hydrology of the entire watershed.

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 E-4** below and **Table C-2** in Section B.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.

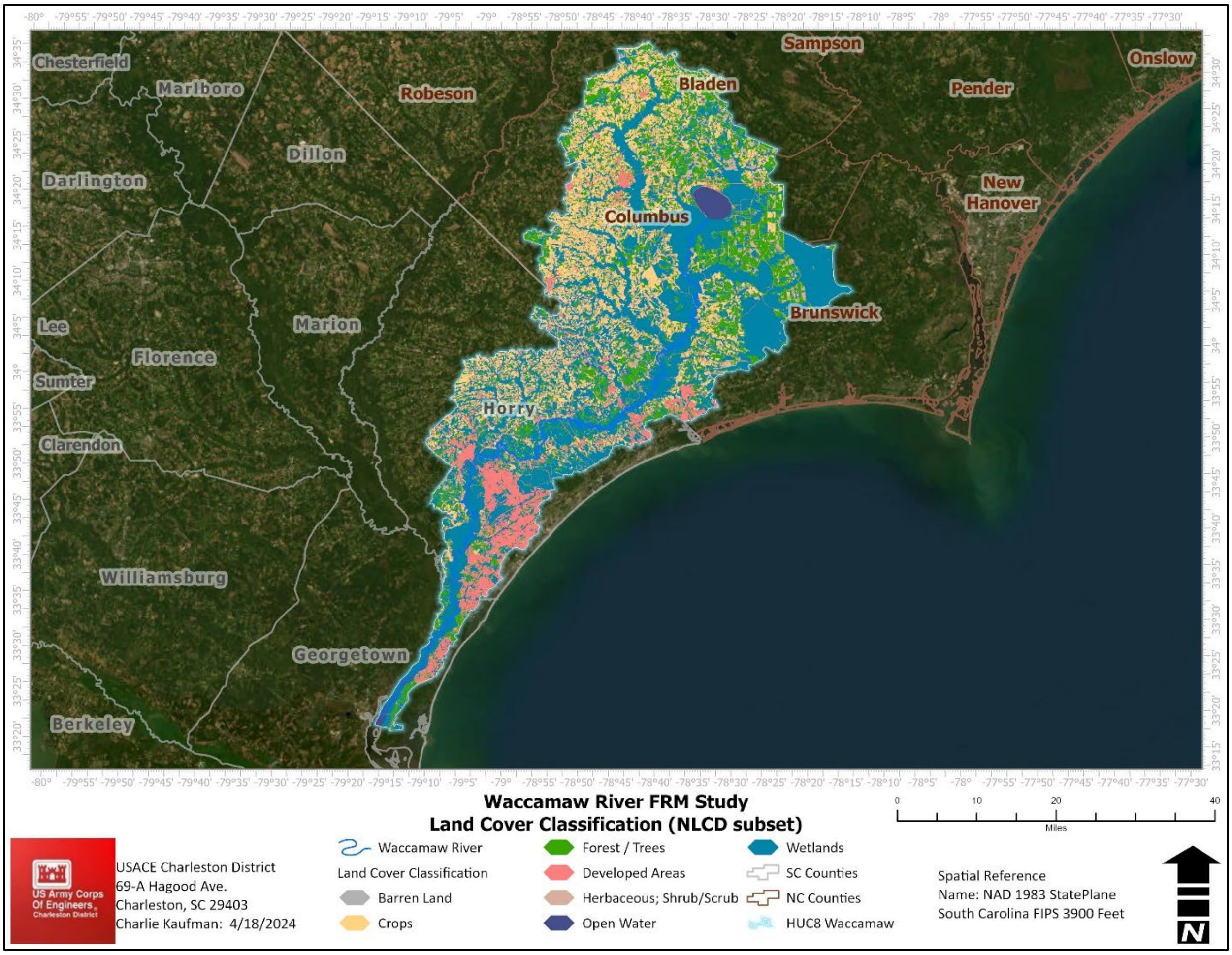


Figure E-4. Land cover classifications from NLCD2019 for the Waccamaw River Basin.

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 E.1 – Hydrology.

E.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 E-5**).

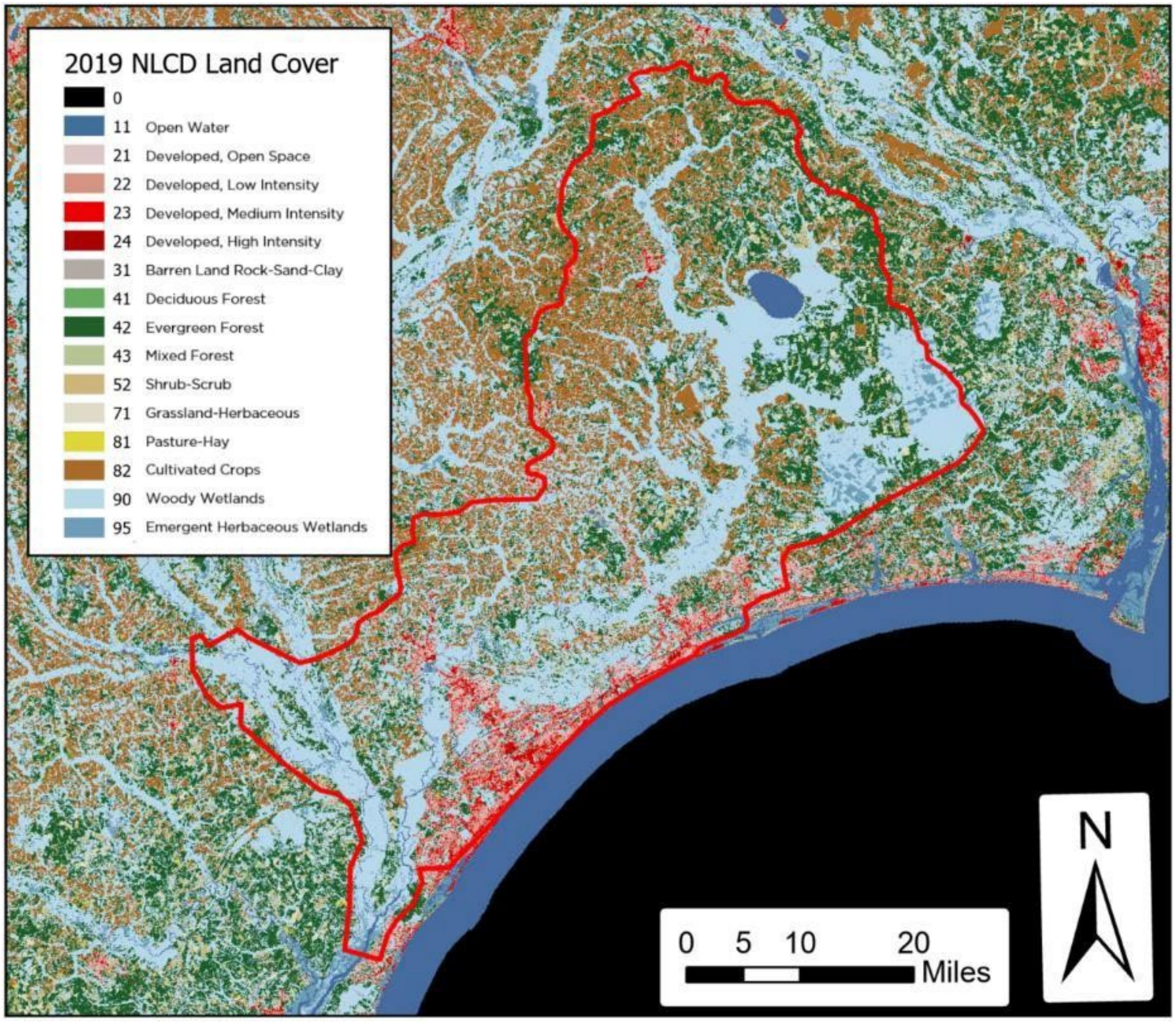


Figure E-5. NLCD (2019) for the project area.

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 the 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 E-6**. The precipitation values calculated from these grids within HEC-HMS is shown for each basin in **Table E-1**.

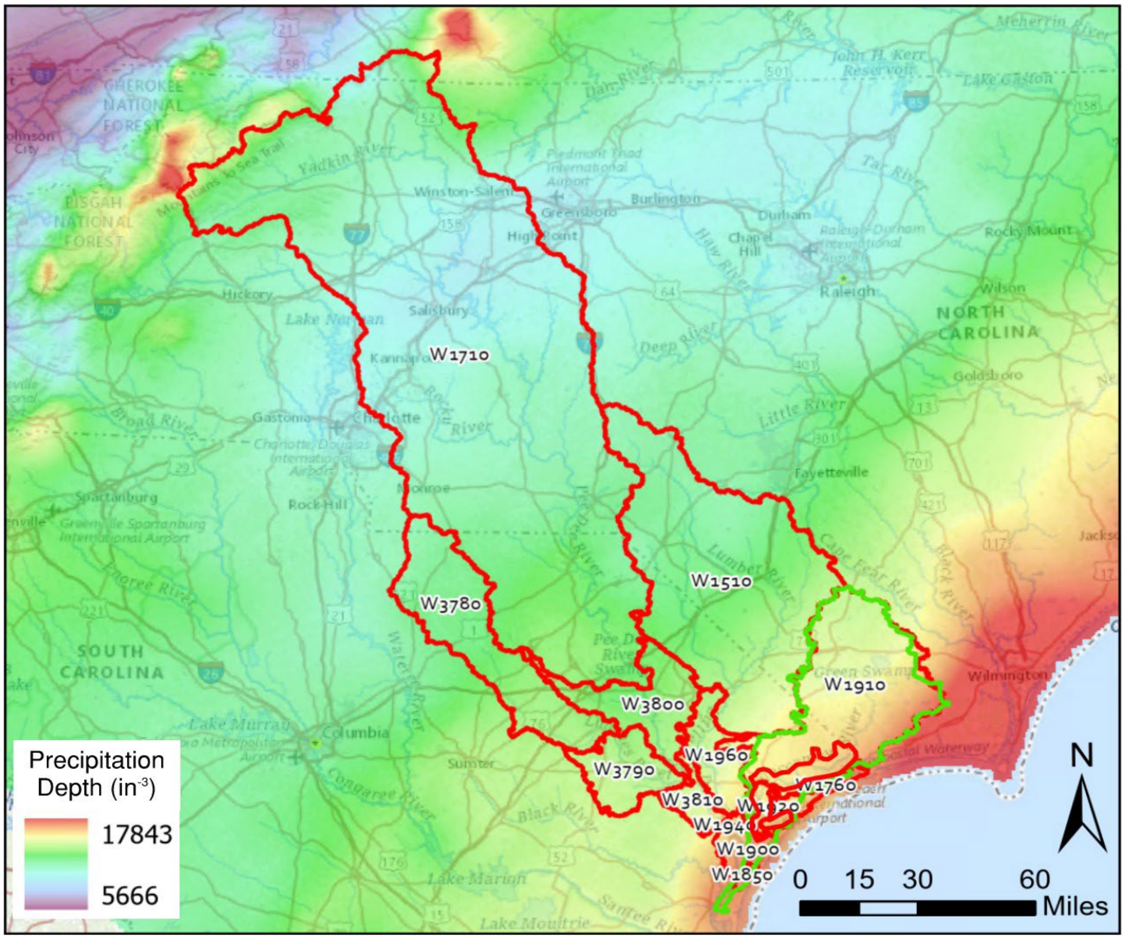


Figure E-6. Example of NOAA Atlas 14 GIS Precipitation Frequency Estimate Grid, 1% AEP.

Table E-1. Rainfall depths for each synthetic rainfall event for sub watersheds in Waccamaw River Basin

HEC-HMS Basin	Rainfall Depth (in.)						
	50%	20%	10%	4%	2%	1%	0.5%
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

HEC-HMS Basin	Rainfall Depth (in.)						
	50%	20%	10%	4%	2%	1%	0.5%
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 and Results Section E.3.4). The 1%-AEP, 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.

E.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 E-7** 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 1% AEP, 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 1% AEP 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 1% AEP 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 1% AEP 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 E-8** 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.

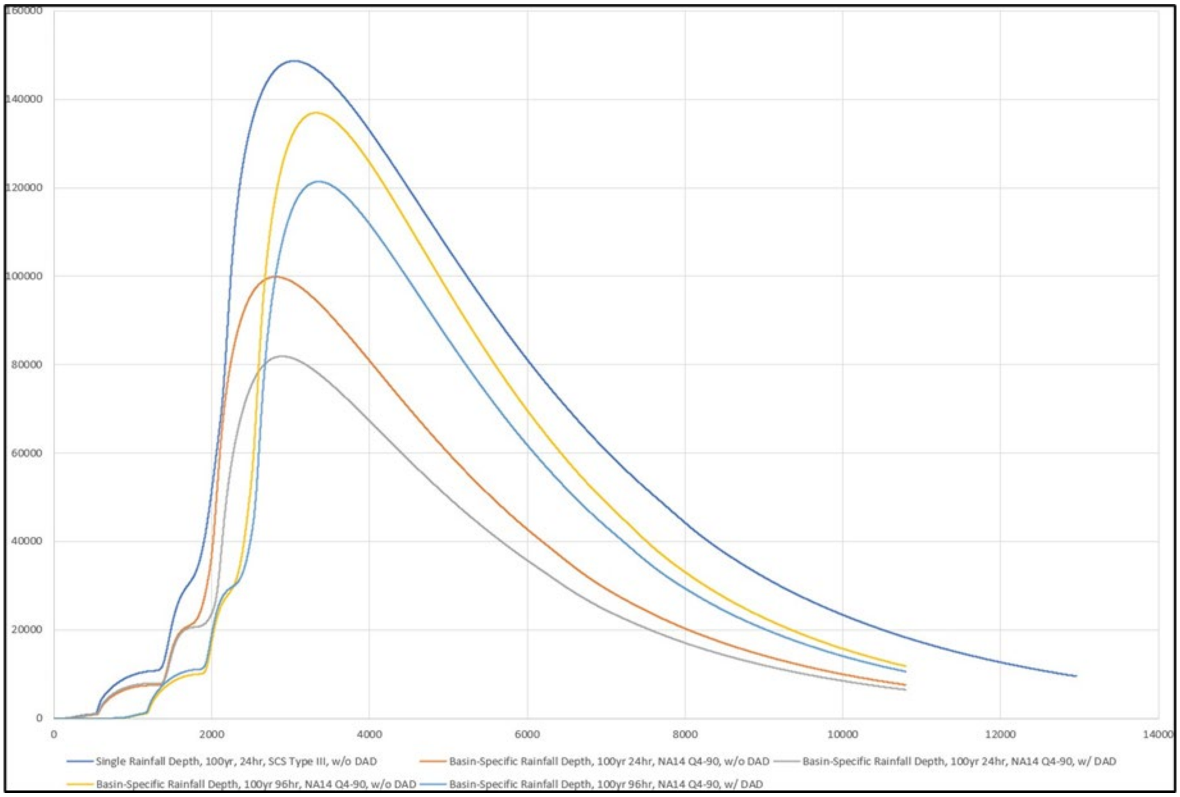


Figure E-7. Comparison of 1% AEP flow estimates with varying hydrologic assumptions.

Figure E-8 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.

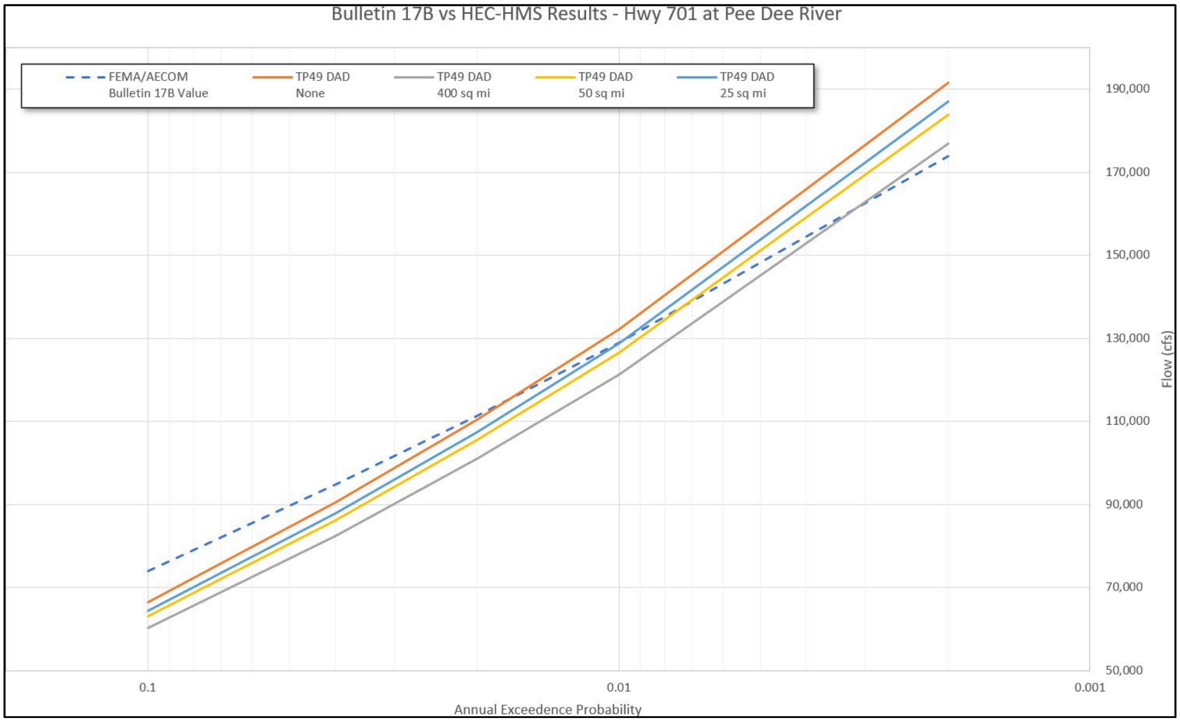


Figure E-8. Comparison of Bulletin 17B Stream Gage Analysis Results vs. HEC-HMS Results for Various Areal Reduction Storm Sizes (25 to 400 sq mil).

E.2 Hydraulics

E.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 E-2**.

Table E-2. 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 E-9** 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.

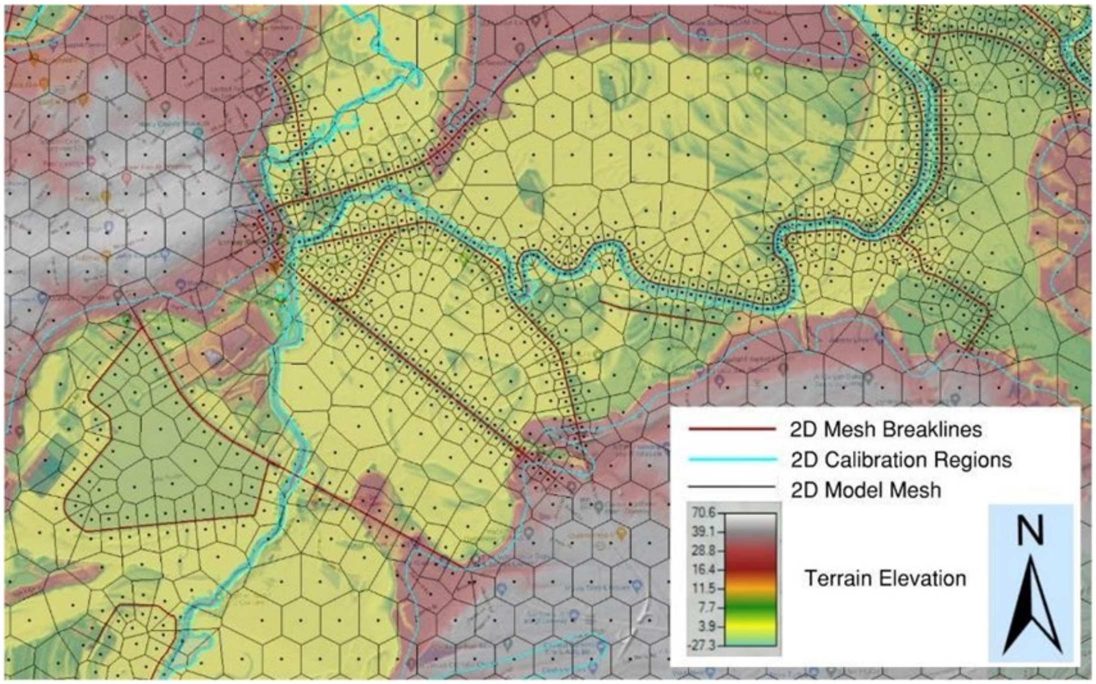


Figure E-9. HEC-RAS 2D Mesh Layout Example.

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 E-9**. 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 E.3 Model Sensitivity and Calibration section of this report.

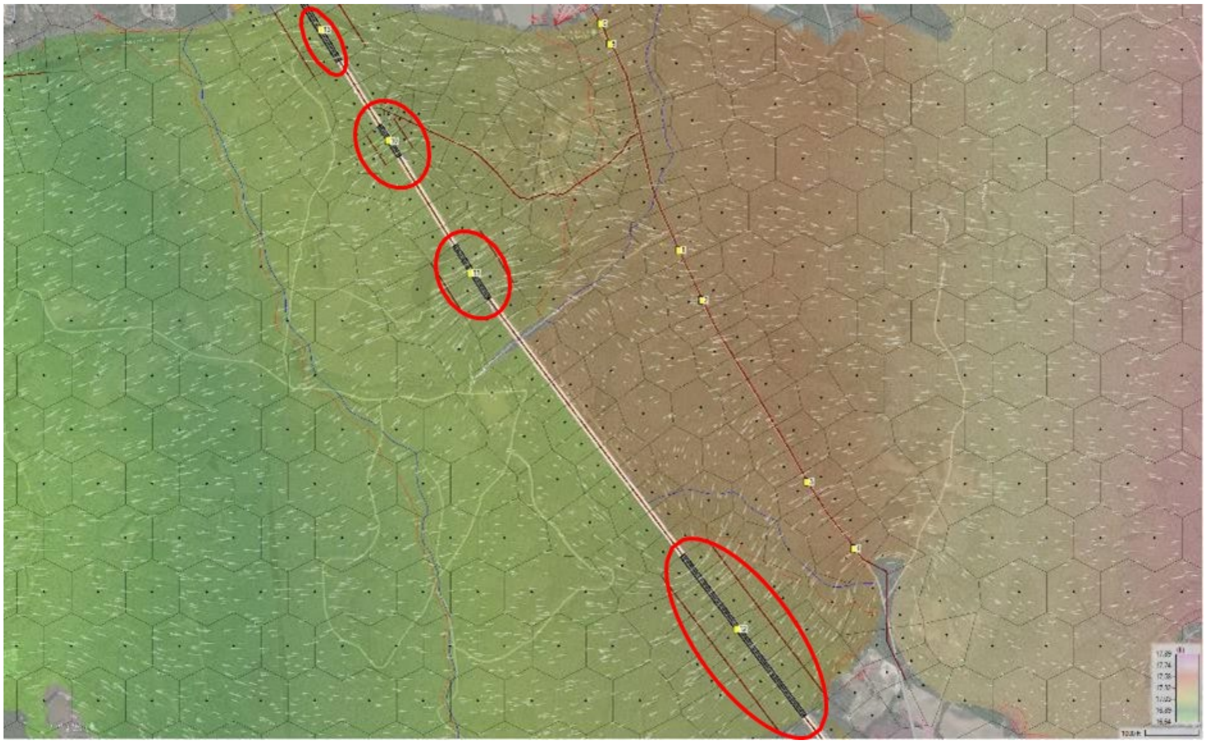


Figure E-10. HEC-RAS 2D Bridge Layout Example, Highway 22 Crossing.

Finally, the inflow and outflow boundaries were added to the mesh's exterior perimeter, as shown in **Figure E-10** with the major inflow and outflow boundaries labeled. The major outflow conditions include Intercoastal Waterway (ICWW) Outflow and Pee Dee River Outflow were set up as stage hydrograph.

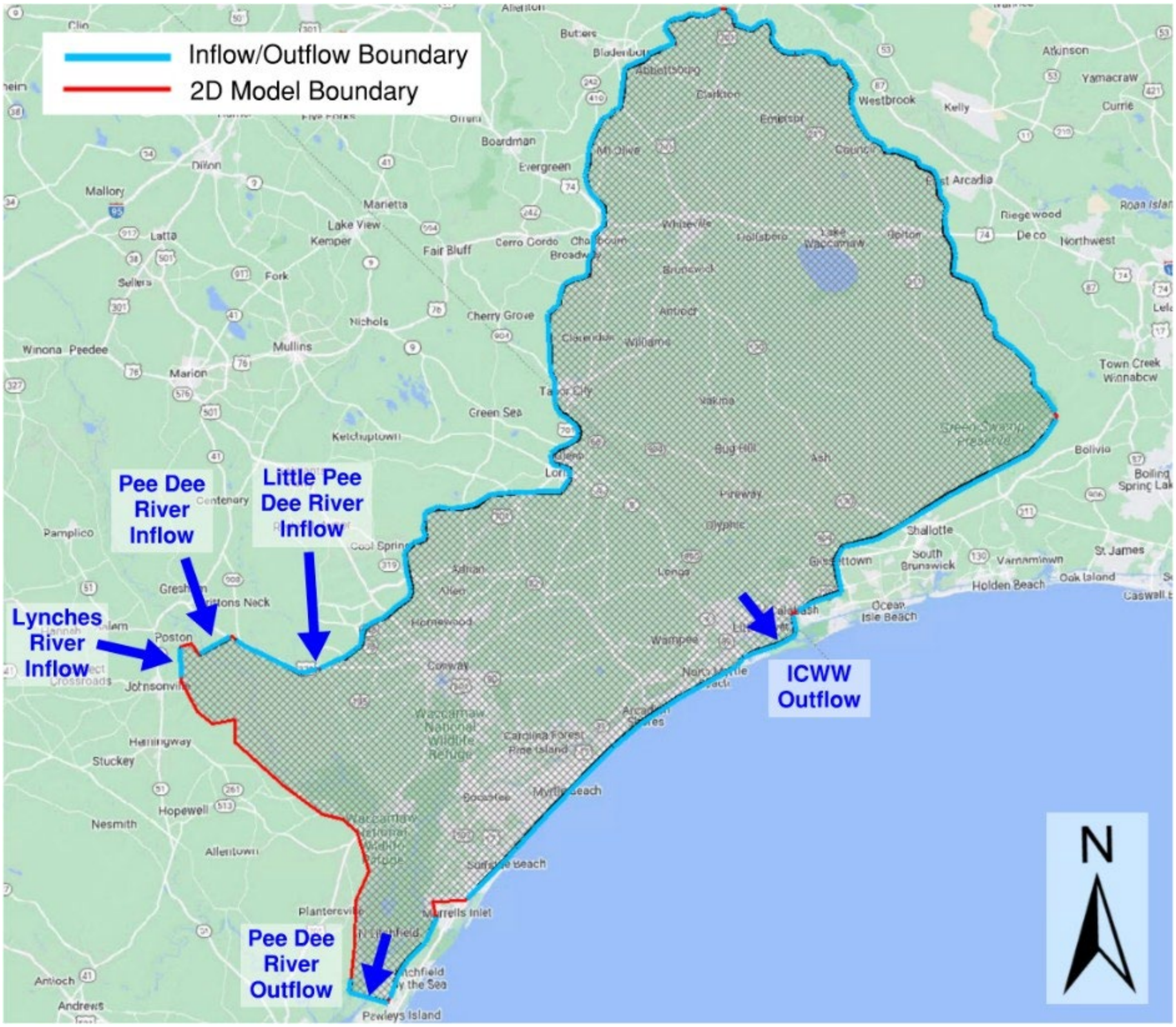


Figure E-11. HEC-RAS 2D Model Boundary Condition Lines.

E.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. Two calibration scenarios included Hurricane Florence (2018) and Hurricane Matthew (2016) and validation for 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. Hurricane Joaquin was a unique rainfall event, characterized by a single peak in the hydrograph, distinguishing it from Hurricanes Florence and Matthew. It was crucial to validate the model with this extreme event to ensure its accuracy under varied flooding conditions. 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 E-13**, **Figure E-25**, **Figure E-23**, **Figure E-24**, **Figure E-36** and **Figure E-37** 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 E-12** and the corresponding names and gage locations are shown in **Table E-3**. 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. Additional evaluation was performed to characterize the influence of boundary conditions on simulated flows at calibration gage locations. Streamflow hydrographs extracted at gage cross sections were decomposed to distinguish contributions from prescribed boundary inflows and internally generated runoff. Results indicate that while boundary conditions govern large-scale flow volumes for major events, local hydraulic responses, timing, attenuation, and water surface elevations remain dependent on model routing, storage representation, and terrain controls. Calibration performance metrics were therefore interpreted considering both boundary forcing and model dynamics.

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 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. Different locations may respond differently to changes in model parameters due to variations in local topography, land use, hydrology, and soil types. This sensitivity can result in calibration success at one location but validation failures at another. Hurricane impacts can be highly localized. For instance, storm surge and flooding can differ dramatically over short distances due to factors like landscape features, existing infrastructure, and natural barriers. A model calibrated at one location may not capture these localized effects elsewhere. The interactions between water flow, sediment transport, and other hydrodynamic processes can vary significantly across different locations. If the model does not adequately account for these processes in certain areas, it may lead to mismatches during validation.

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 elevations show the model being higher, on the order of a quarter of a foot.

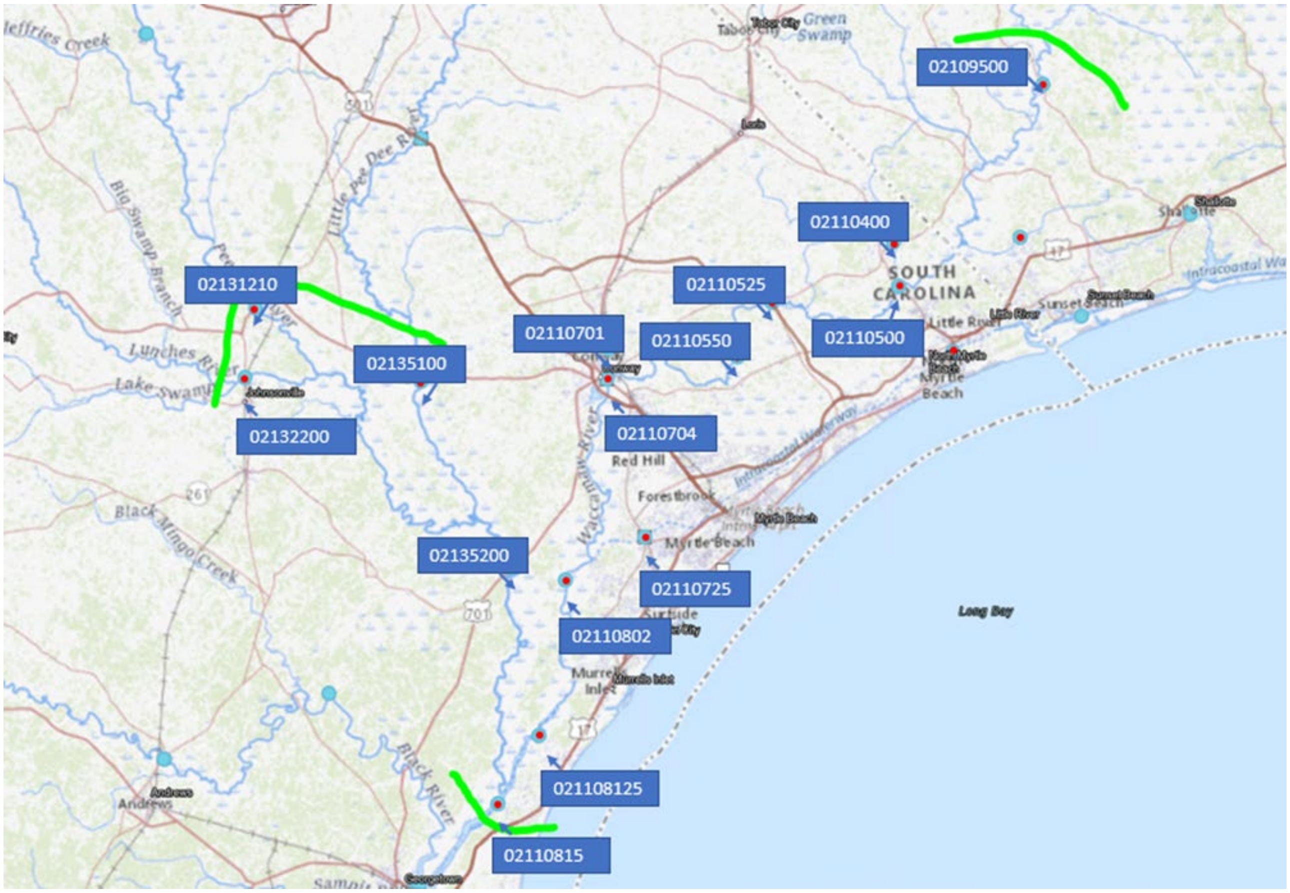


Figure E-12. Locations of streamflow gages within the Waccamaw River basin used to inform hydrologic analyses and support hydraulic model calibration. Gage positions relative to the HEC-RAS model domain and boundary condition inflow locations are considered in the calibration assessment.

Table E-3. 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

Station Number	Station Name
2110500	Waccamaw near Longs
2110400	Buck Creek near Longs
2109500	Waccamaw at Freeland
NOAA only	Caw Caw Swamp

E.3.1 Hurricane Florence Calibration

Model calibration was performed to validate the water surface elevation and flow output results for Hurricane Florence. **Figure E-13** shows total rainfall accumulation across the project area and **Figure E-14** shows 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.

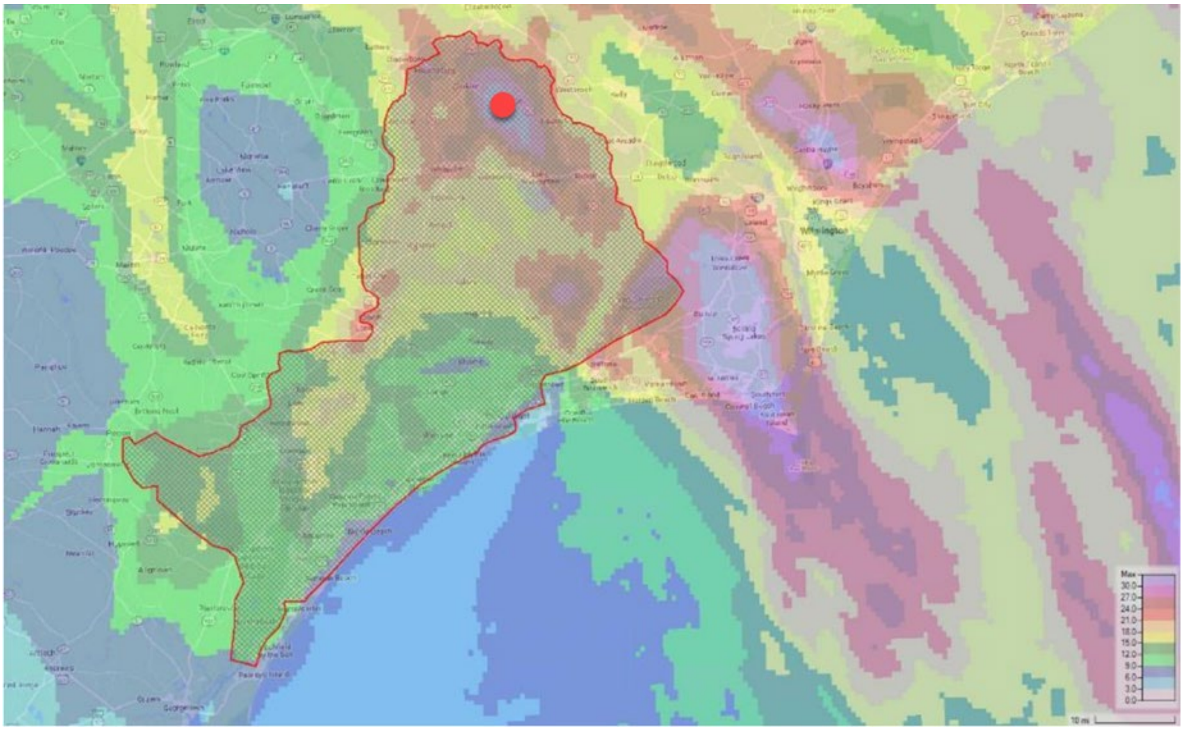


Figure E-13. Total Rainfall Accumulation Map for Hurricane Florence (9/13-9/20/2018).

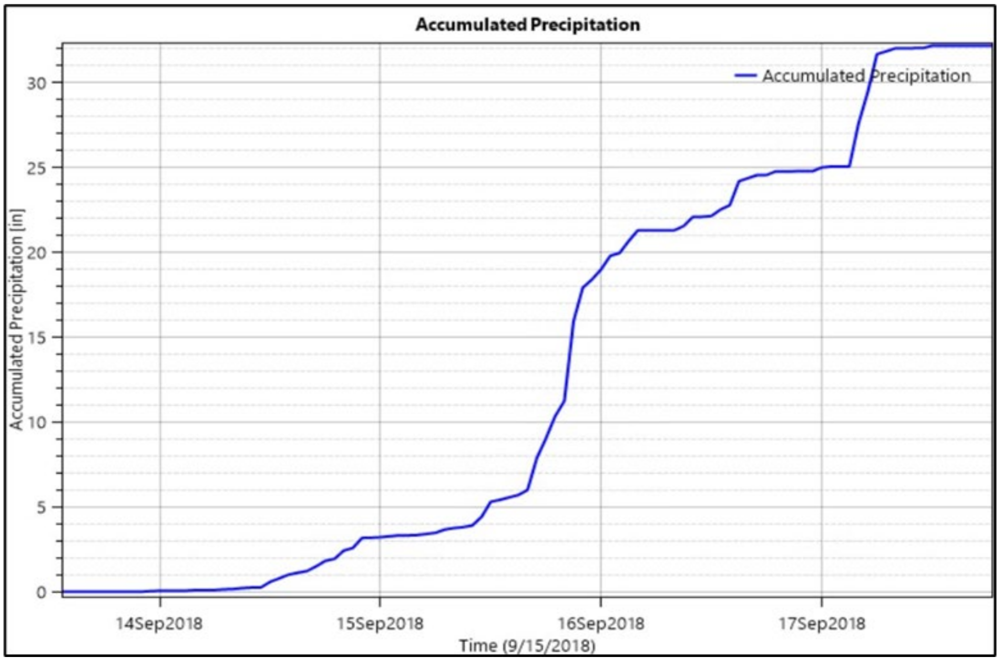


Figure E-14. Approximate Maximum Accumulated Precipitation Point for Hurricane Florence. The location of the hyetograph is indicated in previous figure with a red dot.

Results for the Hurricane Florence calibration event at select USGS gages are shown in **Figure E-15** through **Figure E-24**. Hurricane impacts can be highly localized. For instance, storm surge and flooding can differ dramatically over short distances due to factors like landscape features, existing infrastructure, and natural barriers. A model calibrated at one location may not capture these localized effects elsewhere. Additional evaluation was performed to characterize the influence of boundary conditions on simulated flows at calibration gage locations during Hurricane Florence. Streamflow hydrographs extracted at gage cross sections were analyzed to distinguish contributions from prescribed boundary inflows and internally generated runoff. Results indicate that approximately 80% of the simulated flow volume at the calibration locations was attributable to boundary condition inflows, reflecting the dominant influence of large-scale riverine and tributary inputs associated with this event.

While boundary conditions governed the overall flow magnitude, local hydraulic responses, including timing, attenuation, floodplain storage interactions, and resulting water surface elevations, remained dependent on model routing, storage representation, and terrain characteristics. Calibration performance metrics were therefore interpreted in the context of both boundary forcing and the hydraulic behavior of the model.

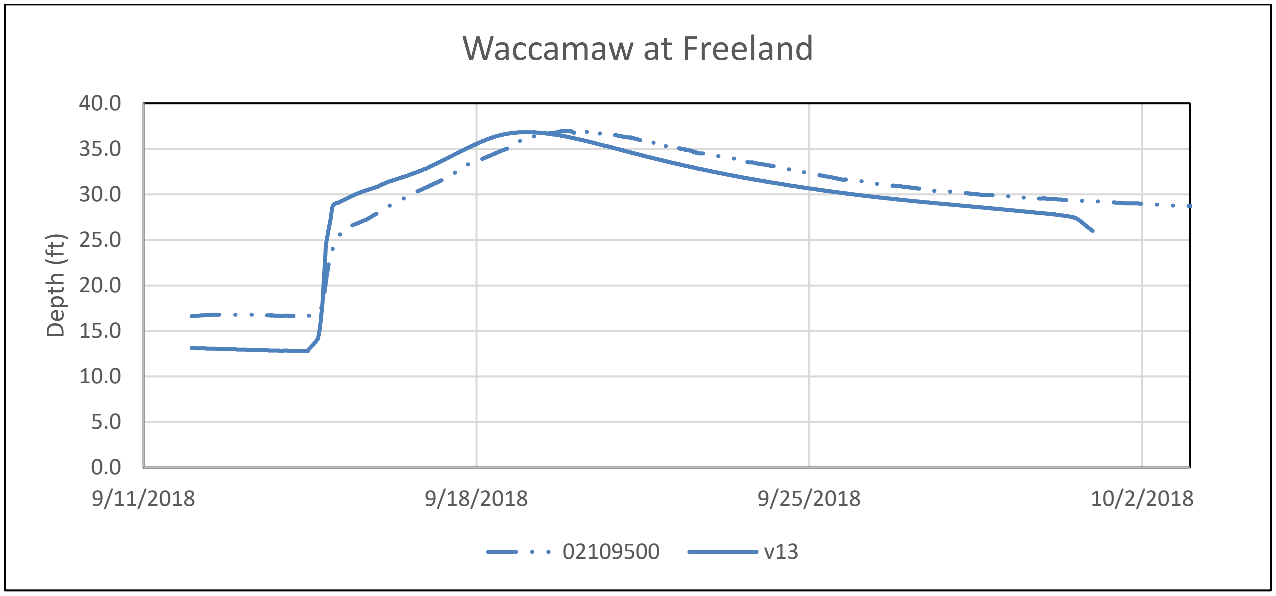


Figure E-15. Calibration Hurricane Florence 02109500.

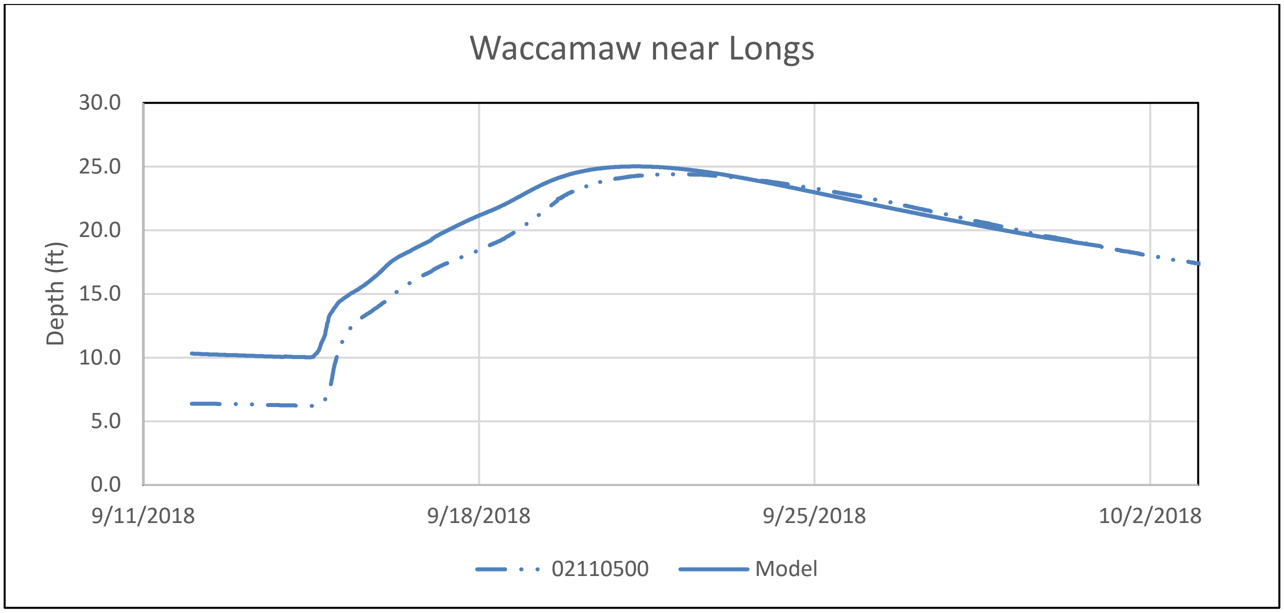


Figure E-16. Calibration Hurricane Florence 021010500.

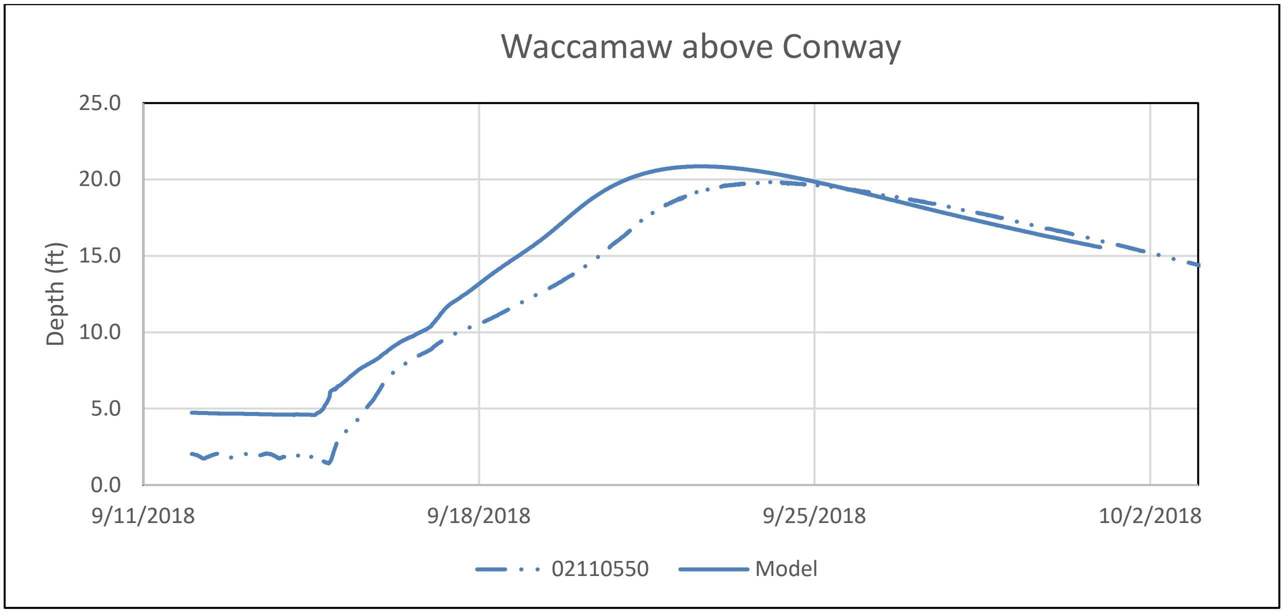


Figure E-17. Calibration Hurricane Florence 02110550.

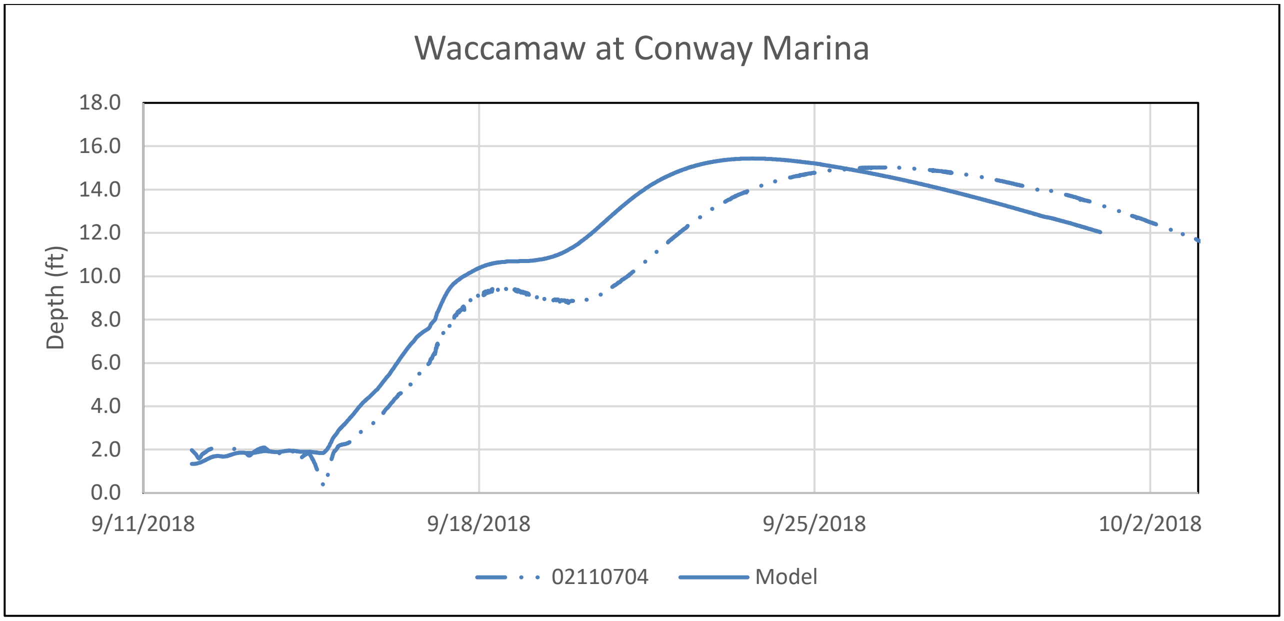


Figure E-18. Calibration Hurricane Florence USGS gage 02110704.

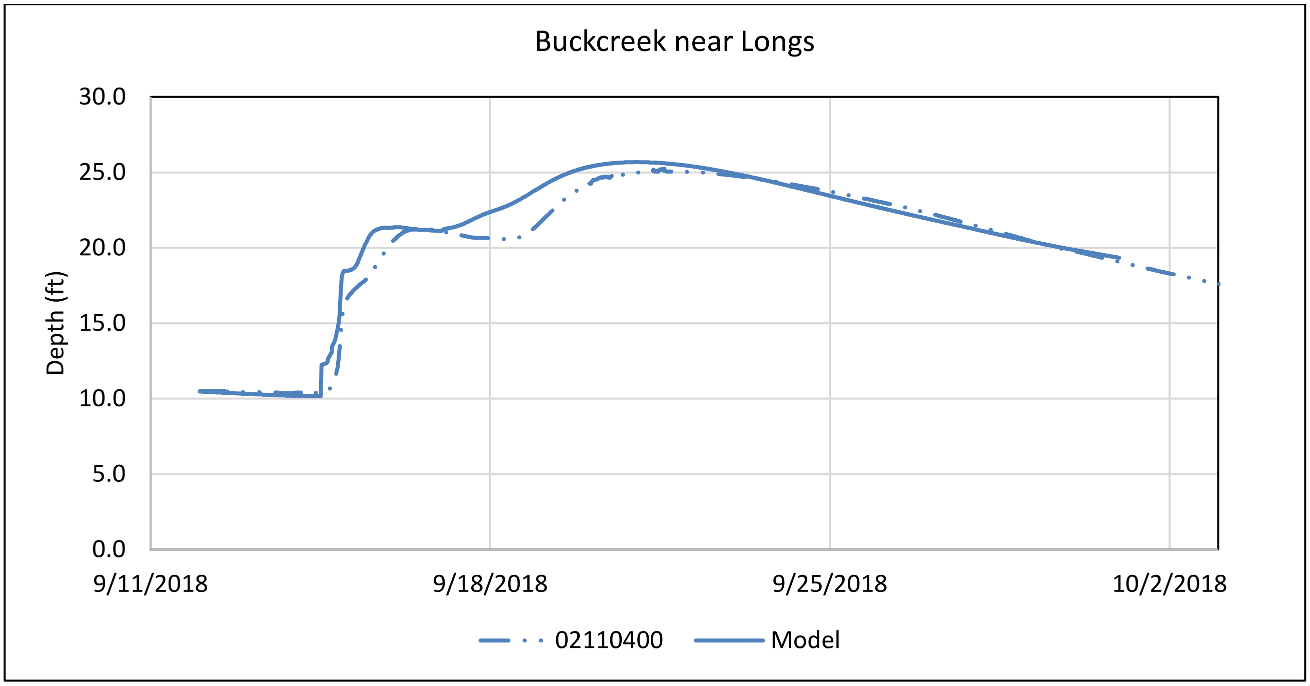


Figure E-19. Calibration Hurricane Florence USGS gage 02110400.

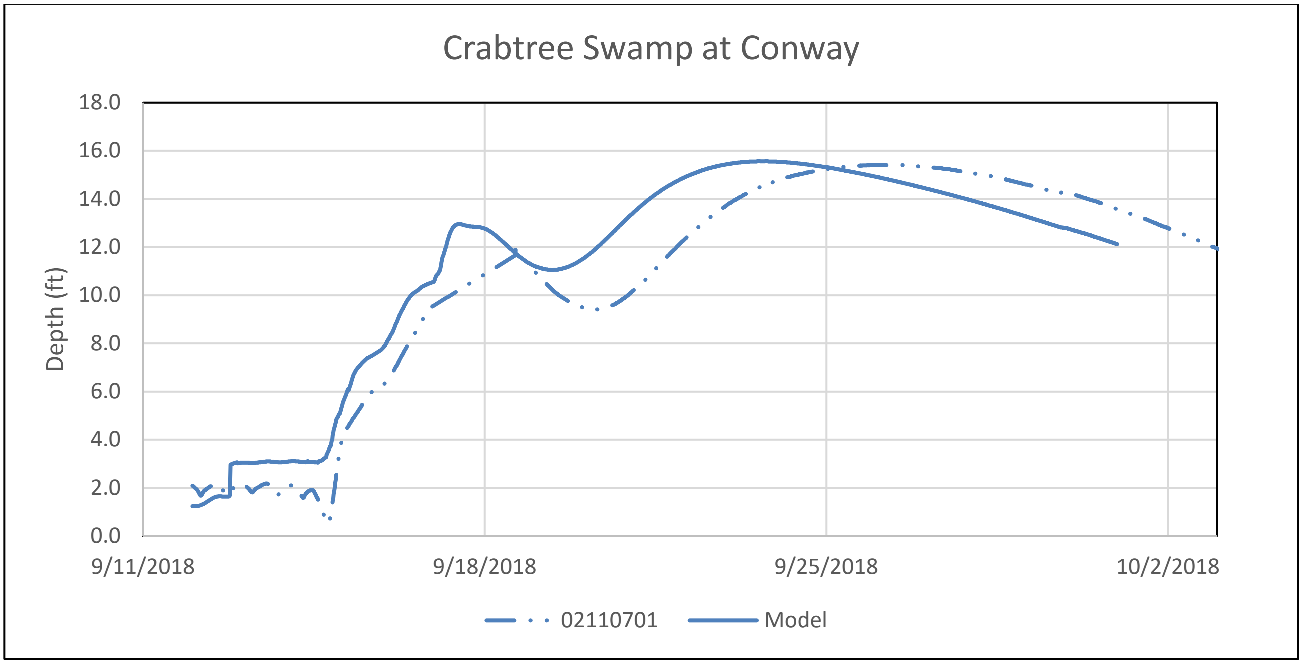


Figure E-20. Calibration Hurricane Florence USGS gage 02110701.

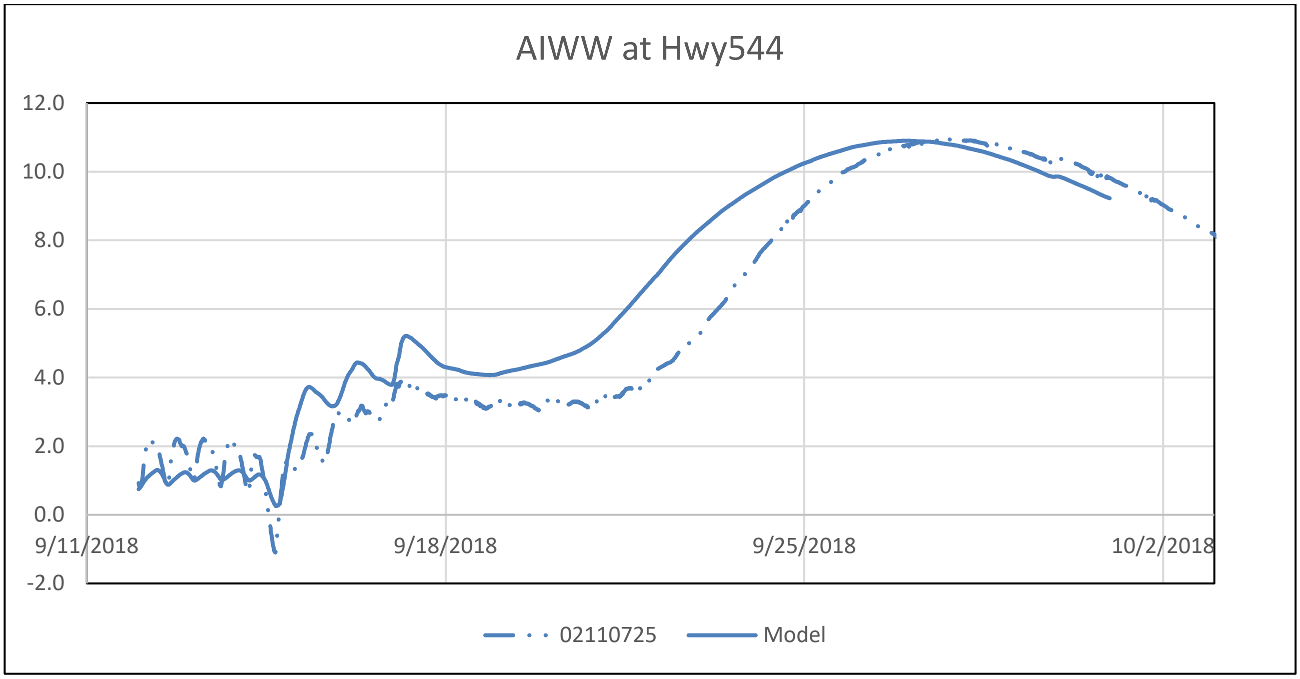


Figure E-21. Calibration Hurricane Florence USGS gage 02110725.

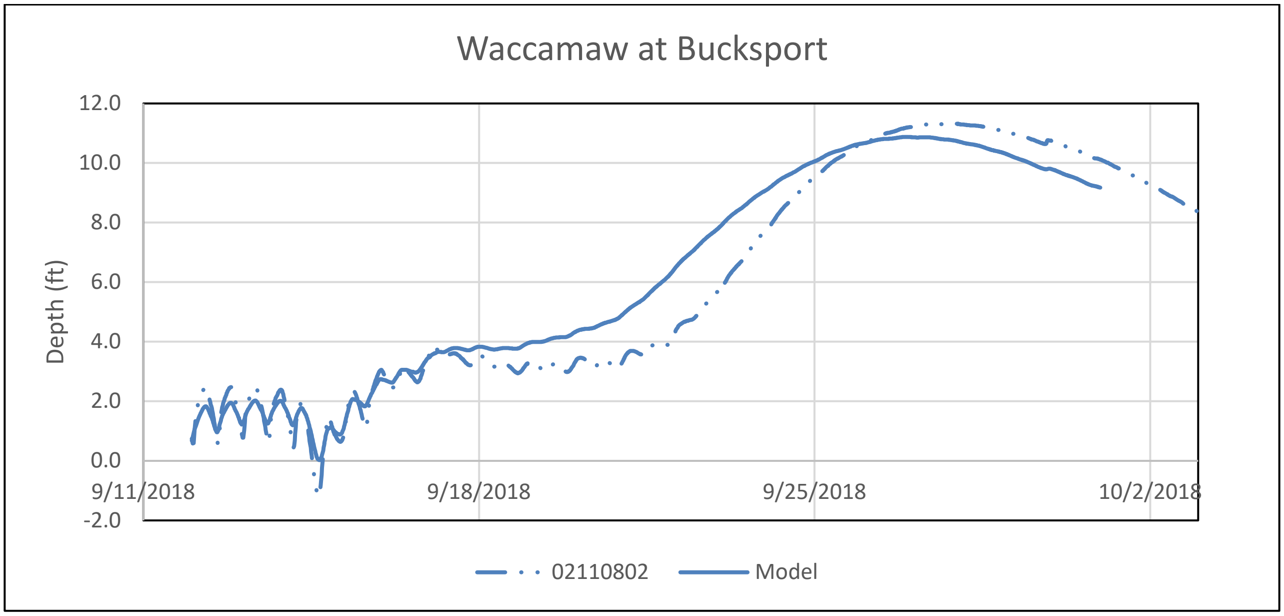


Figure E-22. Calibration Hurricane Florence USGS gage 02110802.

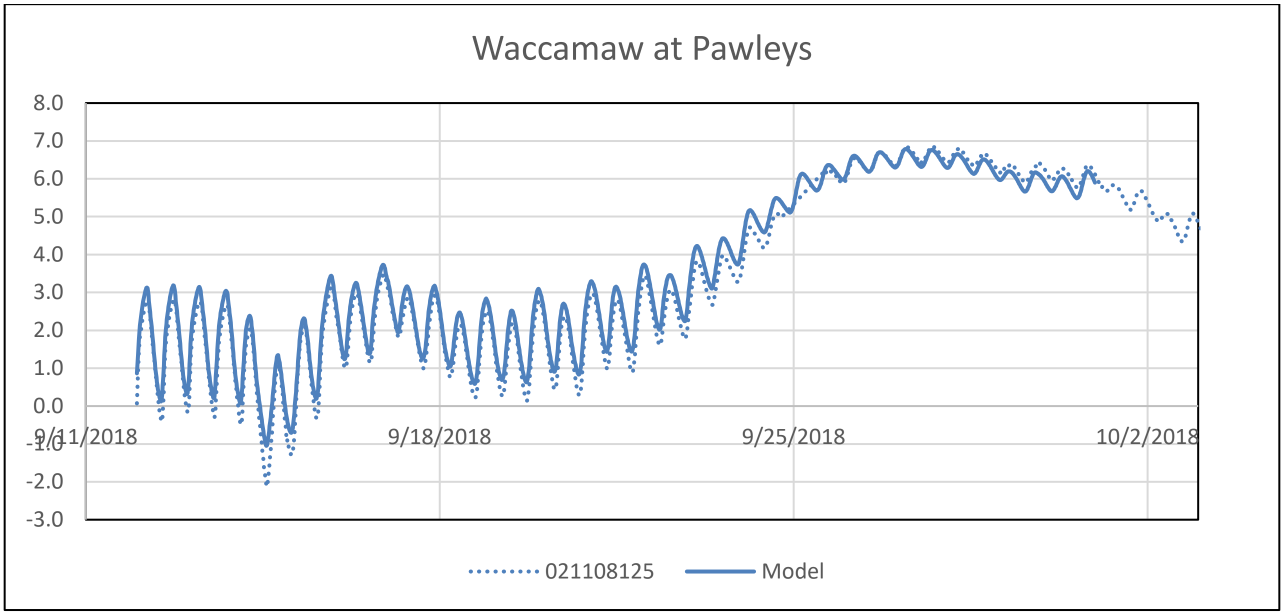


Figure E-23. Calibration Hurricane Florence USGS gage 021108125

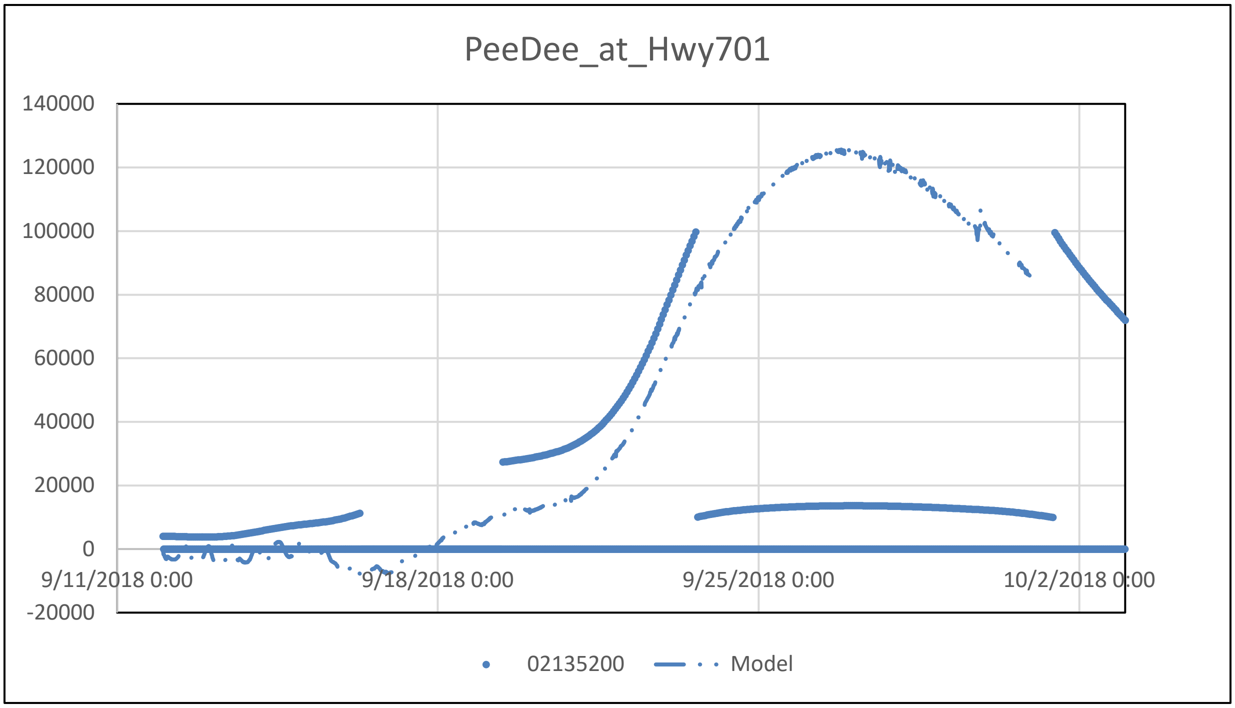


Figure E-24. Flow Calibration Hurricane Florence USGS gage 02135200

Table E-4 shows results from the calibrated Hurricane Florence model run. Additional discussion of sensitivity analysis and parameter adjustment can be found in Section E.3.4.

Table E-4. Summarized Results of Hurricane Florence Calibration stage and flow.

Gage Location	Gage ID	Observed Stage(ft)	Computed Stage (ft)	Std. Dev.	Variance (%)
Waccamaw at Freeland	2109500	37.01	36.835	0.12	0.47%
Waccamaw near Longs	2110500	24.41	25.012	0.43	2.47%
Buck Creek near Longs	2110400	25.25	25.675	0.3	1.68%
Waccamaw in Conway	2110550	19.82	20.858	0.73	5.24%
Crabtree Swamp at Conway	2110701	15.42	15.561	0.1	0.91%
Waccamaw at Conway Marina	2110704	15.02	15.433	0.29	2.75%
AIW_at_Hwy544	2110725	10.95	10.901	0.03	0.45%
Waccamaw at Bucksport	2110802	11.319	10.875	0.31	3.92%
Waccamaw at Pawleys	21108125	6.85	6.786	0.05	0.93%
PeeDee at Hwy701 (flow)	02135200 **	129000 cfs	125655 cfs	2364.82	2.59%

E.3.2 Hurricane Matthew Calibration

Model Calibration was performed to validate the water surface elevation and flow output results for Hurricane Matthew. **Figure E-25** shows total rainfall accumulation across the project area and **Figure E-26** 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.

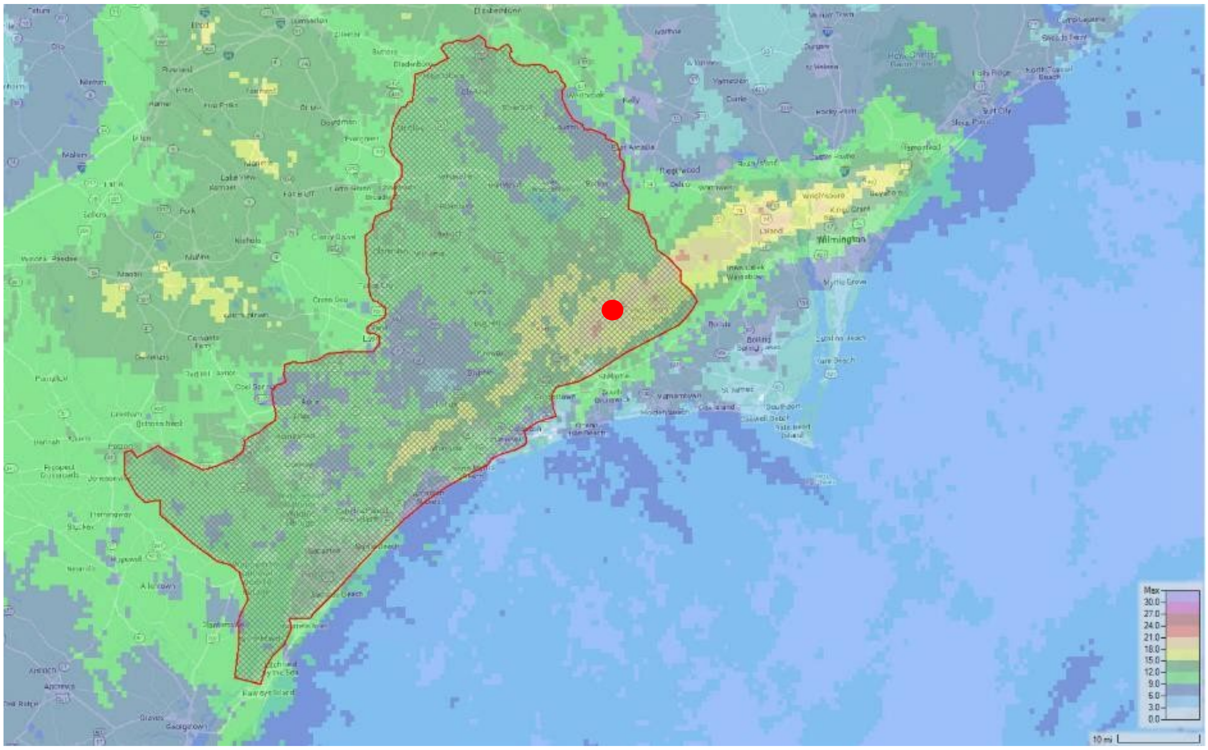


Figure E-25. Total Rainfall Accumulation Map for Hurricane Matthew (9/7/2016- 9/10/2016)

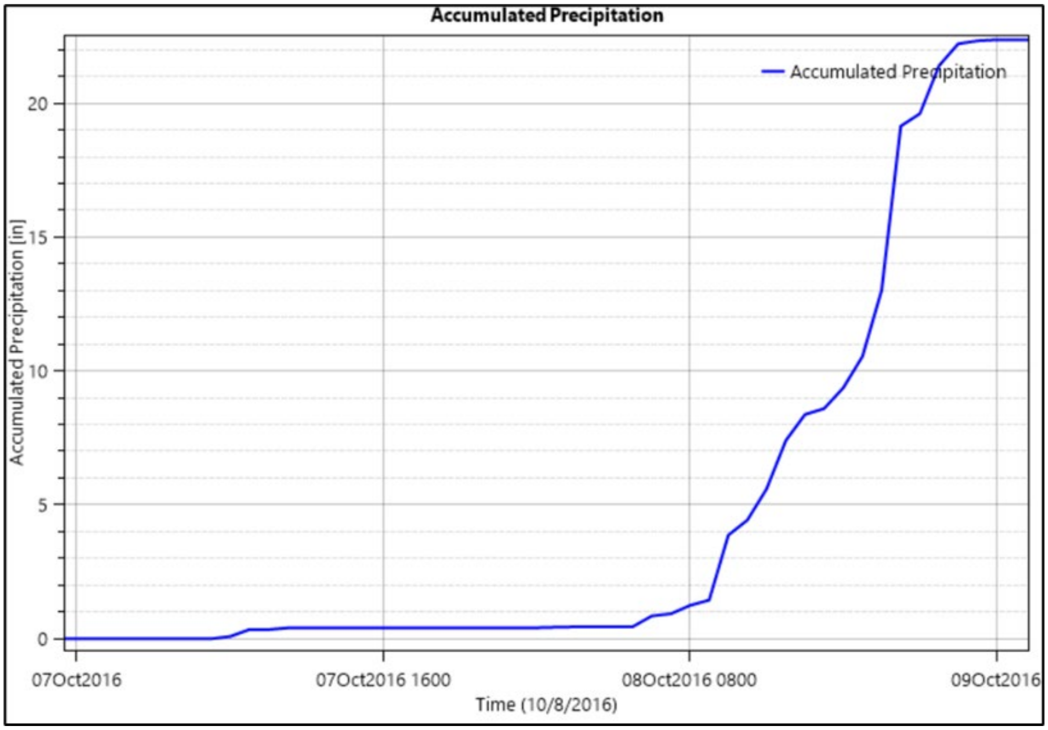


Figure E-26. Approximate Maximum Accumulated Precipitation Point for Hurricane Matthew

Results for the Hurricane Matthew calibration event at select USGS gages are shown in **Figure E-27** through **Figure E-35**. Different locations may respond differently to changes in model parameters due to variations in local topography, land use, hydrology, and soil types. This sensitivity can result in calibration success at one location but validation failures at another. Additional evaluation was performed to characterize the influence of boundary conditions on simulated flows at calibration gage locations during Hurricane Matthew. Analysis of extracted hydrographs indicates that approximately 75% of the simulated flow volume at the calibration locations was attributable to prescribed boundary inflows, consistent with the event's large-scale riverine response.

While boundary conditions governed the overall flow magnitude, local hydraulic behavior, including timing, attenuation, floodplain storage interactions, and water surface elevations, remained dependent on model routing, storage representation, and terrain characteristics. Calibration metrics were interpreted considering both boundary forcing and hydraulic model performance.

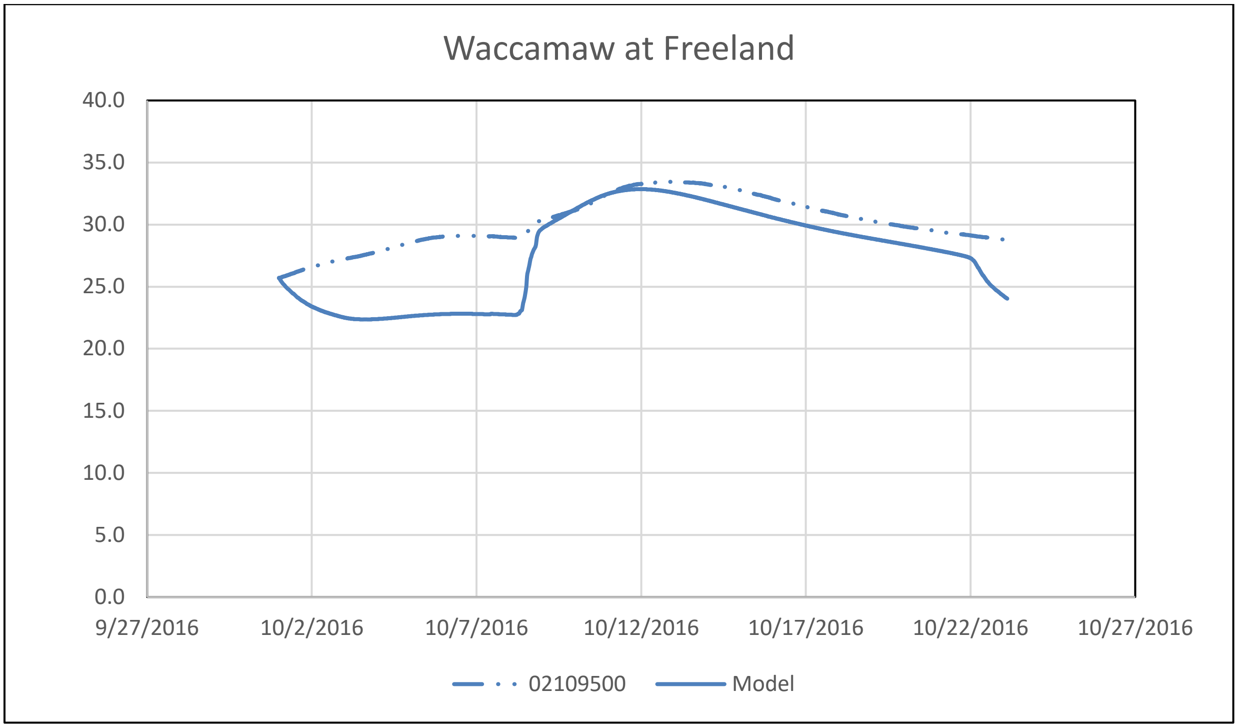


Figure E-27. Calibration Hurricane Matthew USGS 02109500

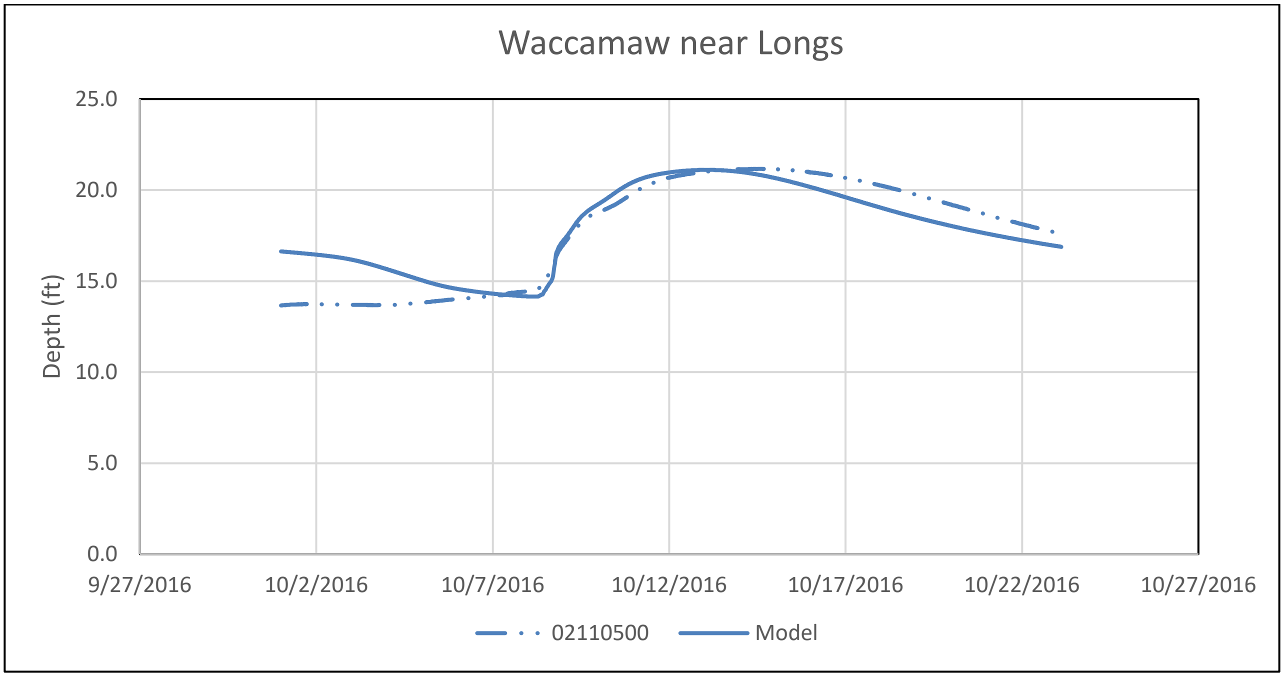


Figure E-28. Calibration for Hurricane Matthew USGS Gage 02110500

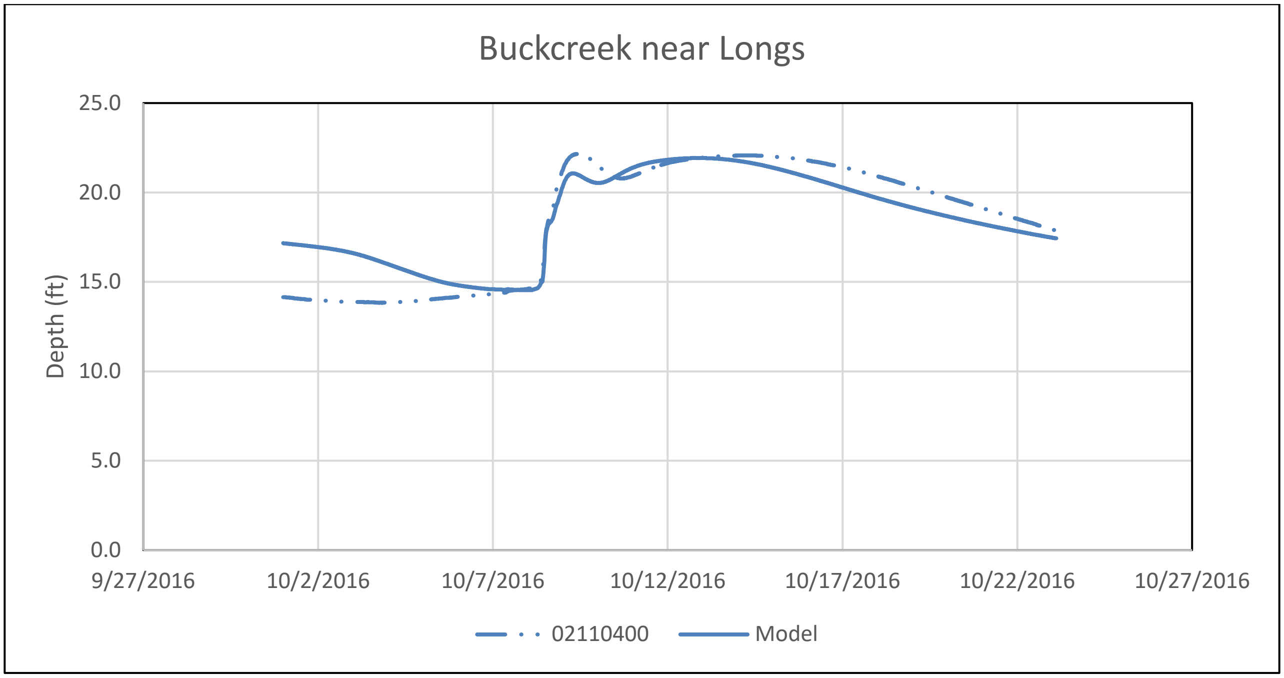


Figure E-29. Calibration for Hurricane Matthew USGS Gage 02110400

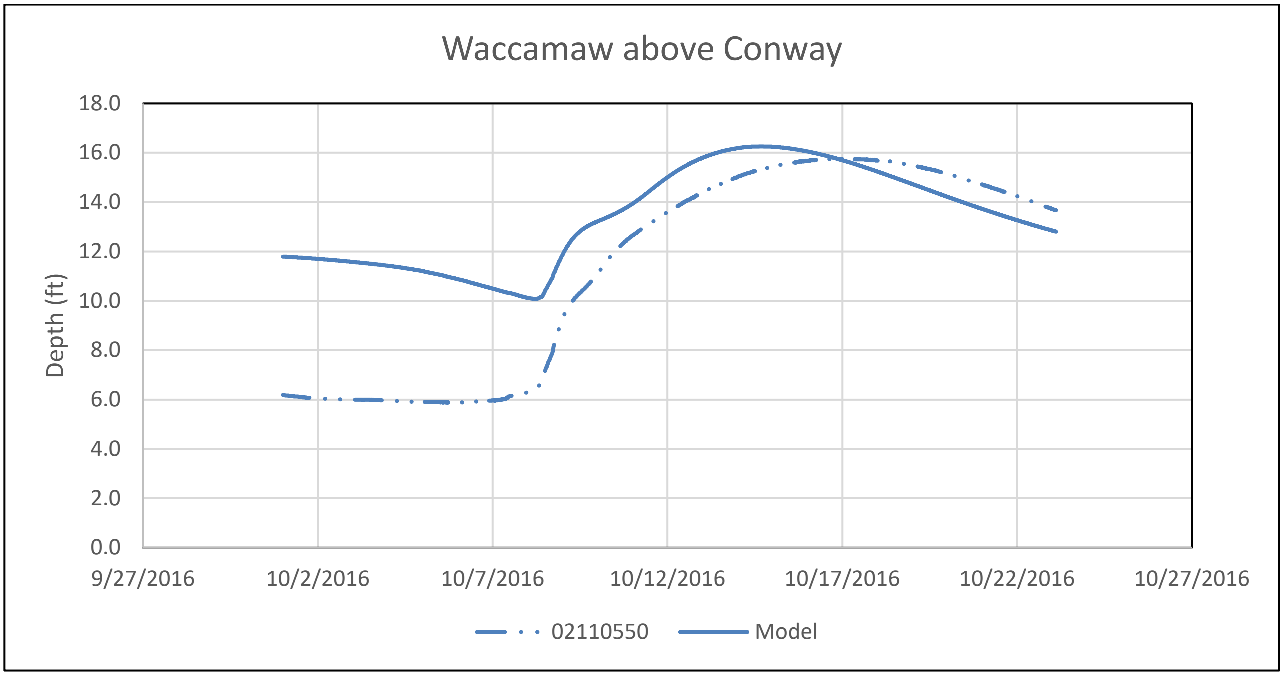


Figure E-30. Calibration for Hurricane Matthew USGS Gage 02110550

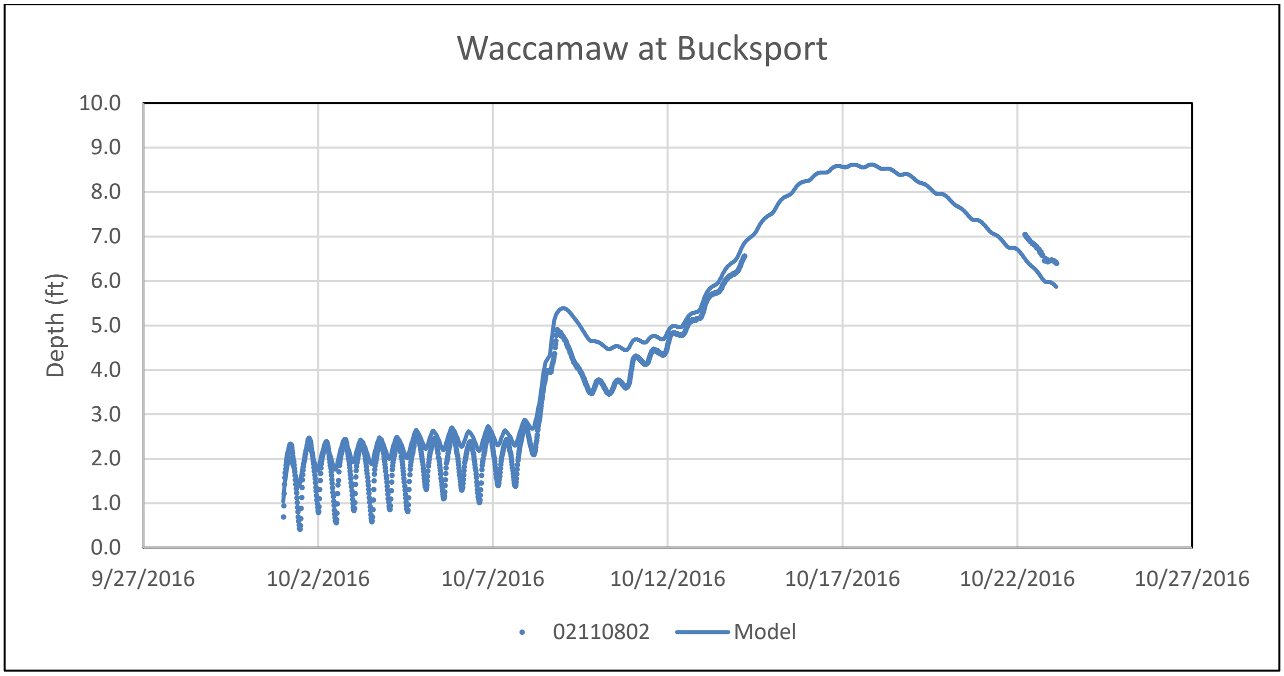


Figure E-31. Calibration Hurricane Matthew USGS Gage 02110802

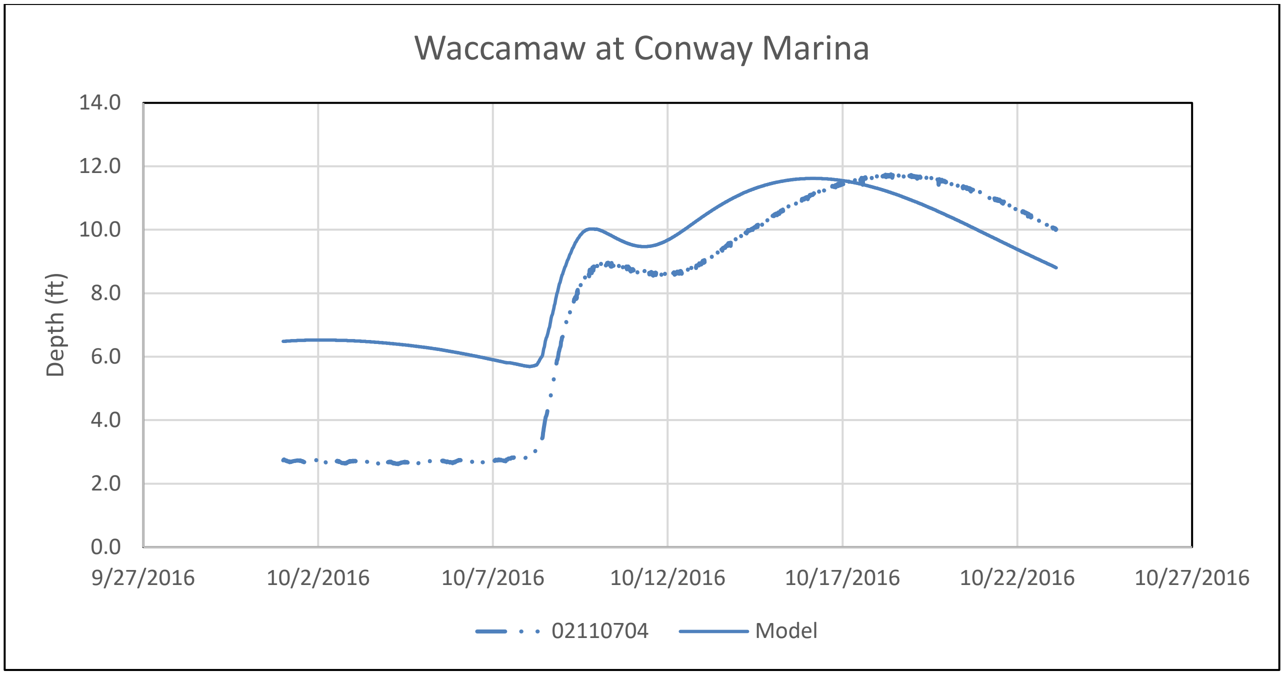


Figure E-32. Calibration Hurricane Matthew USGS Gage 02110704

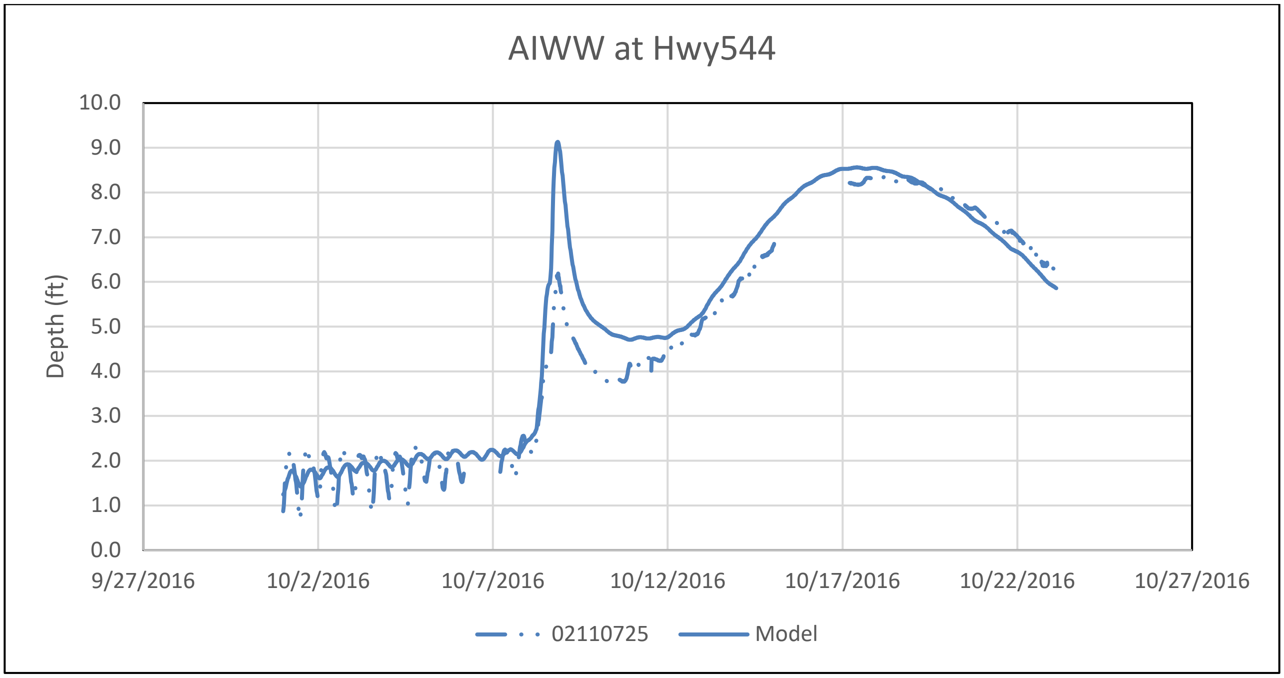


Figure E-33. Calibration for Hurricane Matthew USGS Gage 02110725

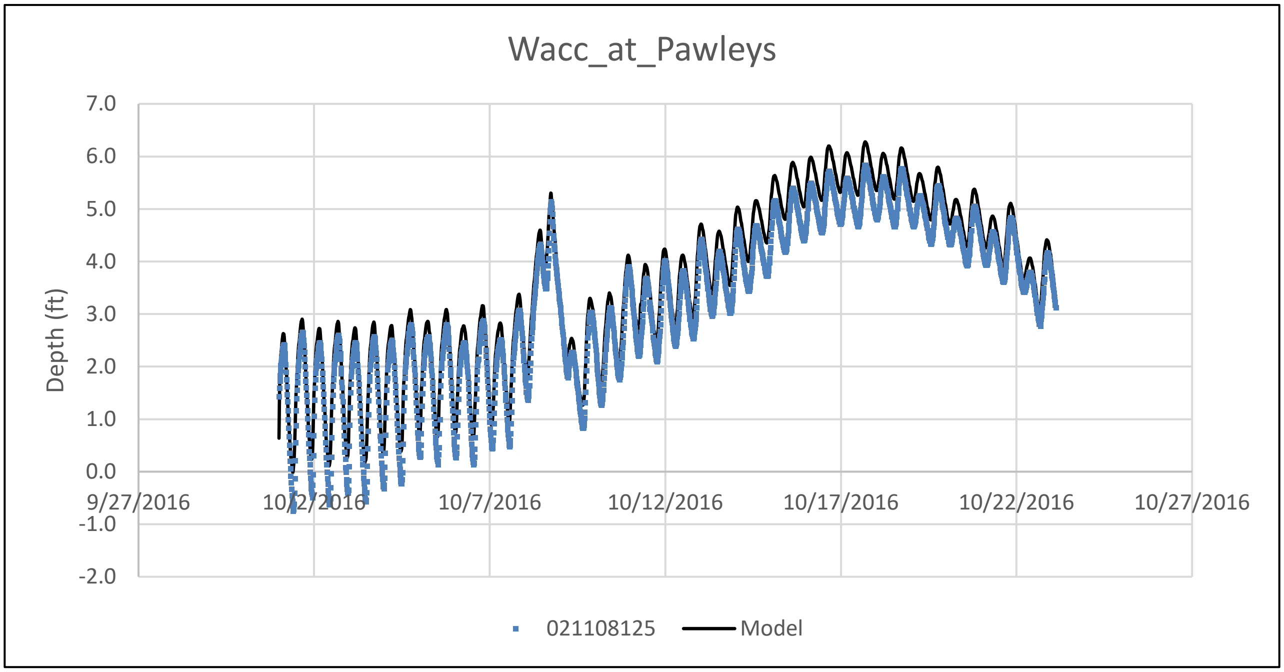


Figure E-34. Stage Calibration Hurricane Matthew USGS Gage 021108125

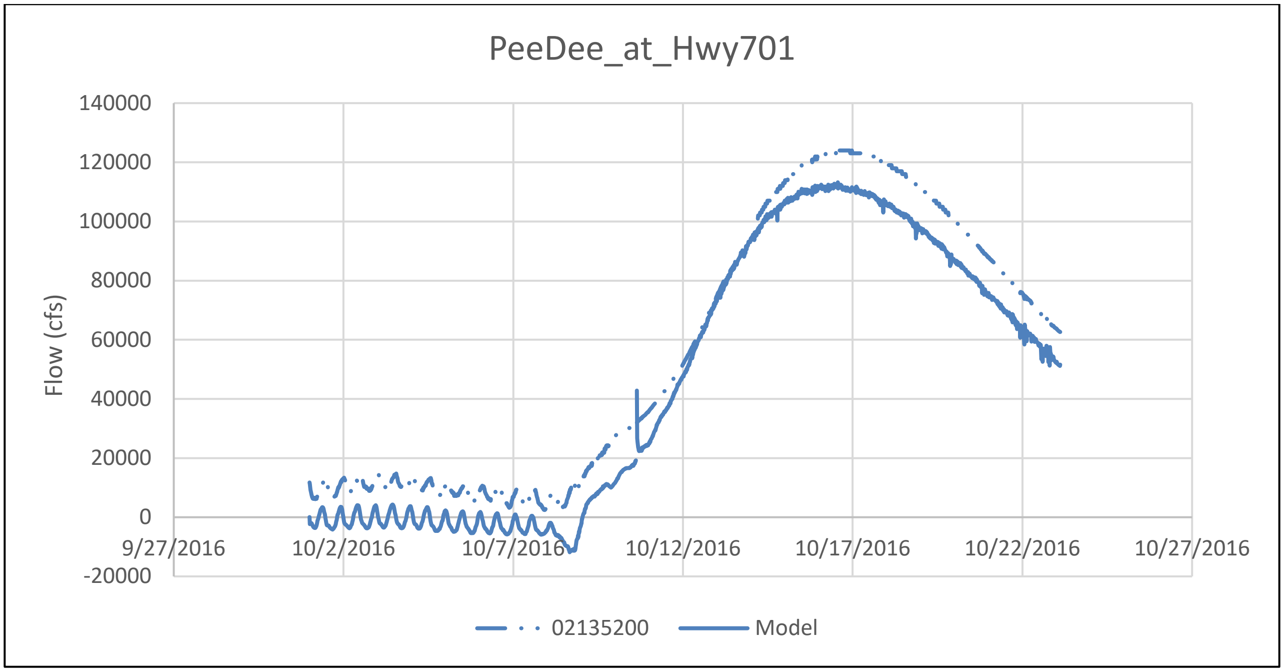


Figure E-35. Flow Calibration Hurricane Matthew USGS gage 02135200

Table E-5 shows results from the validated Hurricane Matthew model run. Additional discussion of sensitivity analysis and parameter adjustment can be found in Section E.3.4.

Table E-5. Summarized Results of Hurricane Matthew Validation

Gage Location	Gage ID	Observed Stage (ft)	Computed Stage (ft)	Std. Dev	Variance (%)
Waccamaw at Freeland	2109500	33.46	32.855	0.43	1.81%
Waccamaw near Longs	2110500	21.17	21.11	0.04	0.28%
Buck Creek near Longs	2110400	22.18	21.926	0.18	1.15%
Waccamaw above Conway	2110550	15.77	16.257	0.34	3.09%
Crabtree Swamp at Conway	2110701	12.07	12.098	0.02	0.23%
Waccamaw at Conway Marina	2110704	11.75	11.62	0.09	1.11%
AIWW at Hwy544	2110725	8.35	9.133	0.55	9.38%
Waccamaw at Bucksport	2110802	8.49	8.619	1.11	1.52%
Waccamaw at Pawleys	21108125	5.84	6.277	0.31	7.48%
PeeDee at Hwy701	02135200 **	121050 cfs	113237.6 cfs	7813.78	6.06%

E.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 E-36** shows total rainfall accumulation across the project area and **Figure E-37** 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.

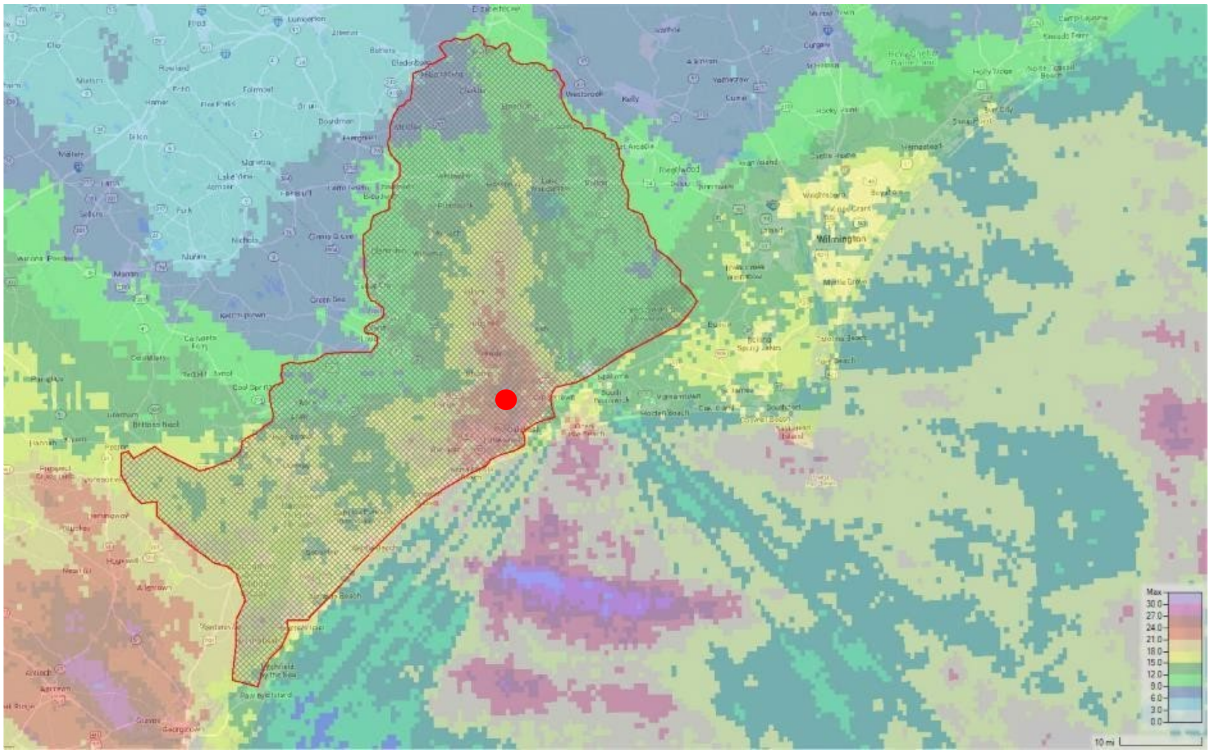


Figure E-36. Total Rainfall Accumulation for Hurricane Joaquin

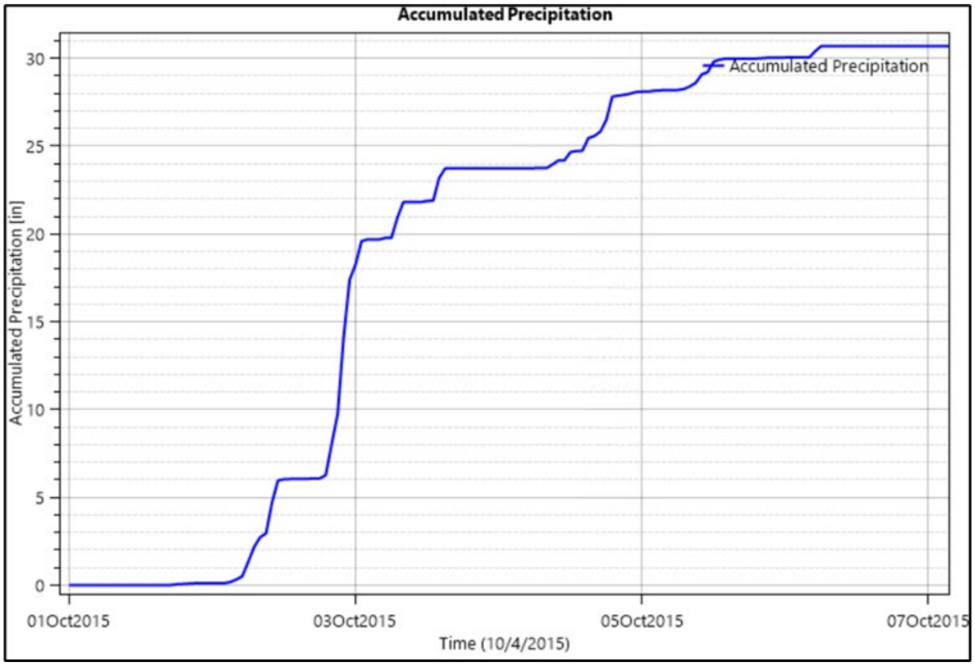


Figure E-37. Approximate Maximum Accumulated Precipitation for Hurricane Joaquin

Results for the Hurricane Joaquin calibration event at select USGS gages are shown in **Figure E-38** through **Figure E-47**. Different locations may respond differently to changes in model parameters due to variations in local topography, land use, hydrology, and soil types. This sensitivity can result in calibration success at one location but validation failures at another. Additional evaluation was performed to characterize the influence of boundary conditions on simulated flows at validation gage locations during Hurricane Joaquin. Analysis of extracted hydrographs indicates that approximately 80% of the simulated flow volume at the validation locations was attributable to prescribed boundary inflows, with the remaining contribution generated from internally modeled runoff and lateral inflows.

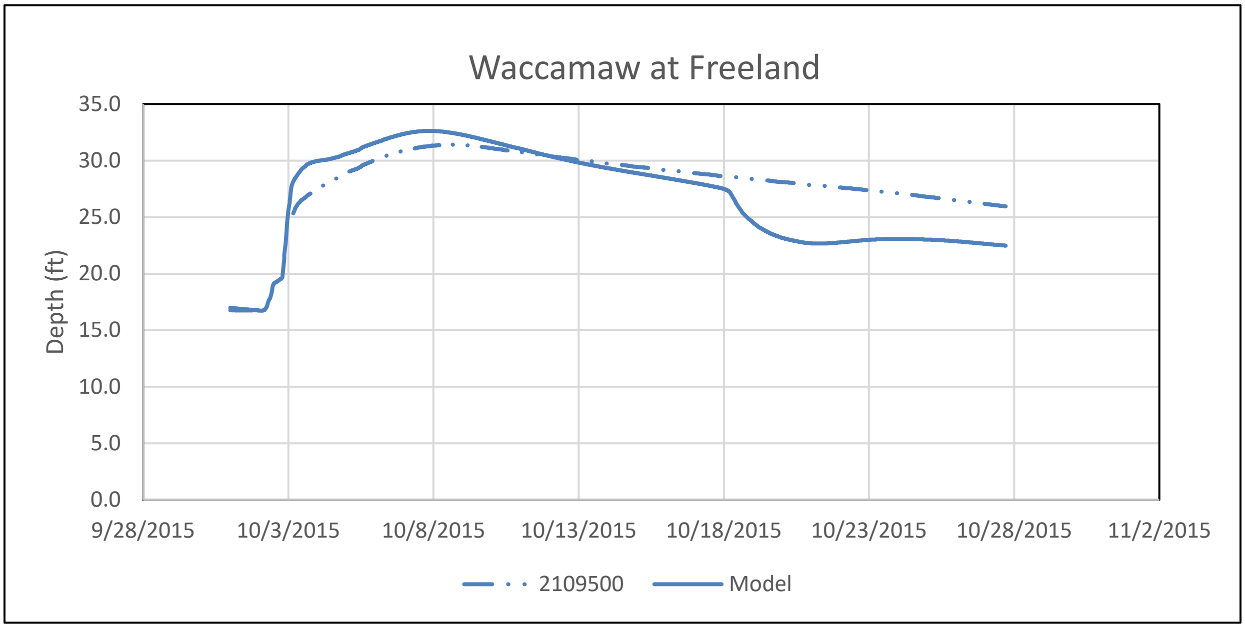


Figure E-38. Validation for Hurricane Joaquin USGS Gage 02109500

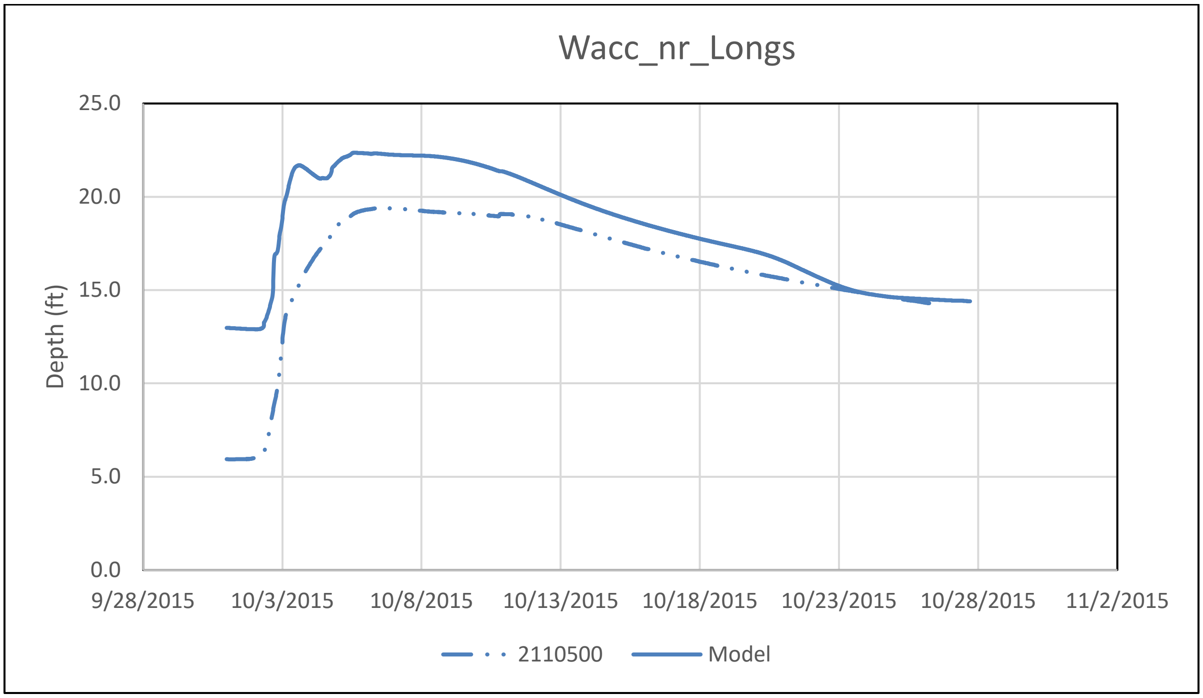


Figure E-39. Validation for Hurricane Joaquin USGS Gage 02110500

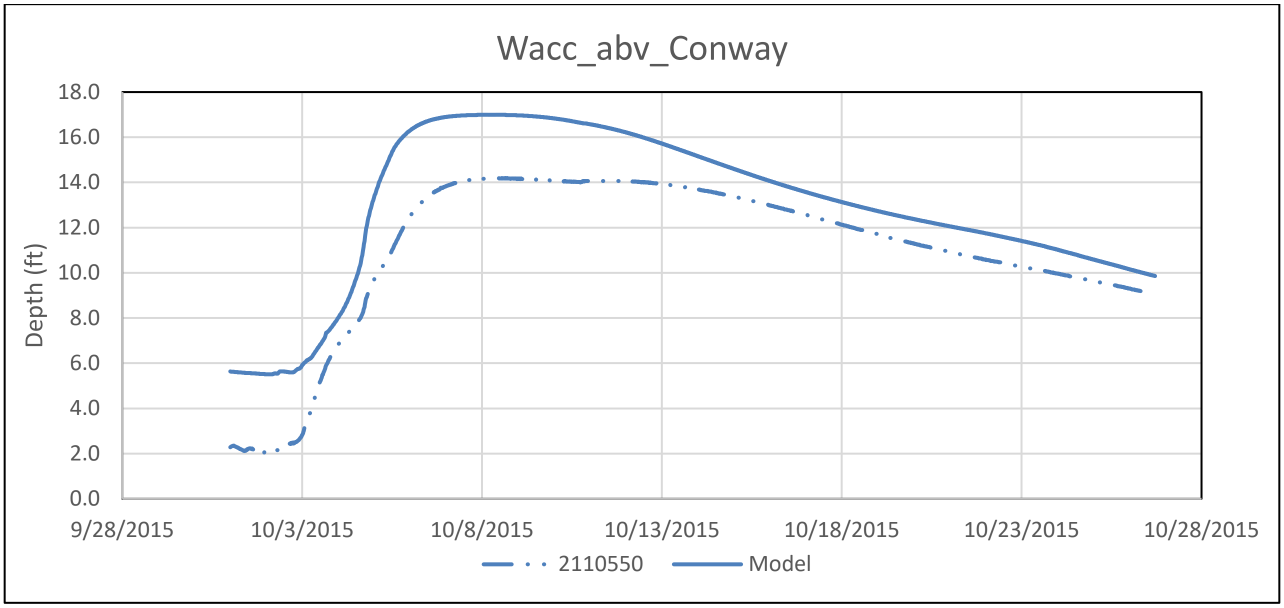


Figure E-40. Validation for Hurricane Joaquin USGS Gage 02110550

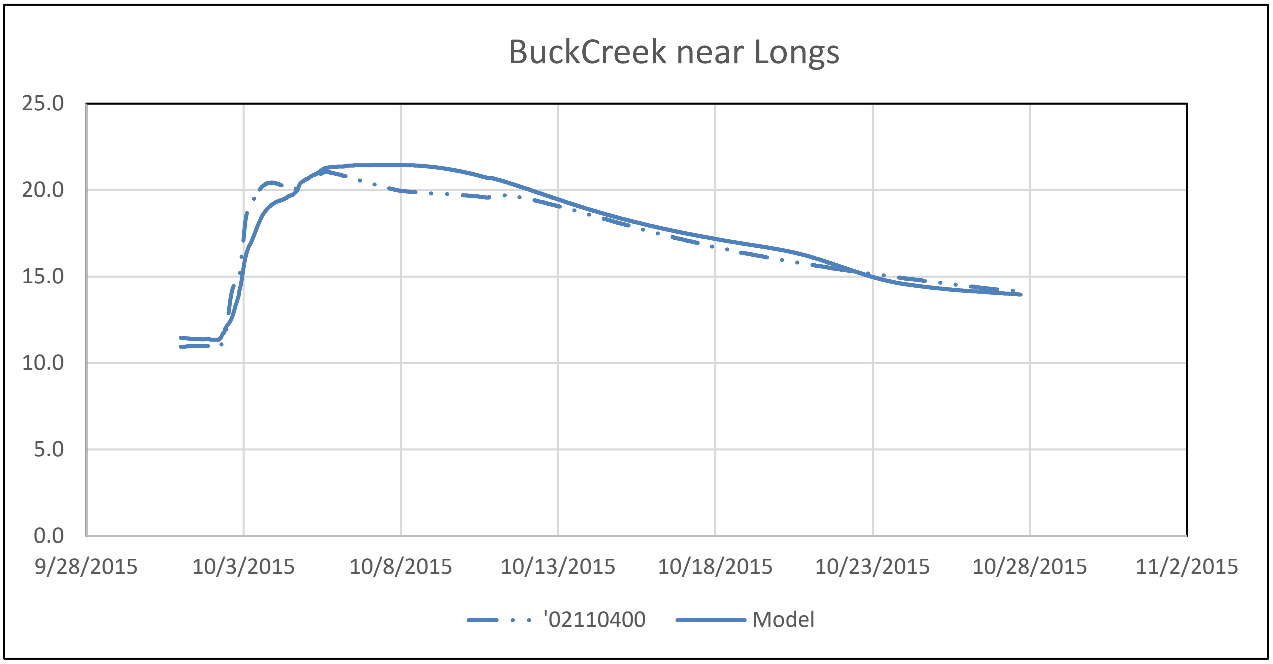


Figure E-41. Validation for Hurricane Joaquin USGS Gage 02110400

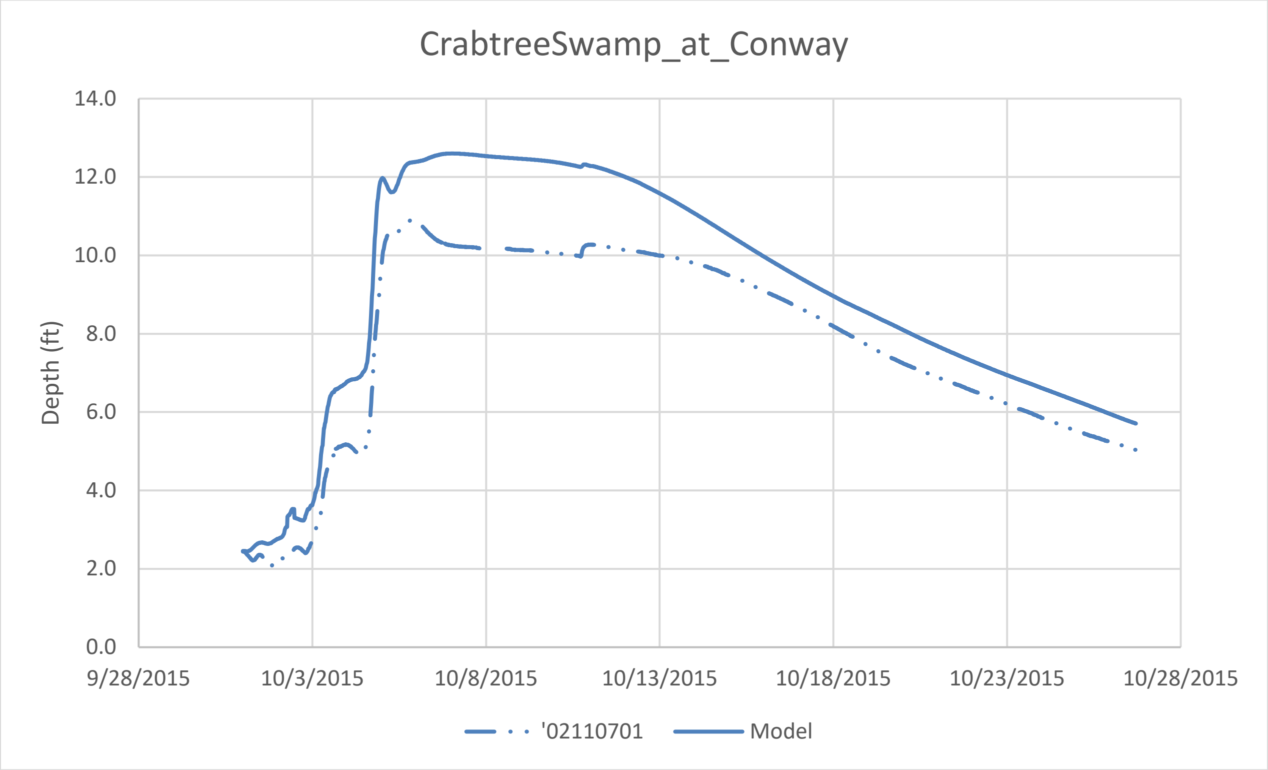


Figure E-42. Validation for Hurricane Joaquin USGS Gage 02110701

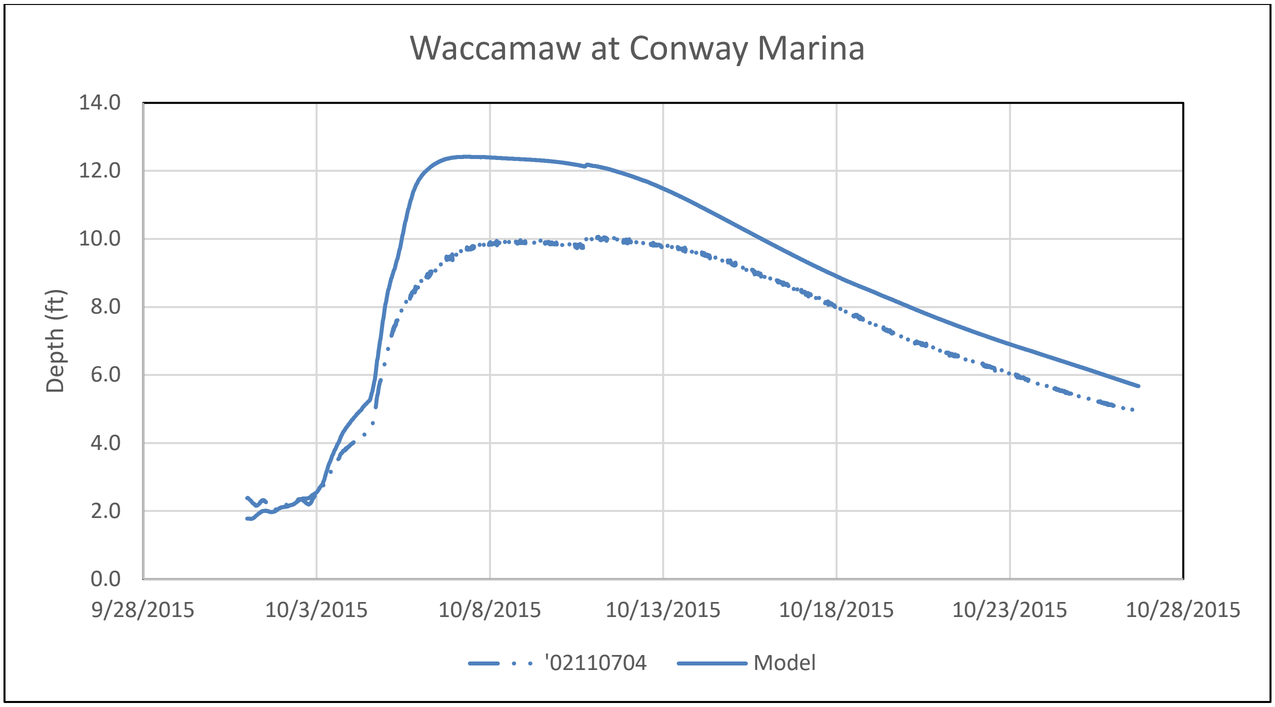


Figure E-43. Validation Hurricane Joaquin USGS gage 02110704

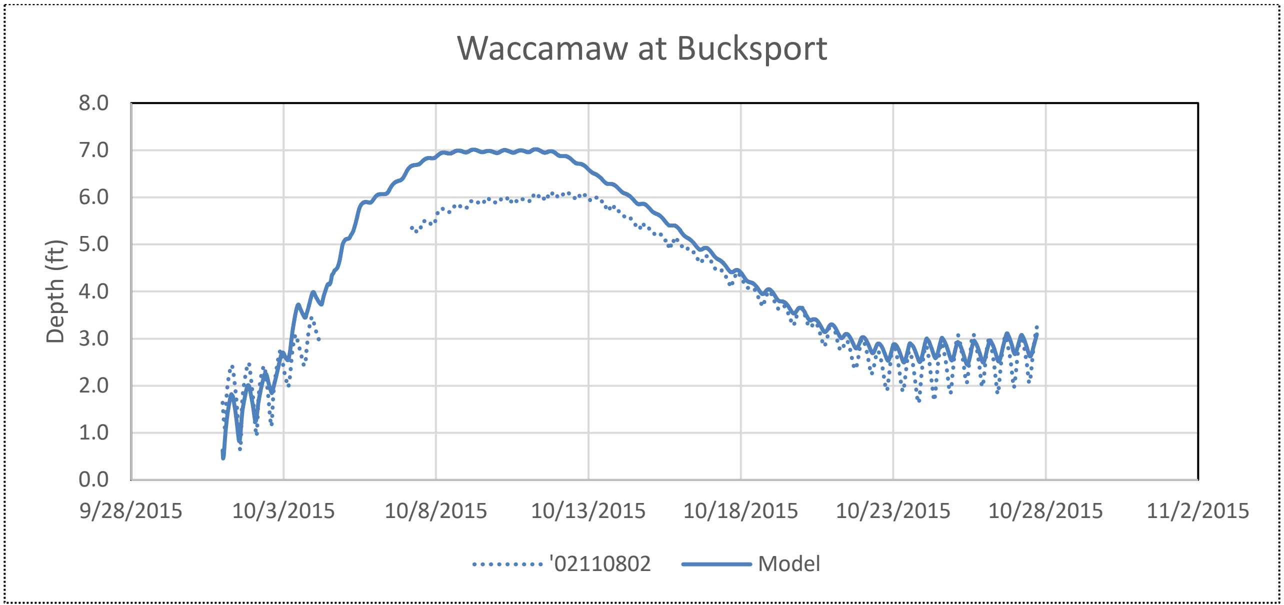


Figure E-44. Validation Hurricane Joaquin USGS gage 02110802

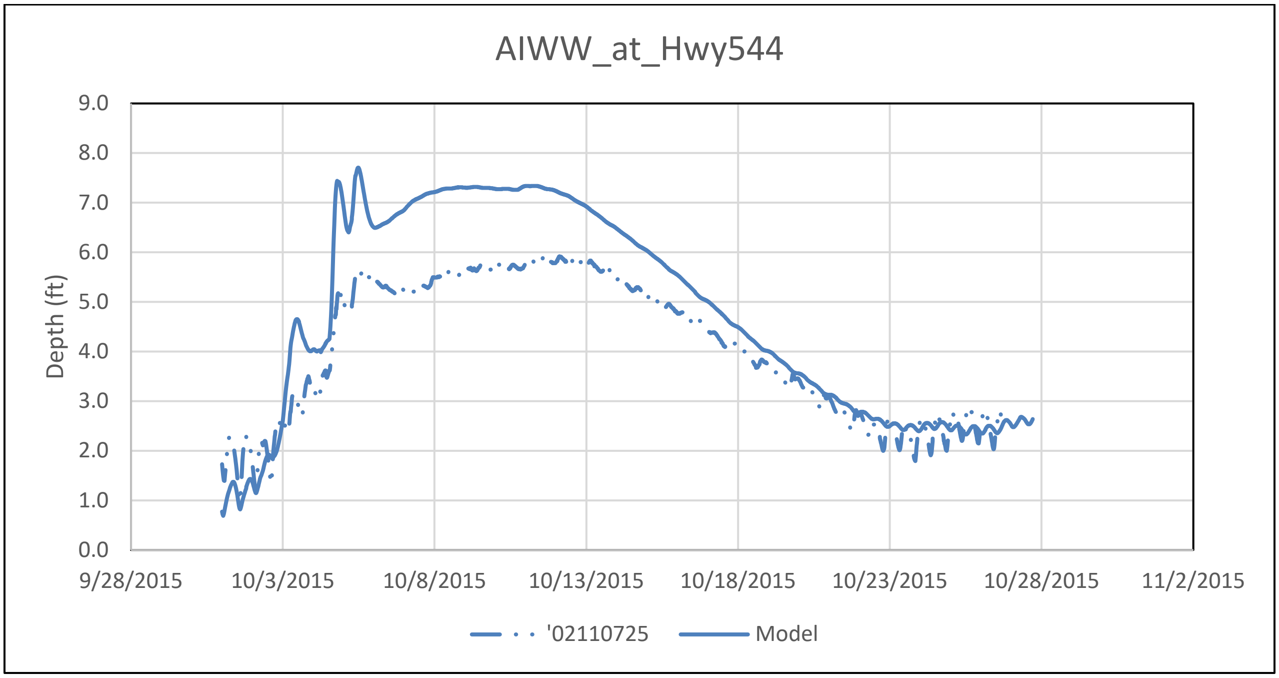


Figure E-45. Validation Hurricane Joaquin USGS gage 02110725

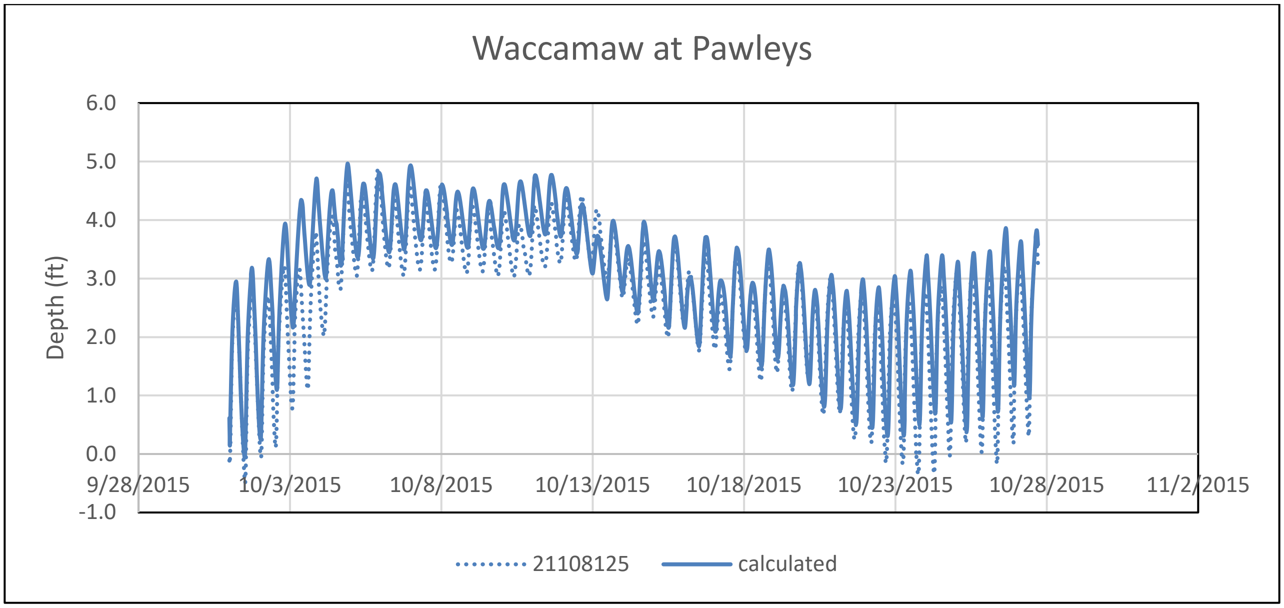


Figure E-46. Validation Hurricane Joaquin USGS gage 021108125

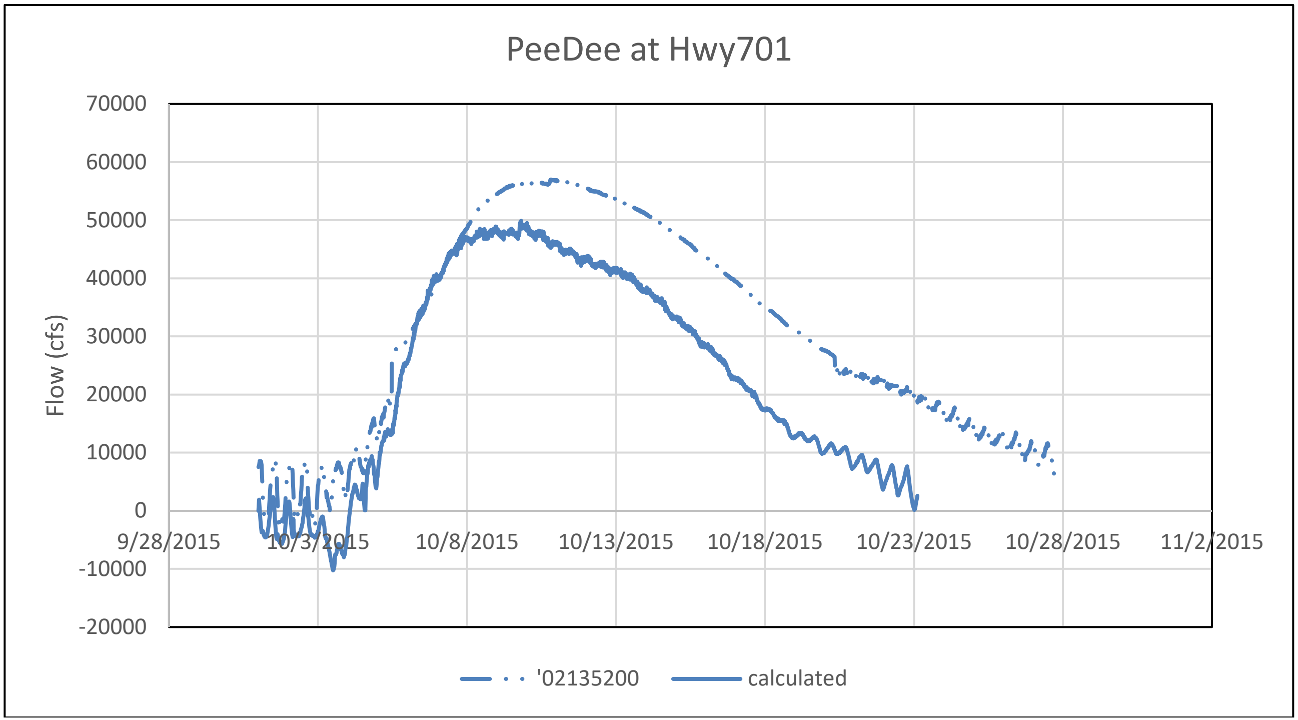


Figure E-47. Validation Hurricane Joaquin at USGS gage 02135200

Table E-6 shows results from the validation of the Hurricane Joaquin model run. Hurricane impacts can be highly localized. For instance, storm surge and flooding can differ dramatically over short distances due to factors like landscape features, existing infrastructure, and natural barriers. A model calibrated at one location may not capture these localized effects elsewhere. Additional discussion of sensitivity analysis and parameter adjustment can be found in Section E.3.4.

Table E-6. Summarized Results of Hurricane Joaquin Validation

Gage Location	Gage ID	Observed Stage (ft)	Computed Stage (ft)	Std. Dev	Variance (%)
Waccamaw at Freeland	2109500	31.41	32.624	0.61	3.68%
Waccamaw near Longs	2110500	19.39	22.366	1.48	2.21%
Buck Creek near Longs	2110400	21.05	21.45	0.28	0.04%
Waccamaw above Conway	2110550	14.19	16.994	1.40	1.96%
Crabtree Swamp at Conway	2110701	10.28	12.6	1.16	1.34%
Waccamaw at Conway Marina	2110704	10.09	12.415	1.162	1.35%
AIWW at Hwy544	2110725	6.02	7.707	0.84	0.711%
Waccamaw at Bucksport	2110802	6.12	8.02	0.95	0.91%
Waccamaw at Pawleys	21108125	4.89	4.966	0.038	0.014%
PeeDee at Hwy701	02135200 **	56100	49890.48	3105	2.63%

E.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 the model to the Initial Flow conditions, Bathymetry and Terrain, Roughness Values, and Hydraulic structures, address the sensitivity of the initial model calibration efforts. 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.

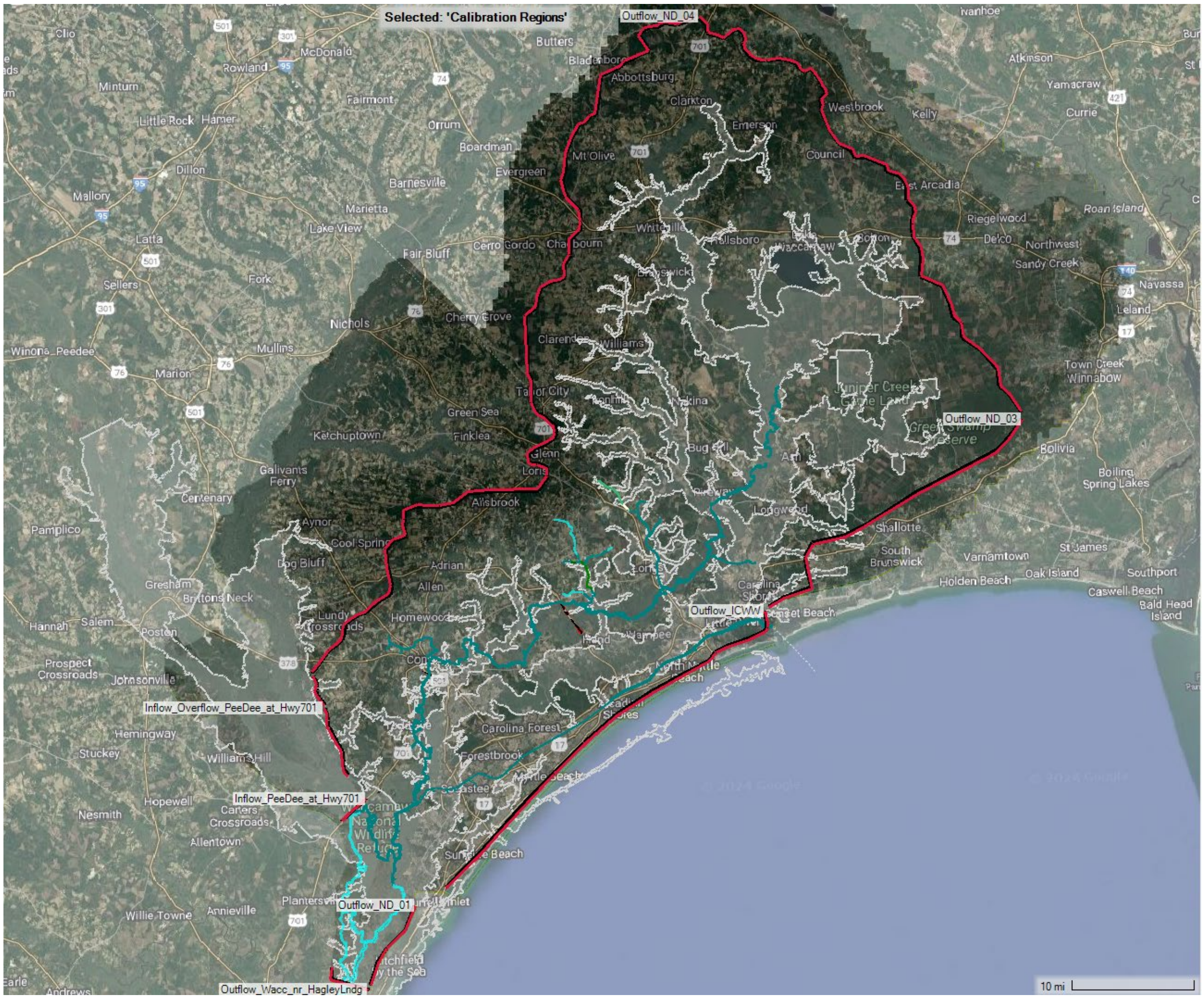


Figure E-48. Model Calibration overview with identified refinement regions, base (red), floodplain (white), and channel (teal)

E.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 condition input file to introduce a base flow and “wet” the model before performing calibration runs. The model calibration refinement regions are identified in **Figure E-48.** Calibration was done by simulating a 50% AEP, 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 gage conditions at the start of the calibration simulation. **Figure E-49** shows the USGS Stream gage at Freeland, NC (01209500) and the model results for without and with initial conditions startup file.

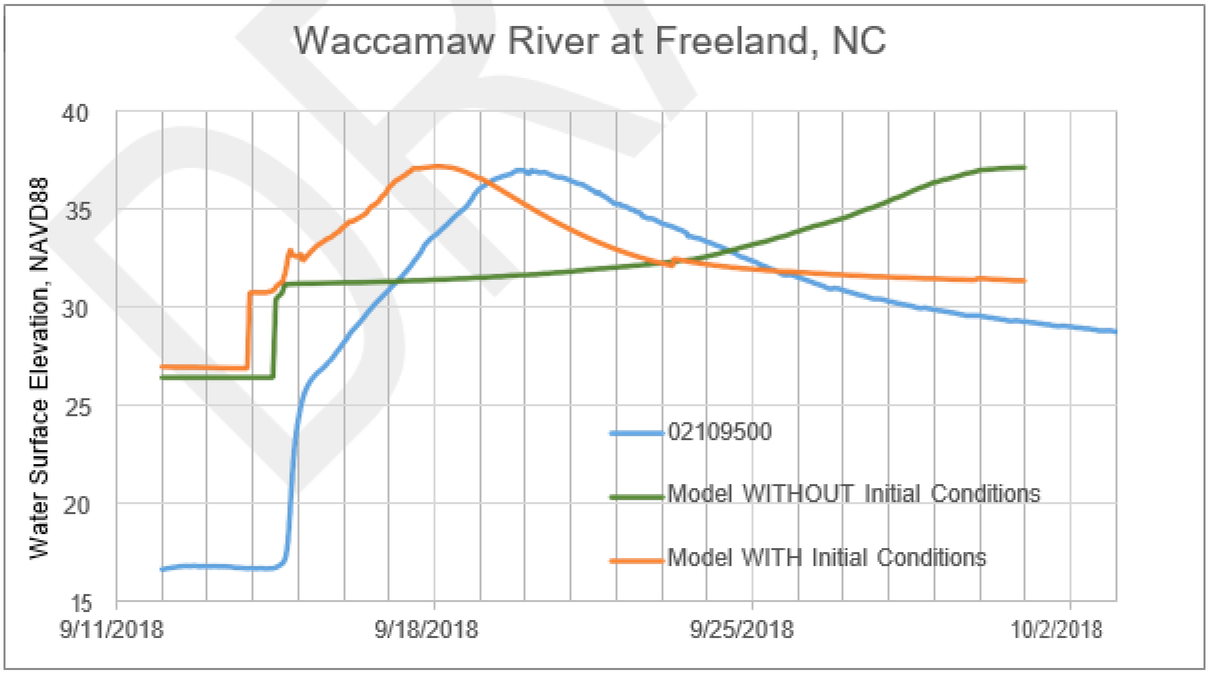


Figure E-49. Waccamaw River USGS Stream Gage vs. Model results for with and without Initial Conditions setup

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 should be considered when developing any models using the 2D rain-on-grid approach for this basin.

E.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 into the HEC-RAS terrain file as discussed in this report's Model Data and Layers section E.3. **Figure E-50** shows the USGS Stream gage at Freeland, NC (01209500) and the model results for the with and without the additional estimated channel bathymetry.

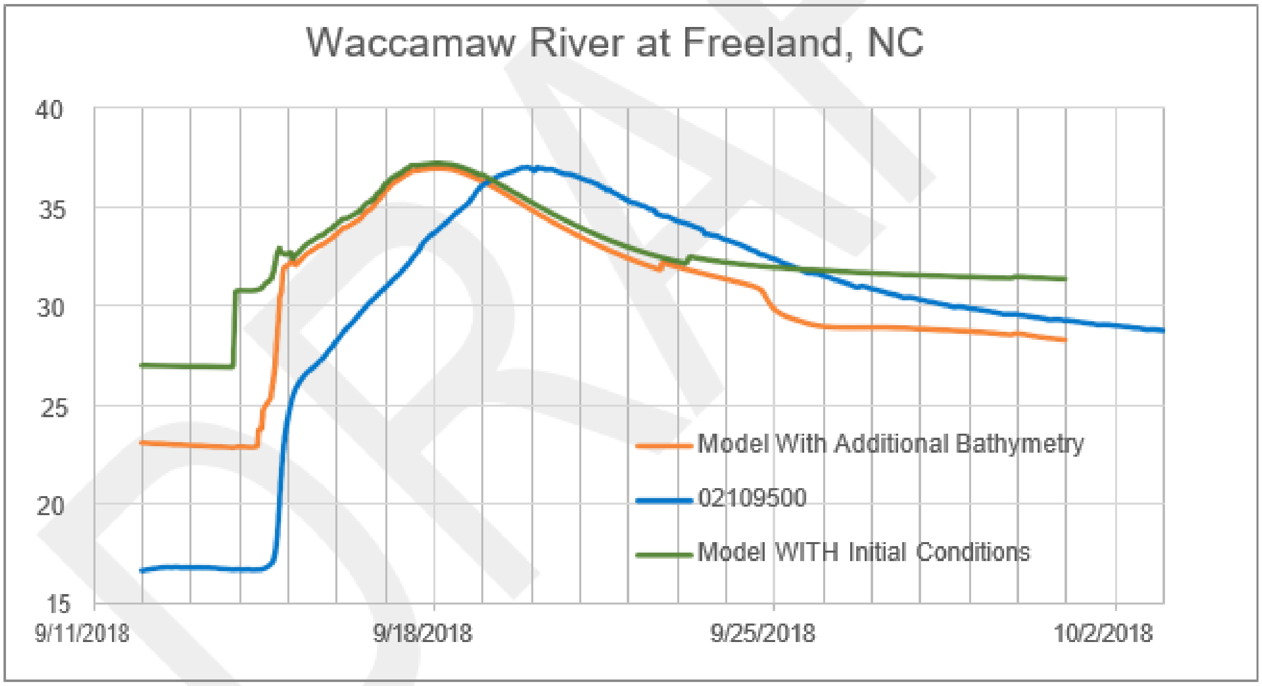


Figure E-50. Waccamaw River USGS Stream Gage vs. Model Results for With and Without Additional Bathymetry

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 E-51** shows an example of a terrain slice.

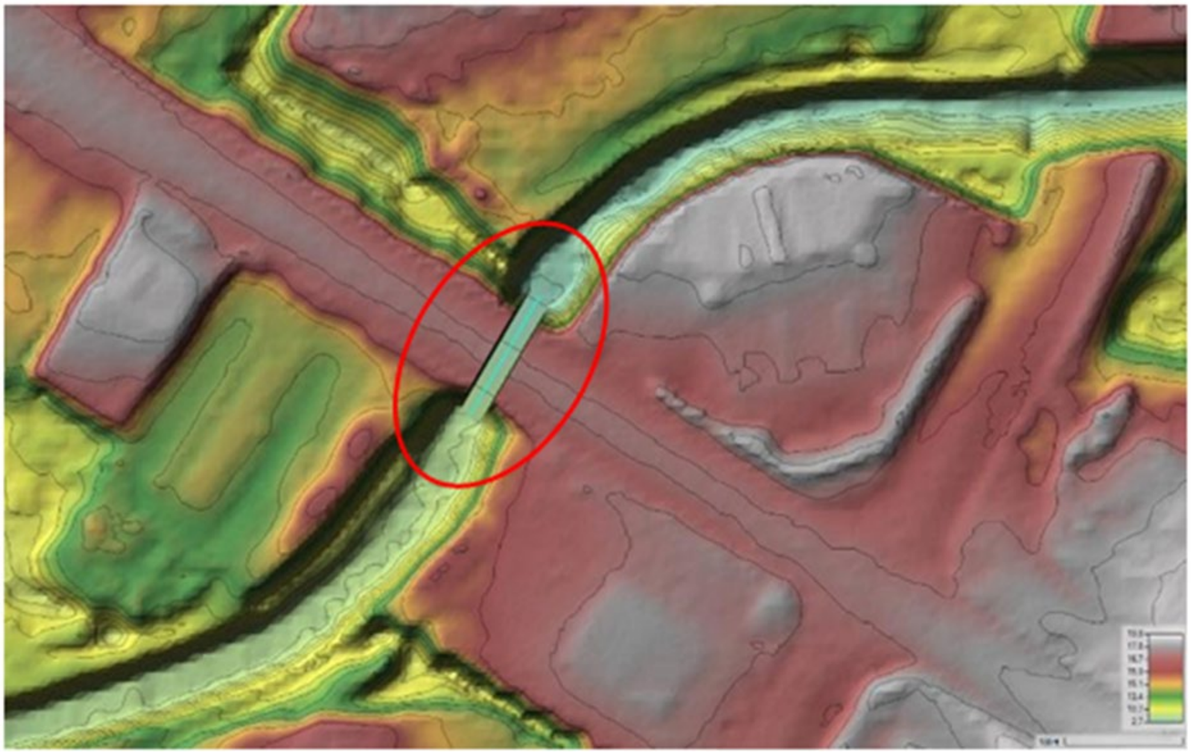


Figure E-51. Example of a “terrain slice” hydro-enforcement through a road embankment

E.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 have 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 E-7**.

Table E-7. 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 Intensity	0.03	0.03	0.04

NLCD ID	Land Cover Description	Base Area	Floodplain Area	Channel Area
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 E-52 shows the impact of the roughness value modifications compared to the “with additional bathymetry” simulation discussed in the previous section.

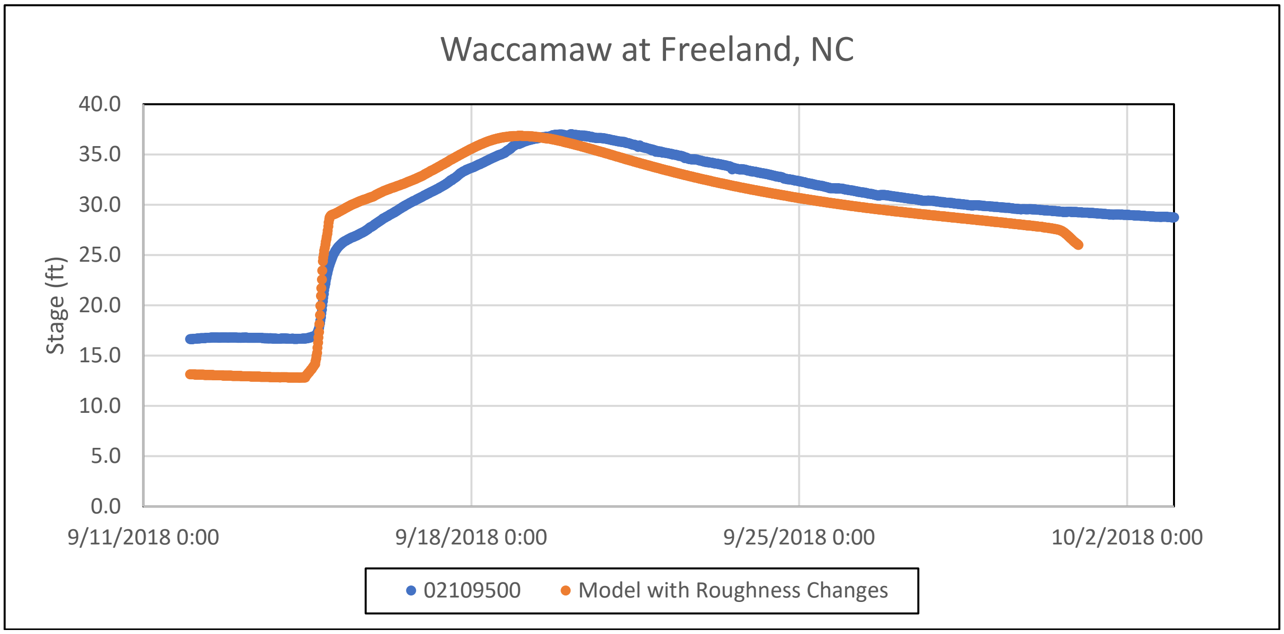


Figure E-52. Waccamaw River USGS Stream Gage vs. Model Results for Modifications to Roughness Values

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.

E.3.4.4 Hydraulic Structures

An analysis of how hydraulic structures, such as bridges, impact water surface elevations and flow within this watershed was performed. Given that most of the flow velocities throughout the Waccamaw River watershed are very low (less than 1 foot per second), it was important to test the benefits of incorporating the bridges into the 2D mesh, as they can sometimes cause local model instabilities. These instabilities can increase model run times and skew results.

After testing multiple bridges along the Waccamaw River, the results showed that the bridges caused minor water surface elevation impacts within the immediate vicinity (typically less than 0.1 feet). These minimal impacts were primarily attributed to the low channel and floodplain velocities. However, the bridge approach embankments had a more significant effect on restricting flow in the floodplains. The embankments are typically the main factor controlling flow losses at each roadway crossing, and as such, they were included in the 2D mesh to account for these losses.

It is important to note that debris, such as trees, at bridge openings could further impact flow and water levels, especially during the calibration and validation events. While this effect was not directly incorporated into the model, the presence of debris likely caused localized detention effects upstream of the bridges. These blockages could have altered flow rates and water levels, potentially impacting downstream gages. This effect, while not explicitly modeled, should be considered when interpreting the results, as debris at the bridges could have influenced flow dynamics during high-flow events.

Additionally, LiDAR imagery was utilized to refine land use and terrain data for the model. The high-resolution LiDAR data allowed for accurate identification of land use, including vegetation, impervious surfaces, and other surface features, which provided a more precise representation of runoff and flow paths throughout the watershed. The integration of LiDAR-derived data helped ensure the model accurately reflected both the natural and built environments, improving the overall hydraulic representation.

In total, nine bridges were included in the model geometry, with the embankments playing a key role in flow restriction across the floodplain. While the effect of debris at bridge openings was not directly included in the model, the analysis accounted for the major flow restrictions posed by the embankments and other structural features.

E.3.4.5 Hydrometeorological Non-Stationarity Sensitivity Analysis

The sensitivity of the Waccamaw River's hydrologic response to rainfall 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/ and summary description is included in Appendix A-2. 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 values account for potential future increases in the frequency and intensity of extreme rainfall events (see Appendix A-2 for more detail regarding future precipitation trends).

We recognize that the NCICS data is provided for research purposes, and the results of the sensitivity test are not meant to imply precise predictive capabilities, instead the sensitivity evaluation conducted was used to gauge the possible magnitude of changing IDF relationships relative to baseline hydrologic conditions.

Future period IDF relationships provided by the NCICS tool can be used to adjusts the current NOAA Atlas 14 IDF curves. For the sensitivity test, the 2075 RCP4.5 scenario (mid-range greenhouse gas scenario with an approximately 50-year horizon) was selected. This scenario represents the 30-year period centered around 2075 (2060-2090) and coincides with the end of the project's period of economic analysis (2085).

The NCICS tool supports evaluation by selected longitude and latitude. A representative precipitation depth for the Waccamaw River area is selected using representative output from the NCICS tool near Conway, SC to generate IDF relationships IDF (Latitude: 33.85559 and Longitude -78.9368).

At the location evaluated, the 96-hour (4-day), 1% AEP rainfall increases from the Atlas 14 baseline value of approximately 12.68 inches to 14.53 inches. This is an increase of approximately 14.6%. This percent increase was then applied to the 1% 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 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, the percentage of change varies depending on event magnitude evaluated. When the NCICS 2075 IDF relationship is compared to the Atlas 14 baseline curve there is a 10.85% increase in the 2-year event and a 12.03% difference in the 10-year event. The percent increase may also be higher or lower depending on the scenario analyzed. For instance, the percent increase for a 96-hour, 1% AEP event is 26.58% for RCP 8.5. Further investigation is required to fully evaluate this variability.

While we understand the suggestion to test a broader range of rainfall increases (e.g., 5%, 10%, 15%) and intensities in HEC-HMS, we view this use of NCICS data as sufficient for the scope of this sensitivity analysis. The goal was to provide an initial understanding of potential future increases in flood risks. .

The results of the simulation indicate that rainfall 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 E-53**. It should be noted that the 90% confidence intervals for the rainfall values are large for the 1% AEP event, 10.70 to 15.93 inches for Atlas 14 and 11.73 to 19.12 inches for the NCICS values. This represents distribution fitting uncertainty. There is additional considerable uncertainty associated with the future condition IDF relationships (see Appendix A-2 for more information).

Any further refinement or broader sensitivity testing can be considered in future project phases if needed. Risks associated with future increases in extreme precipitation intensity and frequence have been identified in the residual risk table in Appendix A-2.

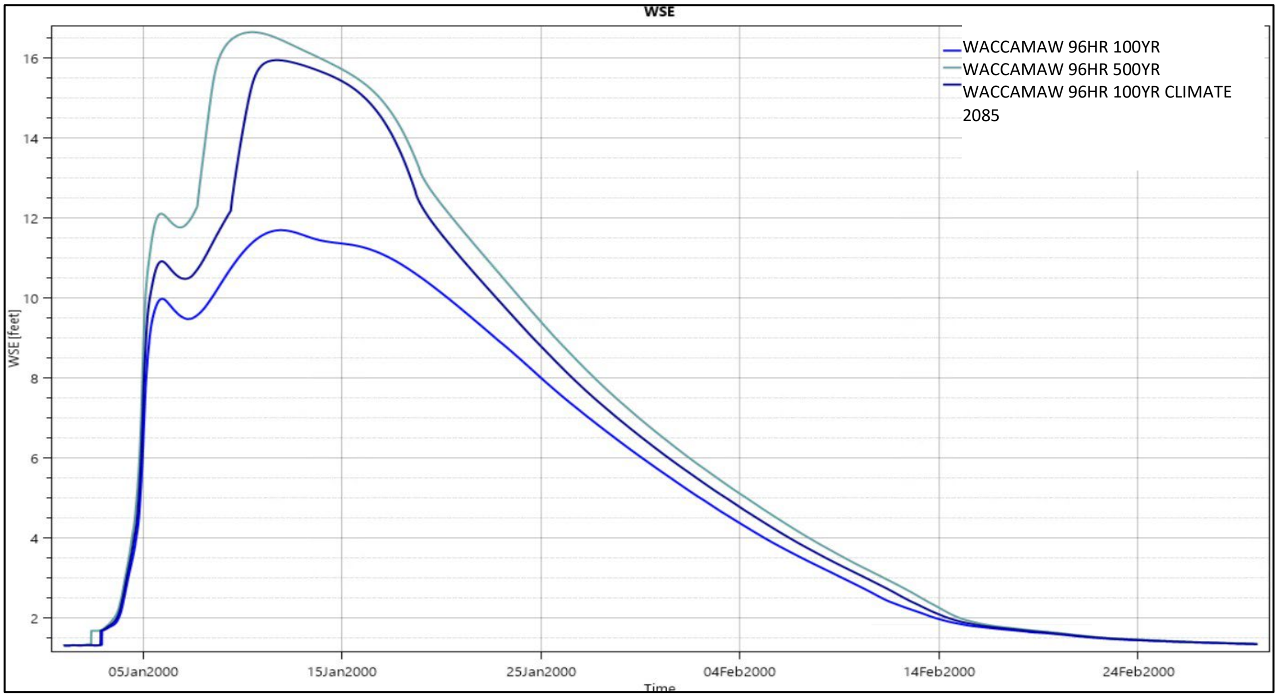


Figure E-53. Model Results for Waccamaw River at Conway, SC- Comparison of Current versus Future Hydrometeorological Conditions

E.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, the period of economic analysis (2035-2085), and the design life of the project (approximated by 2035-2135). 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 both the 50-Year period applied for economic analysis and across the 100-year adaptation horizon 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 nearby NOAA water level gage with a sufficiently long data record and then to use the USACE Sea Level Analysis Tool (SLAT) to define SLC at the coast for the three USACE SLC scenarios. This evaluation is included in detail in Appendix A2: Changing Conditions.

In addition to using the SLAT. The NOAA Sea Level Rise Viewer Level Viewer is used to assess areas that are tidally influenced. **Figure E-54** shows NOAA Sea Level Rise Viewer in the Bucksport focus area during MHHW conditions. Bucksport is located towards the downstream end of the study area and is heavily affected by the riverine flooding from both the Waccamaw and Pee Dee River.

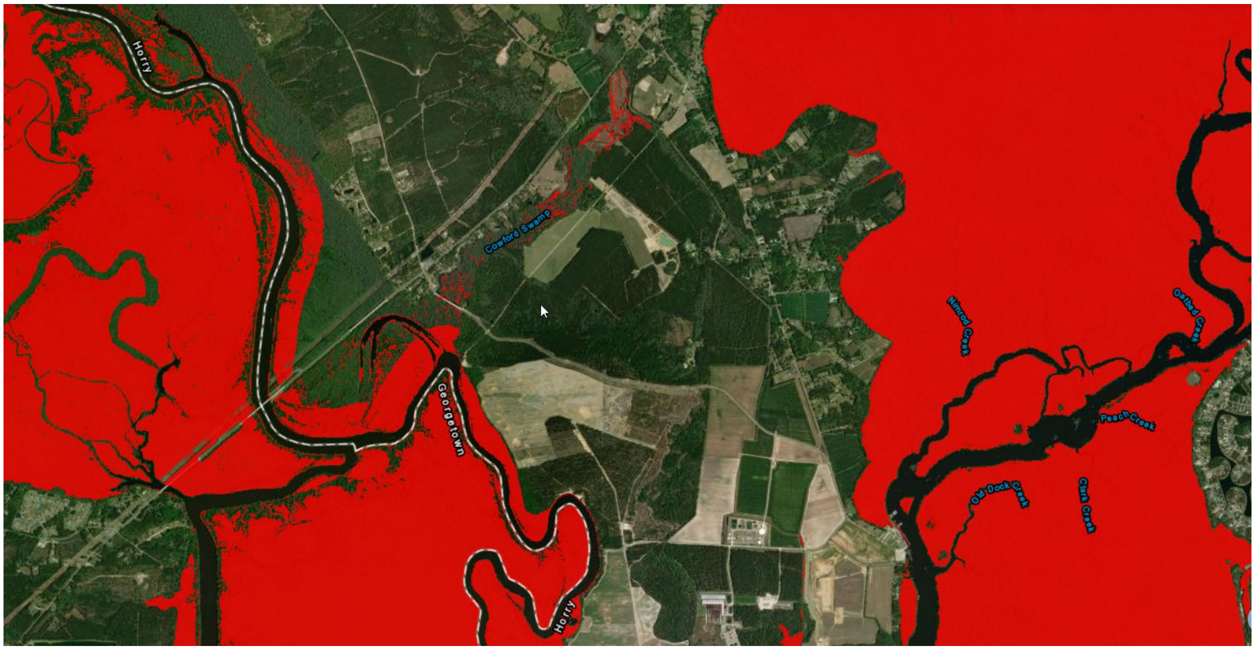


Figure E-54. MHHW level for Bucksport, SC from the NOAA Sea Level Rise Viewer

Four cross sections were obtained at various locations along the Pee Dee River and Waccamaw River with the cross-sectional value of water surface elevation comparisons with (1)fluvial-only 1% AEP WSL, (2) USACE High SLC (2085) at 1% AEP WSL, and (3) the SLC and Astronomical High Tide (HAT) combination at the 1% AEP WSL. HAT 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 **Figure E-55** through **Figure E-62**. The SLC and tidal effect further upstream near the proposed structural measures at Conway and Socastee (**Figure-E56** profile 2) was observed to be less than a 0.05 ft in difference for the combination of SLC and extreme tidal conditions relative to the 1% AEP WSL modeled without coastal effects. Differences in water surface elevation even closer to the project features near Conway and along the Peewee River are not discernable (see **Figure E-60** and **Figure E-62)**. The furthest downstream cross-section experienced 1.44 ft difference (see **Figure E-58**). However, there are no proposed structural measures at this location or nearby.

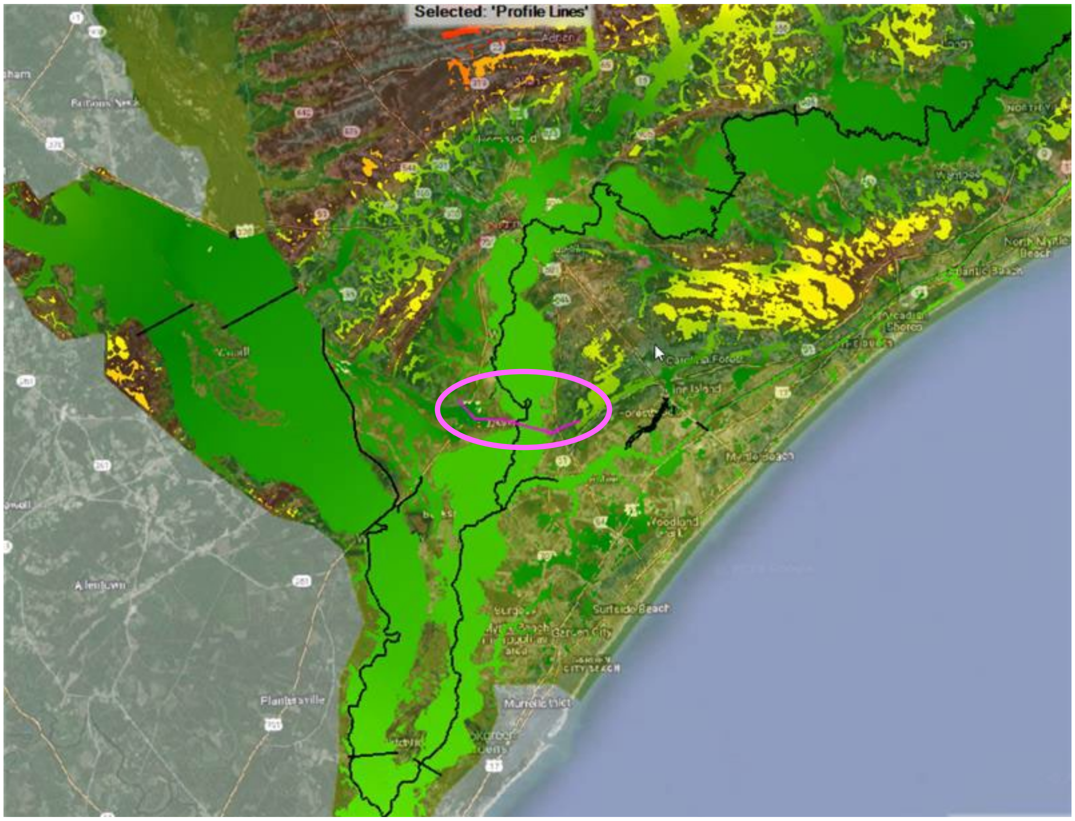


Figure E-55. Cross section profile line 2 downstream of Conway and Socastee

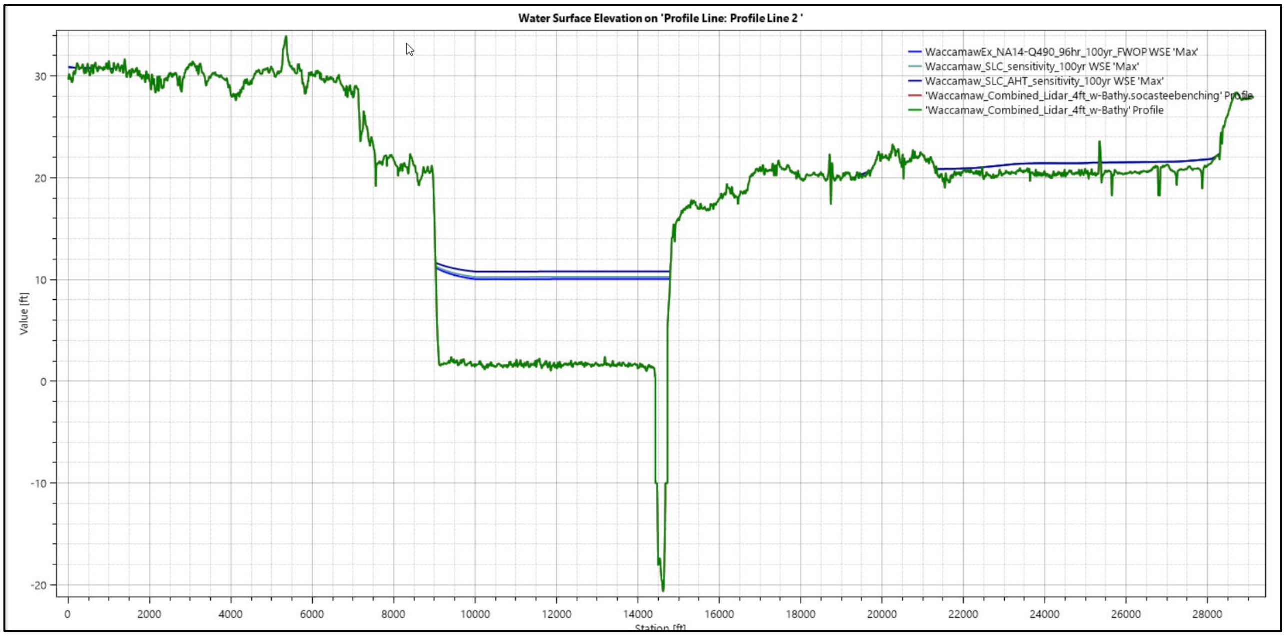


Figure E-56. Cross section WSE (max) with FWOP (baseline), 2085 High SLC only, and SLC with Astronomical high tide at Profile 2

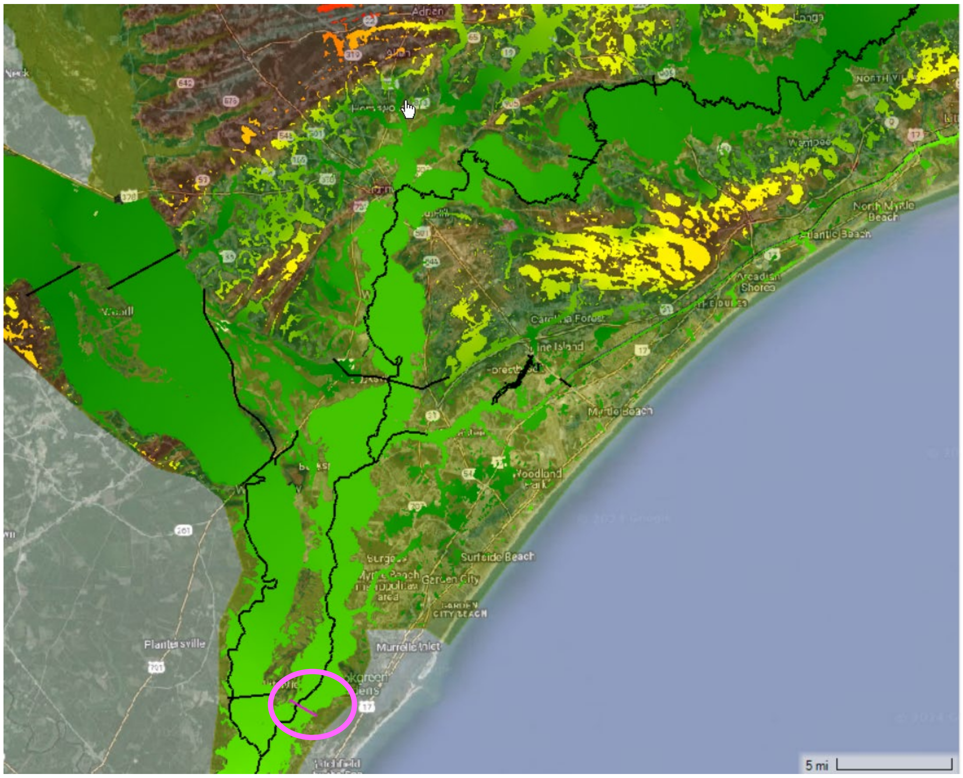


Figure E-57. Furthest downstream cross section profile line ()

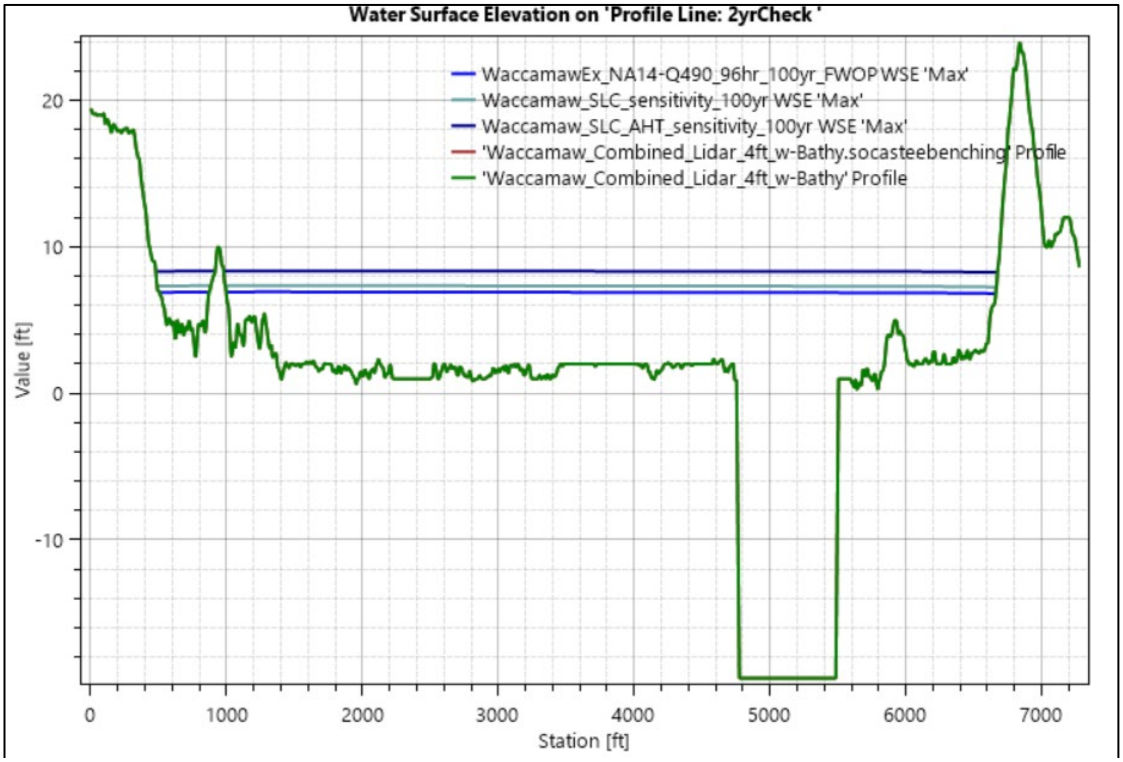


Figure E-58. Cross section WSE (max) with FWOP (baseline), 2085 High SLC only, and SLC with Astronomical high tide at Downstream Most Profile

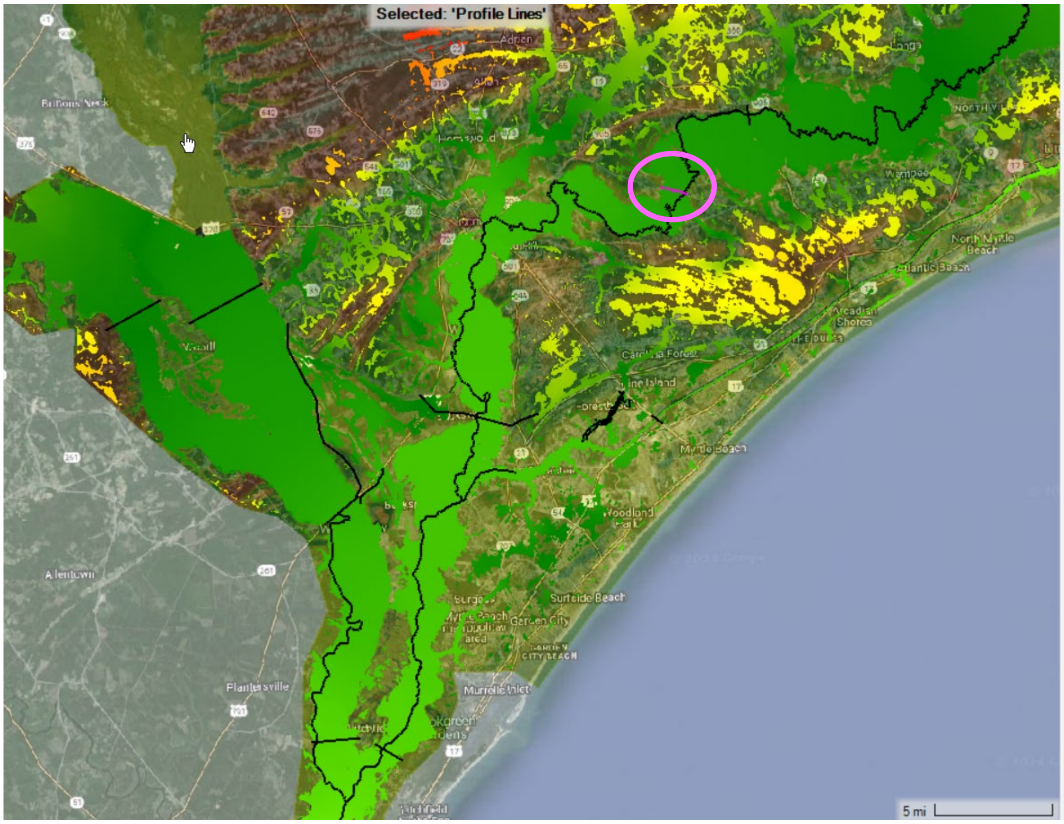


Figure E-59. Cross section location 'Profile Line 10' (in pink) in Conway

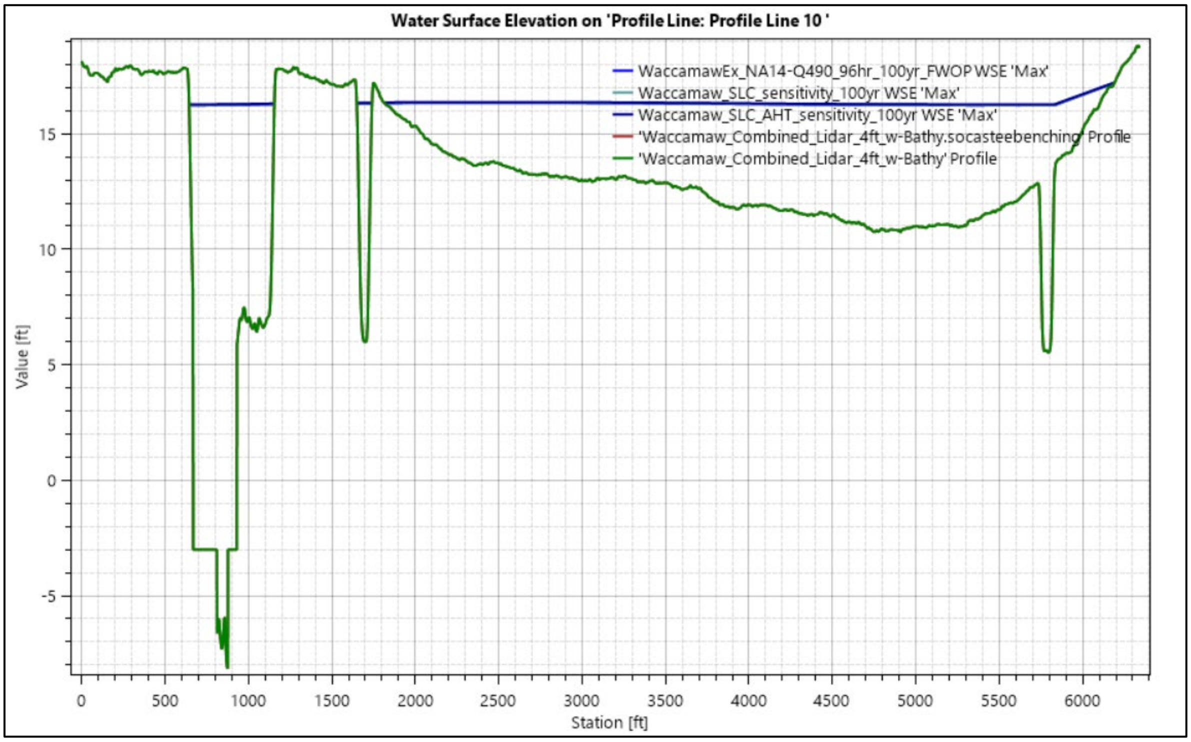


Figure E-60. Cross section WSE (max) with (baseline), 2085 High SLC only, and SLC with Astronomical high tide at Profile Line 10

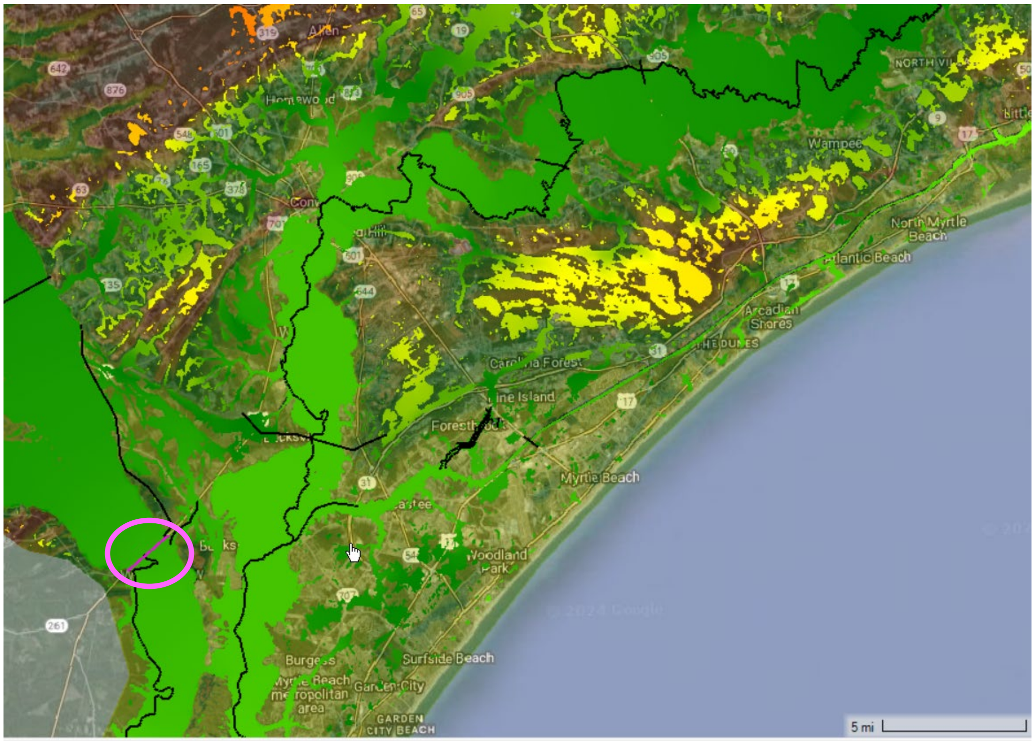


Figure E-61. Cross section location (in pink) Pee Dee River

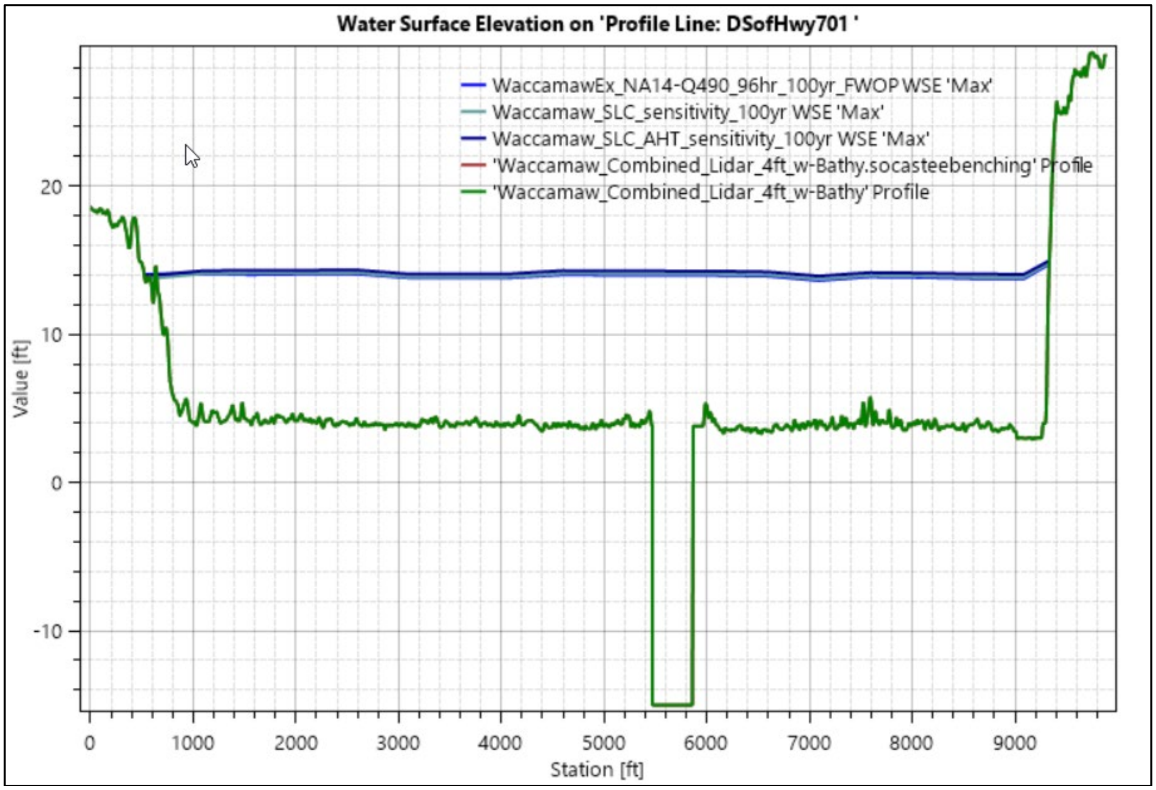


Figure E-62. Cross section WSE (max) with FWOP (baseline), 2085 High SLC only, and SLC with Astronomical high tide at profile downstream of Highway 701- Pee Dee River

E.3.4.7 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)
- Rainfall 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 have 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 rainfall 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 E-30**. 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 E-8**.

Table E-8. 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

Model	Model Description
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 E-9**, **Table E-10**, and **Table E-11** 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 E-9. 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
Waccamaw near Longs	2110500	9/21/2018 19:30	9/21/2018 6:15	0.6
BuckCreek near Longs	2110400	9/21/2018 10:00	9/20/2018 23:15	0.4
Waccamaw above Conway	2110550	9/23/2018 21:45	9/22/2018 14:15	1.3
Crabtree Swamp at Conway	2110701	9/26/2018 0:30	9/23/2018 15:30	2.4
Waccamaw atConwayMarina	2110704	9/26/2018 0:45	9/23/2018 16:15	2.4
AIWW at Hwy544	2110725	9/27/2018 13:30	9/26/2018 23:00	0.6
Waccamaw at Bucksport	2110802	9/27/2018 7:45	9/26/2018 21:15	0.4
Waccamaw 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 E-10. 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
Waccamaw near Longs	2110500	10/14/2016 10:30	10/12/2016 23:45	1.4
BuckCreek near Longs	2110400	10/9/2016 6:30	10/12/2016 19:30	3.5
Waccamaw above Conway	2110550	10/16/2016 18:30	10/14/2016 15:00	2.1
Waccamaw ConwayMarina	2110704	10/18/2016 5:15	10/16/2016 2:00	2.1
AIWW at Hwy544	2110725	10/18/2016 0:30	10/8/2016 20:30	9.2
Waccamaw at Bucksport	2110802	10/22/2016 1:00	10/17/2016 19:45	4.2
Waccamaw 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 E-11. 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
Waccamaw at Longs	2110500	10/6/2015 7:00	10/5/2015 14:00	0.7
Buck Creek near Longs	2110400	10/5/2015 10:15	10/7/2015 12:45	2.1
Waccamaw above Conway	2110550	10/8/2015 2:00	10/8/2015 4:00	0.1
Crabtree Swamp at Conway	2110701	10/5/2015 16:15	10/6/2015 23:45	1.3
Waccamaw at Conway Marina	2110704	10/10/2015 17:00	10/7/2015 7:30	3.4
AIWW at Hwy544	2110725	10/11/2015 22:45	10/5/2015 11:45	6.5
Waccamaw at Bucksport	2110802	10/12/2015 2:15	10/11/2015 6:00	0.8
Waccamaw 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

E.3.5 Future Sea Level Change Considerations

Per Engineering and Construction Bulletin 2018-14, EP 1100-2-1 "Procedures to Evaluate Sea Level Change: Impacts, Responses and Adaptation" and ER 1100-2-8162 "Incorporating Sea Level Change in Civil Works Programs", a 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 Bucksport, Socastee, Conway, and Longs/Red Bluff were under consideration. **Figure E-63** through **Figure E-66** show the NOAA Sea Level Rise Viewer with the combination of MHHW tidal conditions and two feet of projected Sea Level Rise..

Figure E-63. NOAA Sea Level Rise Viewer – MHHW + 2' SLR for focus area, Bucksport.

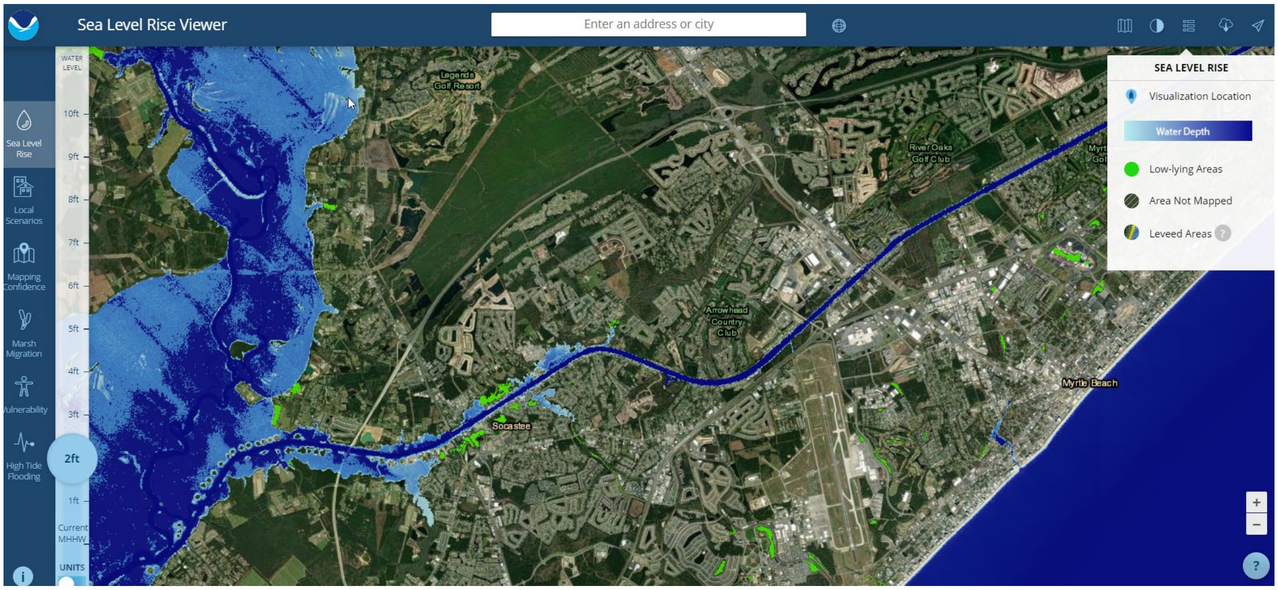


Figure E-64. NOAA Sea Level Rise Viewer – – MHHW + 2’ SLR at Socastee Creek

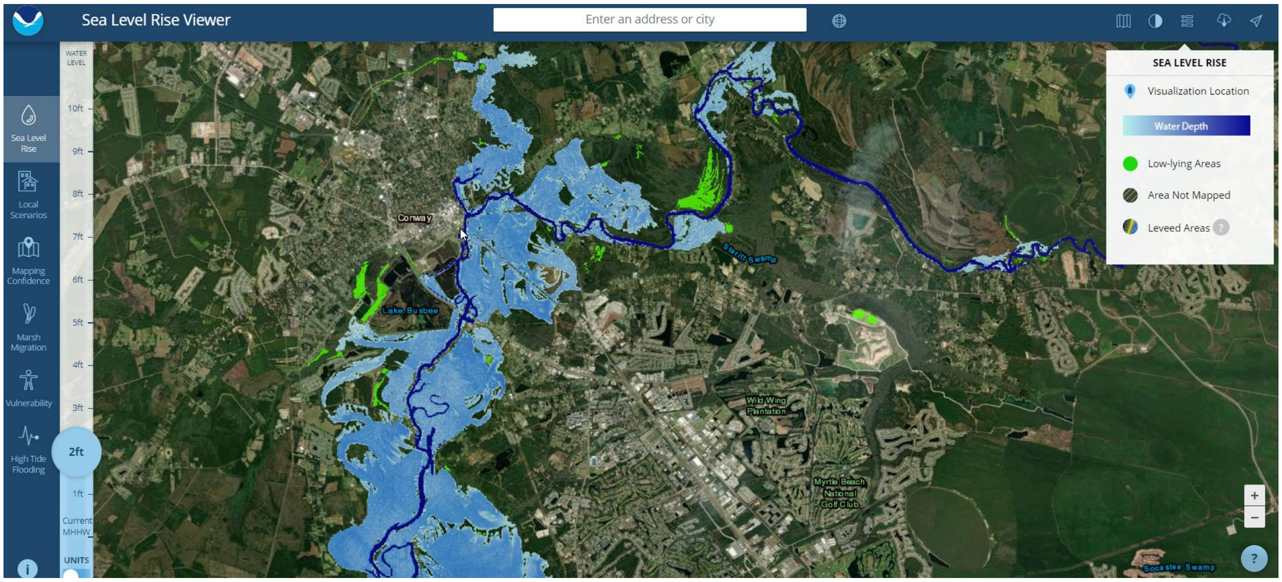


Figure E-65. NOAA Sea Level Rise Viewer – – MHHW + 2’ SLR at Conway

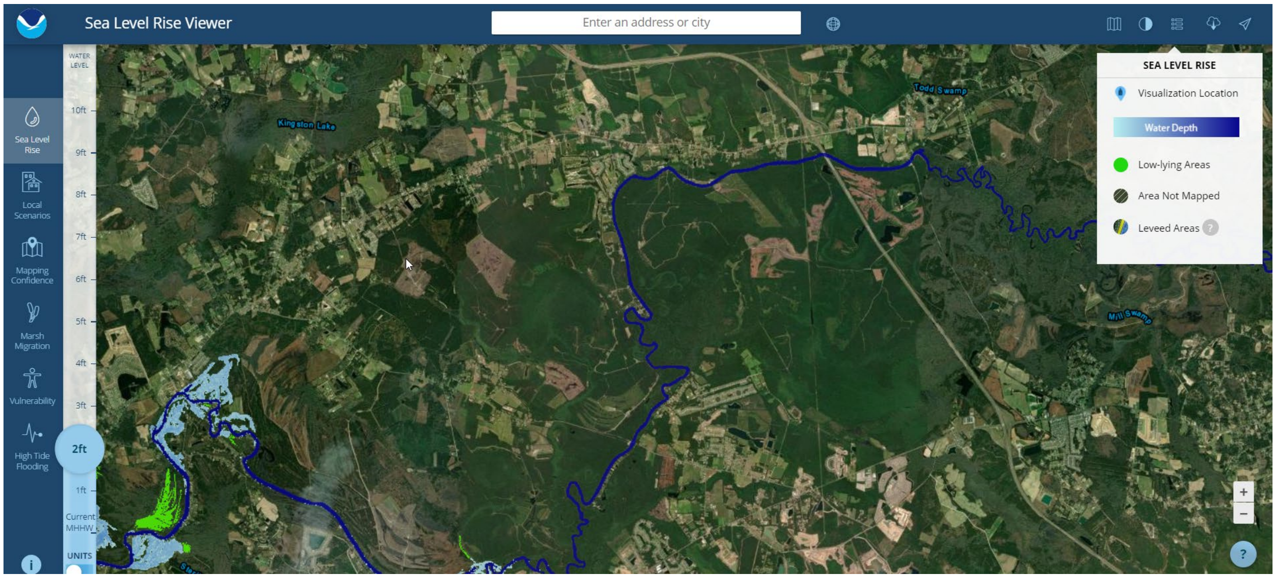


Figure E-66. NOAA Sea Level Rise Viewer – – MHHW + 2’ SLR at Longs/Red Bluff

The study utilized the USACE Sea Level Analysis Tool (SLAT), 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 gages with USACE SLC scenarios (Low, Intermediate, High) derived from global and local effects as per USACE guidelines. The study conducted a sensitivity analysis to assess the impact of sea level change (SLC) on hazard levels for the Waccamaw River project. The SLC analysis is presented in Appendix A2.

The SLAT 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 0.0108 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 0.54 ft based on this regional rate.

SLC sensitivity simulations were conducted using HEC-RAS. SLC was modeled based on all three SLC scenarios. The USACE High scenario represents the most conservative SLC scenario projecting a 3.76 ft increase in the water surface at the coastline 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, High Scenario-year 2085 SLC, and High Scenario-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 E-36**-**Figure E-44**). This methodology allowed for a comprehensive analysis of how projected SLC could influence flood hazard levels along the Waccamaw River in the vicinity of project features, aiding in resilience planning and infrastructure design considerations. Based on the analysis conducted, SLC will not affect the performance of retained measures modeled within the study over the both the 50-year economic period of analysis or the longer, 10--year adaptation horizon(see Appendix A-2 for more detail).

F EXISTING CONDITIONS MODEL AND RESULTS

F.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 E.2 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 F-2** through **Figure F-4**.

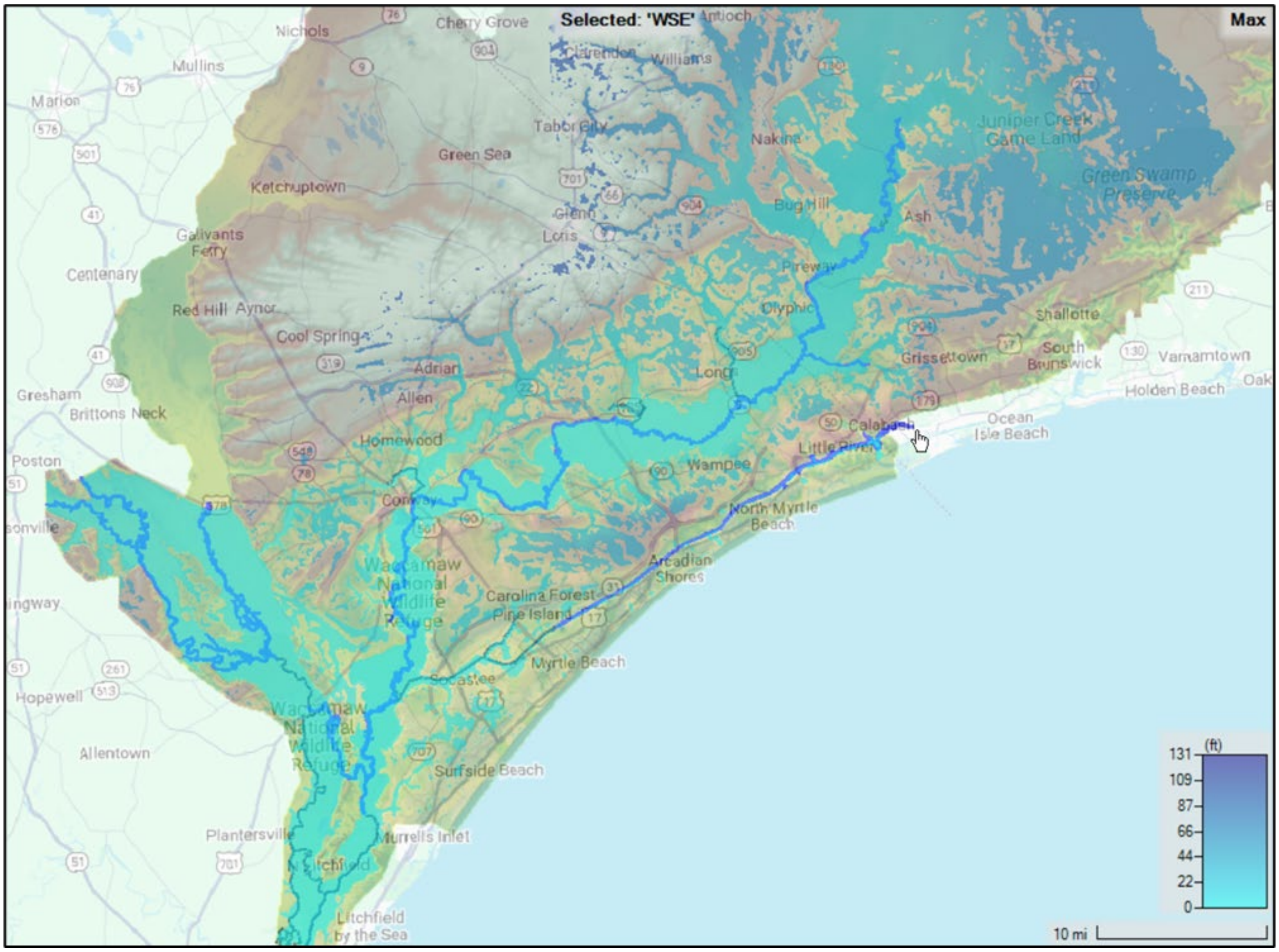


Figure F-1. 1% AEP floodplain for entire watershed

The synthetic events were developed using the HEC-HMS model with updated rainfall parameters. The HEC-HMS model used in this study was part of the Flood Insurance Study (FIS) for the Waccamaw River, updated in 2017 and 2019, with calibration performed using Bulletin 17B/C guidelines. This calibration confirmed that the peak flows reflected in the model align with historical gage data. Our team reviewed the model and the accompanying report and found it suitable for the purposes of this study.

Additionally, the 2018 AECOM 2D model provided further validation, using USGS gage data and NOAA Atlas 14 precipitation data. The results of that model were within 1% for discharge and 1.5 feet for water surface elevations at the USGS 02135200 gage, demonstrating strong agreement with historical observations. Based on this and the prior calibration efforts, we consider the existing model reliable for this study, and further re-calibration is not anticipated to add significant value.

While we recognize that all models have inherent limitations, especially when relying on historical data, we believe that this model's consistency with observed data makes it appropriate for the analysis. 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. **Figure E-54** through **Figure E-56** 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.

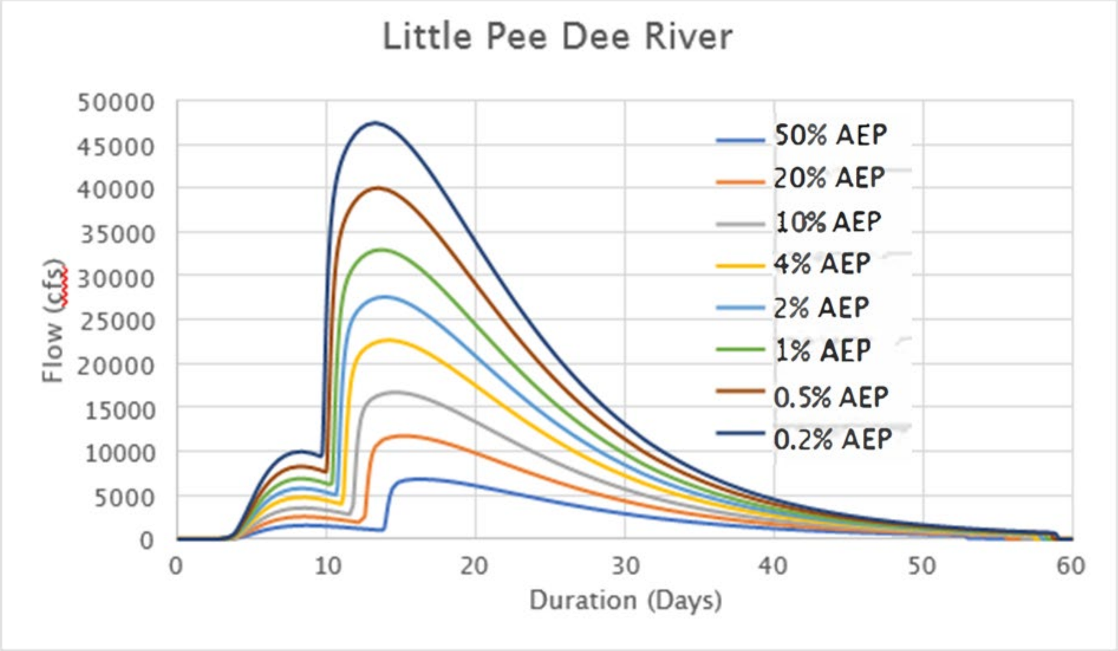


Figure F-2. Inflow hydrographs for the Little Pee Dee River for the Synthetic Rainfall events

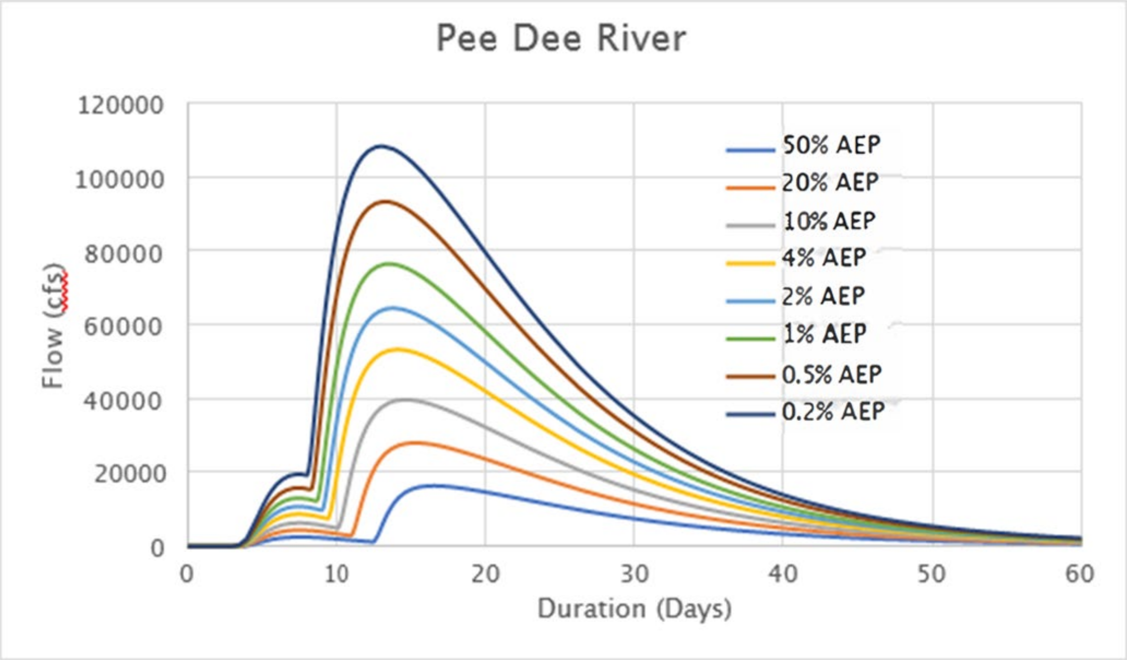


Figure F-3. Inflow hydrographs for the Pee Dee River for the Synthetic Rainfall events

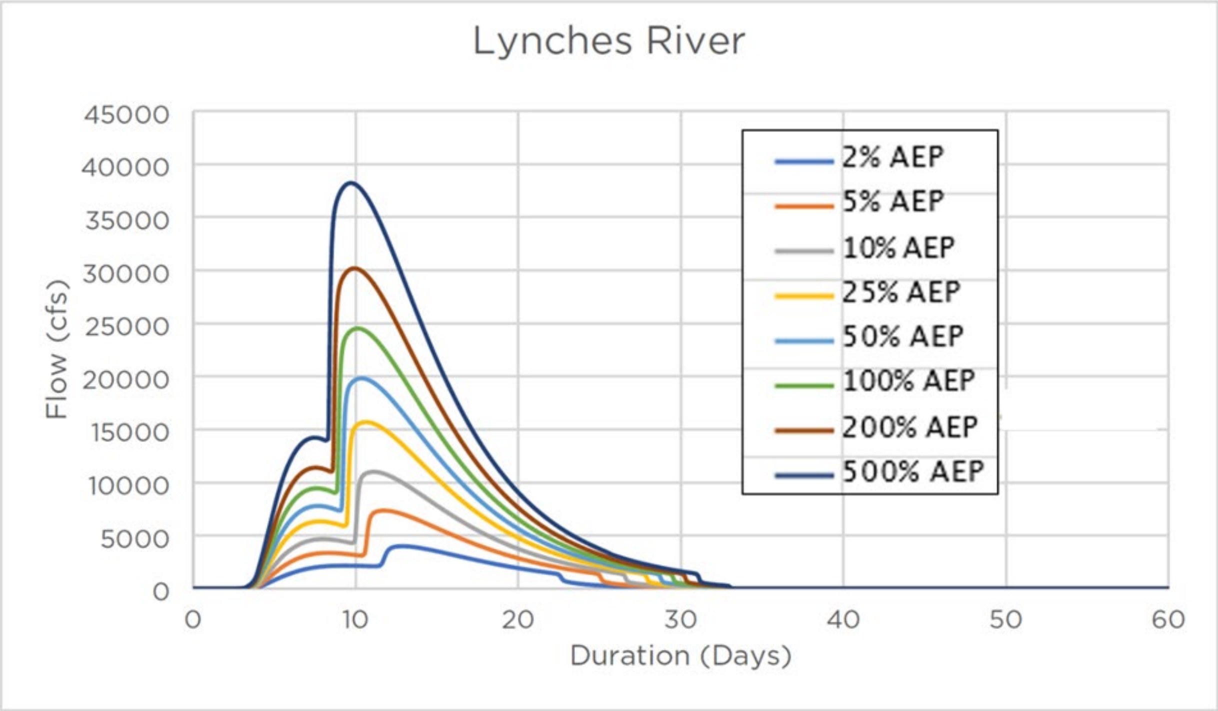


Figure F-4. Inflow hydrographs for the Lynches River for the Synthetic Rainfall events

Table F-1. List of simulated events

Simulation Description	Purpose	Percent Error
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.	0.02391
Hurricane Florence	Simulate Hurricane Florence for purposes of model calibration.	0.00967
Hurricane Matthew	Simulate Hurricane Matthew for purposes of model calibration.	0.0156
Hurricane Joaquin	Simulate Hurricane Joaquin for purposes of model calibration.	0.0457
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.	0.02391
50% AEP, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 2-year storm event.	0.06146
20% AEP, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 5-year storm event.	0.03573
10% AEP, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 10-year storm event.	0.02292
4% AEP, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 25-year storm event.	0.009057
2% AEP, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 50-year storm event.	0.007329
1% AEP, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 100-year storm event.	0.01782
0.5%, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 200-year storm event.	0.01949
0.2% AEP, 96-hour, NOAA Atlas 14 Q4 90% Rainfall	Simulate the approximate 500-year storm event.	0.004790

Table F-1 shows the list of simulated events, the purpose and the percent error associated with each simulation. The average percent volume error for all runs was 0.0228 with a max of 0.06146 and minimum of 0.00479. **Figure F-5** through **Figure F-7** show the extents of the floodplain for synthetic events, 2%, 1% and 0.5% AEP.

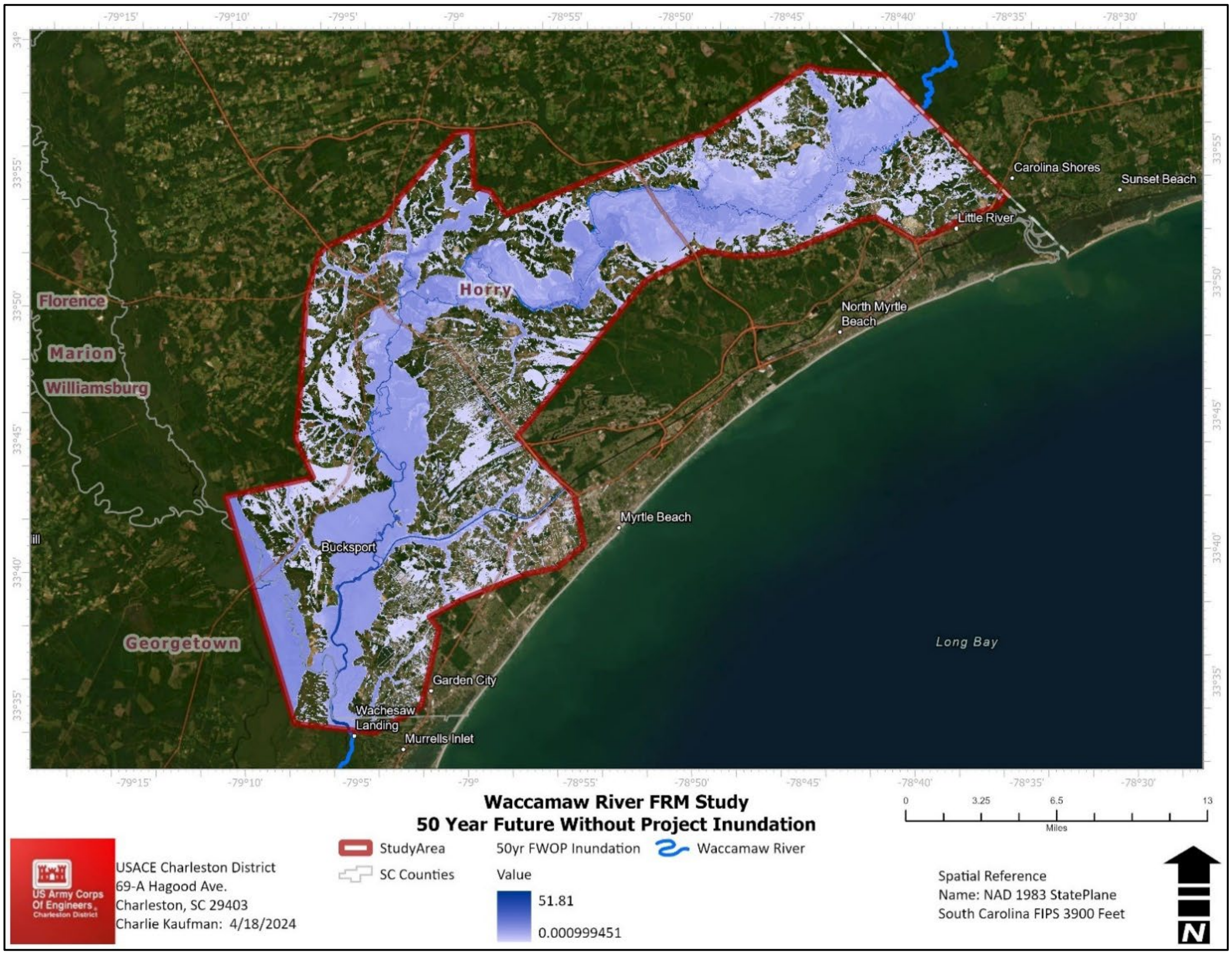


Figure F-5. 2% AEP (50-year) 96 hour NOAA Atlas 14 Q4 90% Rainfall event

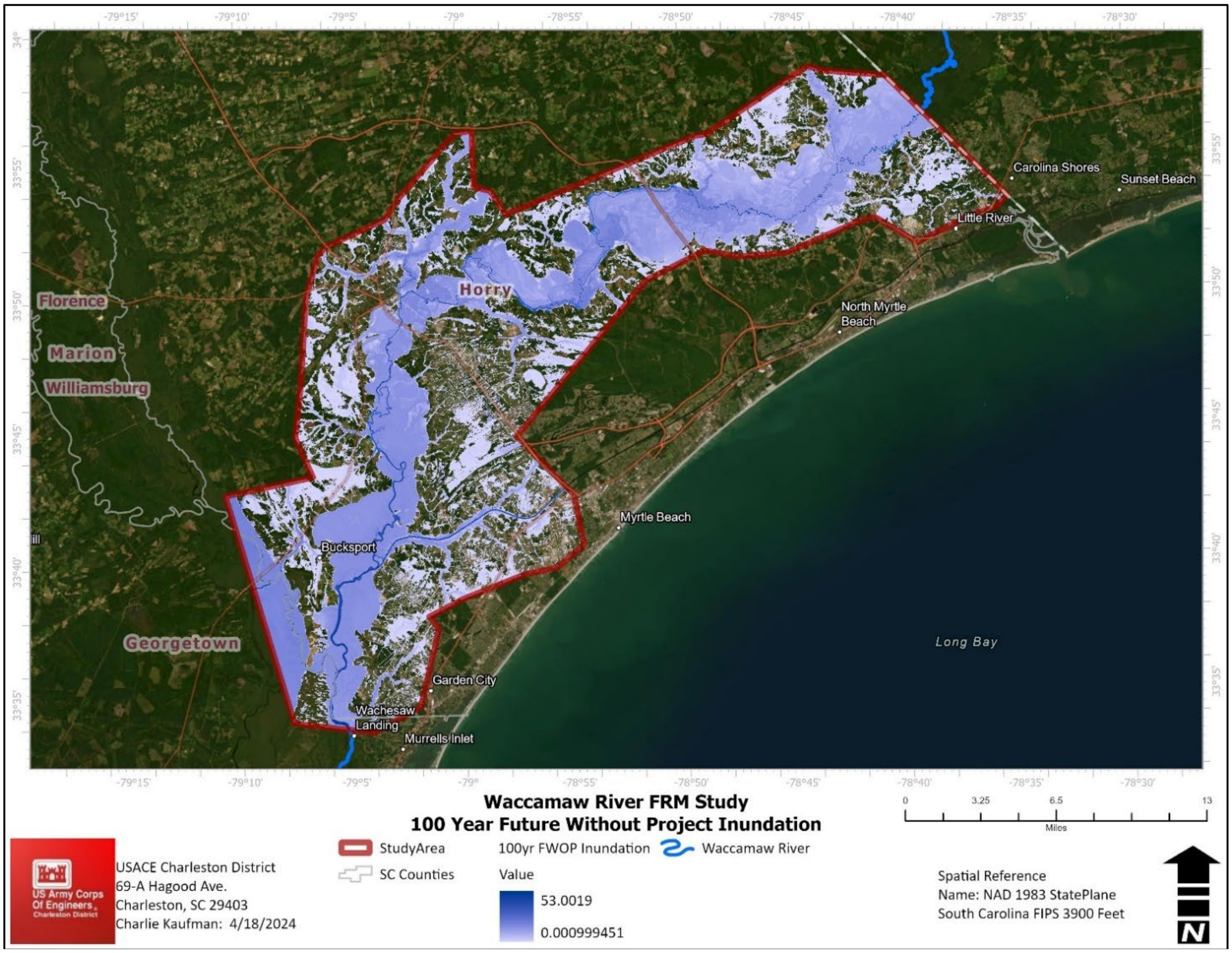


Figure F-6. 1% AEP (100 Year) 96 hour NOAA Atlas 14 Q4 90% Rainfall event

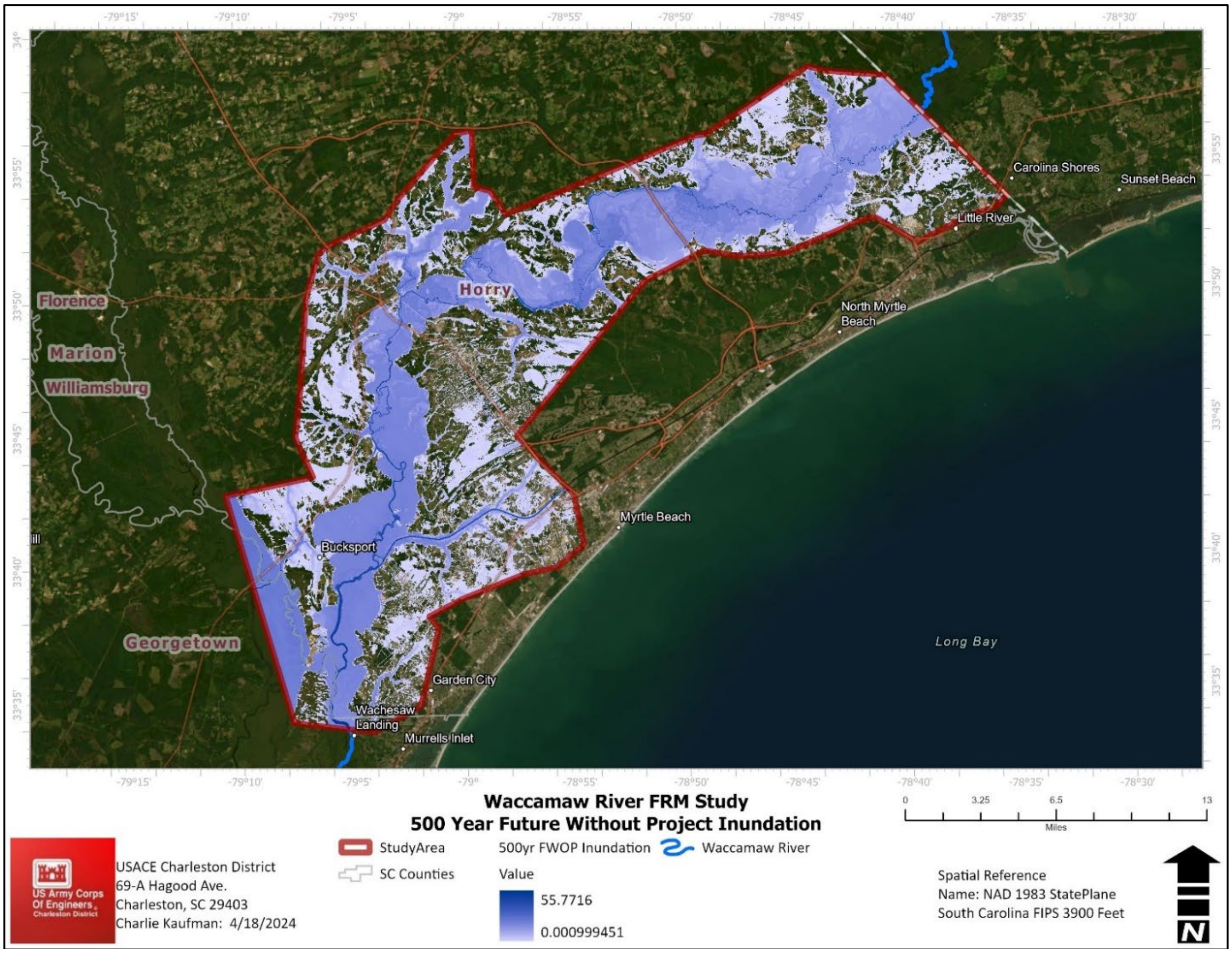


Figure F-7. 0.2% AEP (500 year) 96 hour NOAA Atlas 14 Q4 90% Rainfall

F.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 G-1** through **Figure G-17** 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 rainfall adjustment to 2085 is also included in these data comparisons.

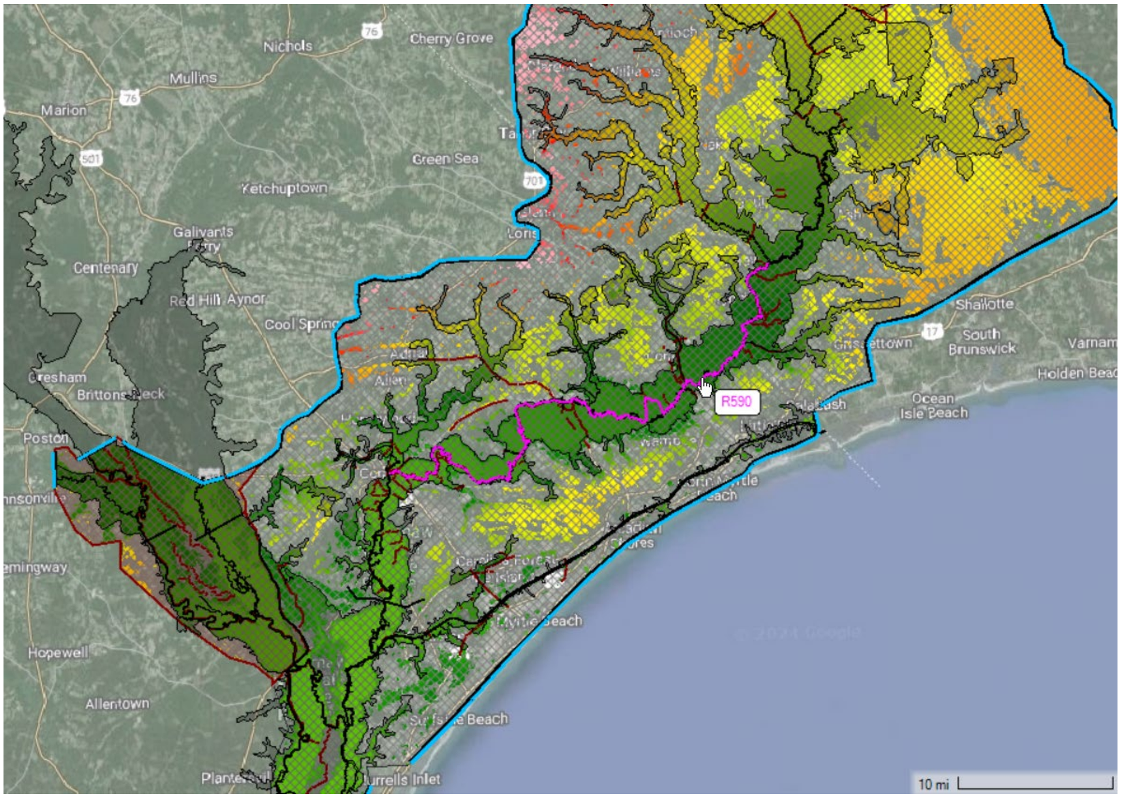


Figure F-8. Location (in pink) of the Middle Waccamaw River WSE (max) data comparison (Conway)

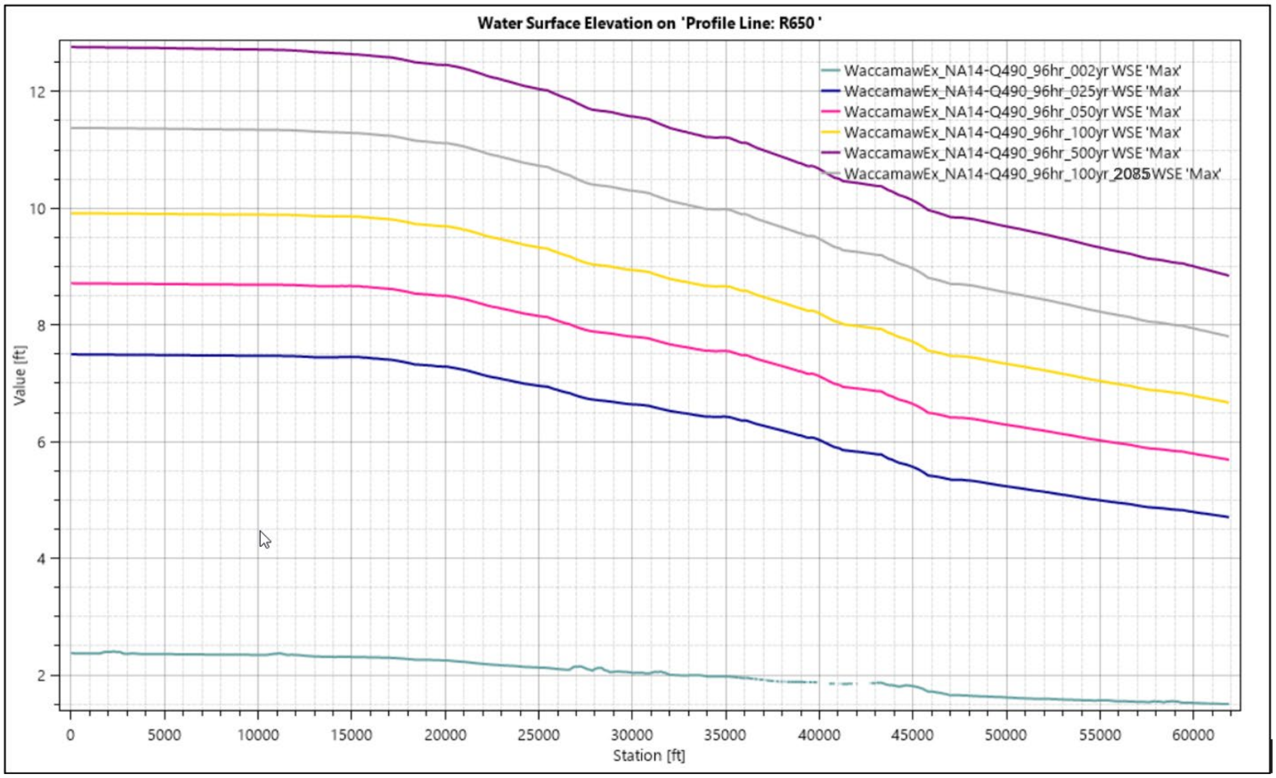


Figure F-9. WSE (max) data comparison for middle location (Conway)

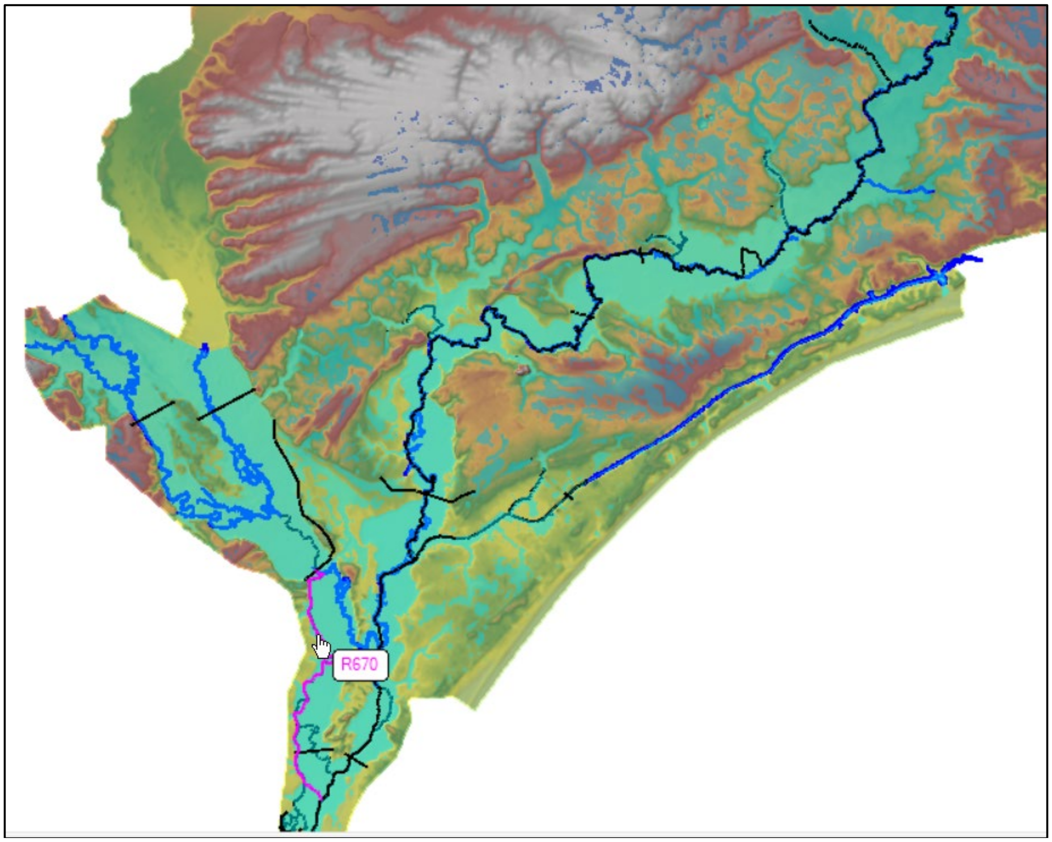


Figure F-10. Location (in pink) of the Pee Dee River WSE (max) data comparison (Bucksport)

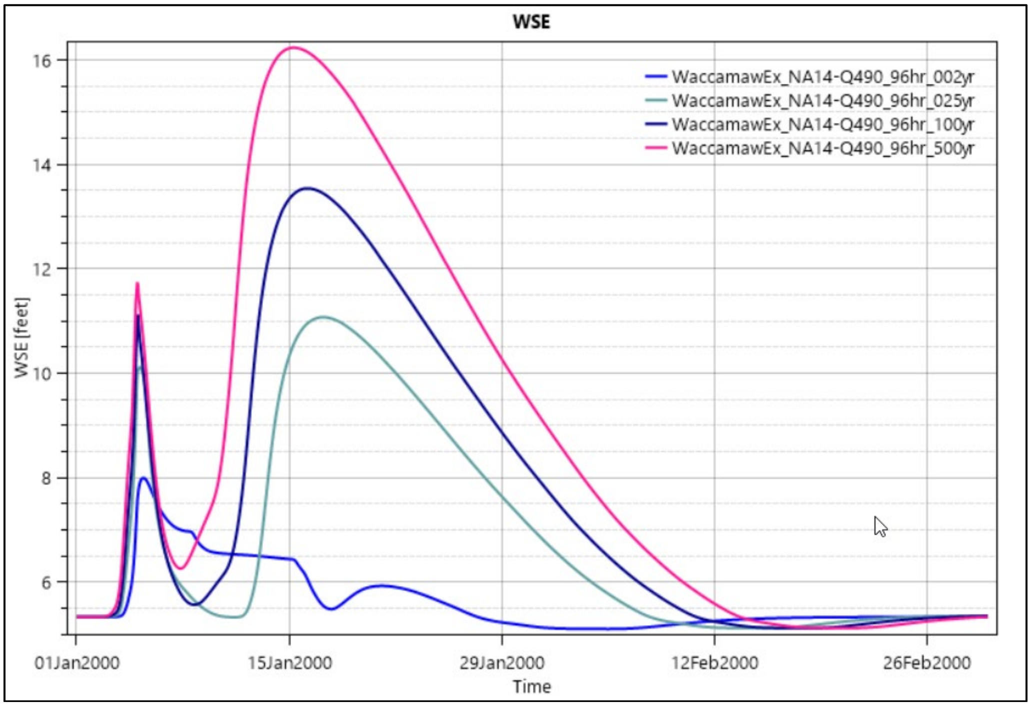


Figure F-11. WSE comparison at a data point along the Pee Dee River for different synthetic storm events (Bucksport)

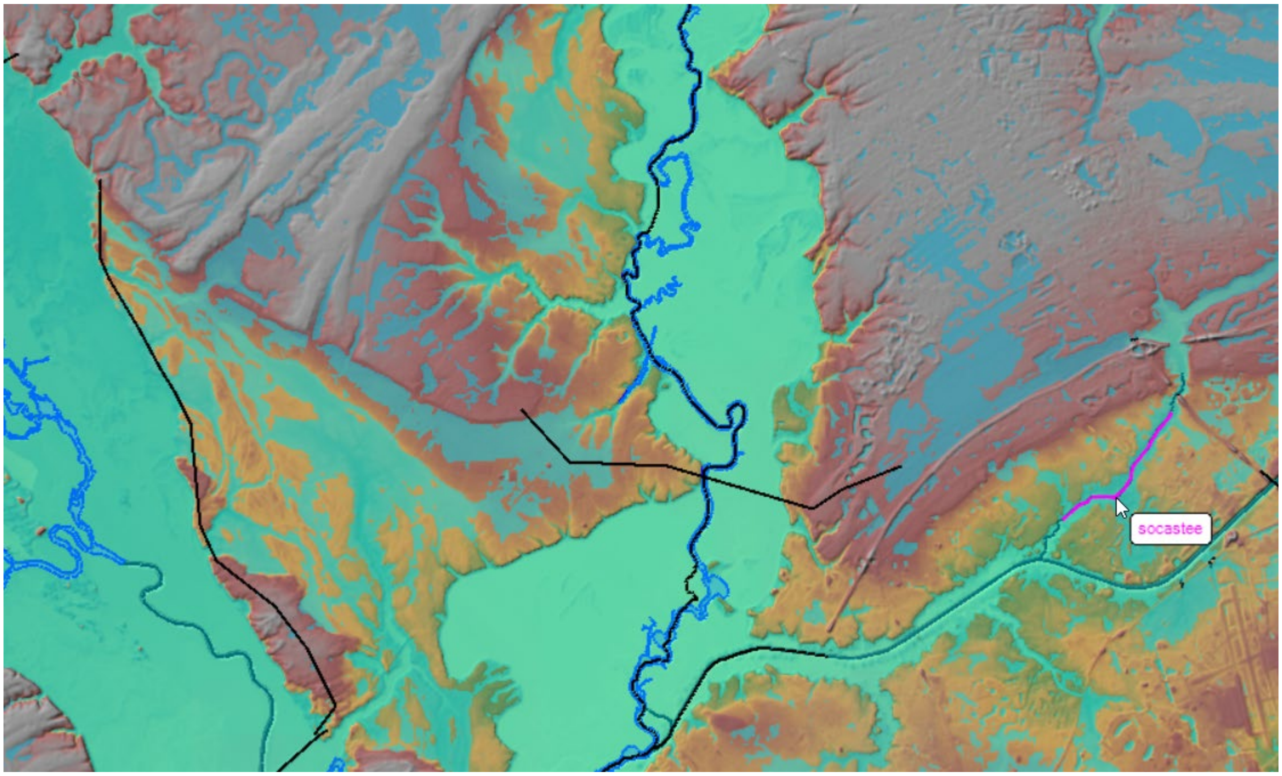


Figure F-12. Location (in pink) of the WSE (max) data comparison (Socastee Creek)

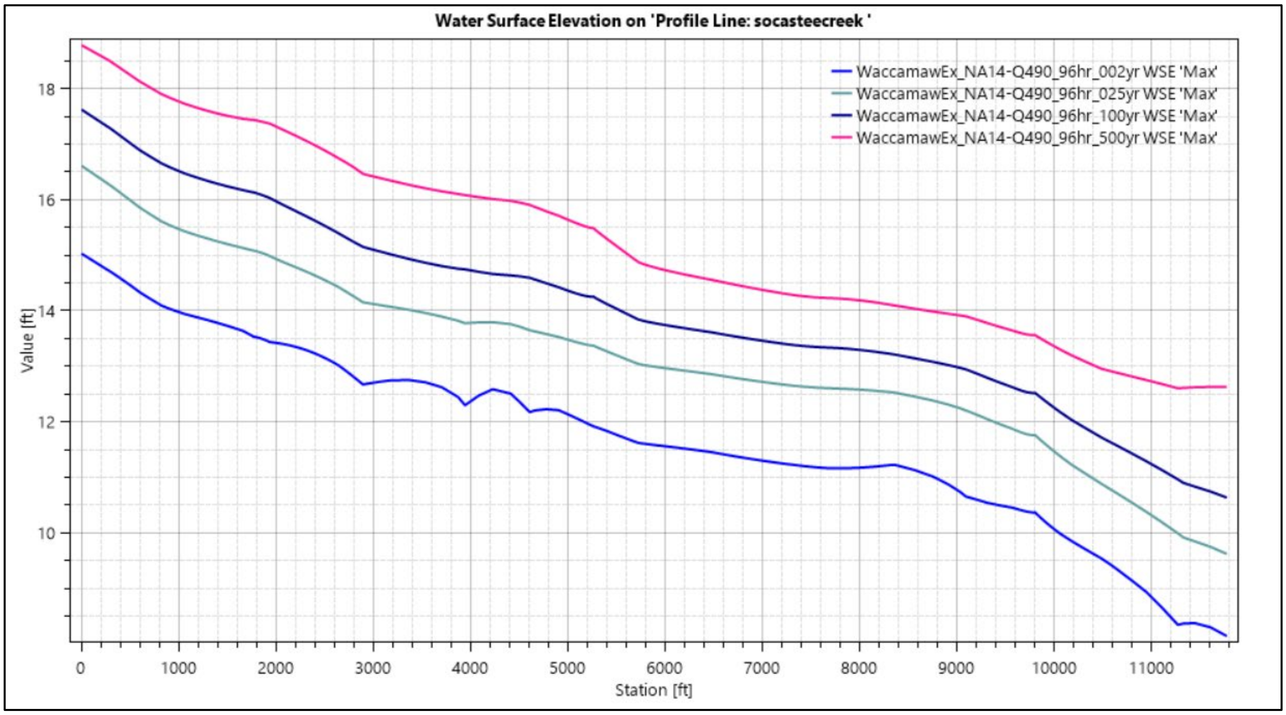


Figure F-13. WSE data comparison along Socastee Creek

Figure F-14 through **Figure F-17** show the 50%, 4%, 1% and 0.2% AEP events mapped together for the study areas of interest.

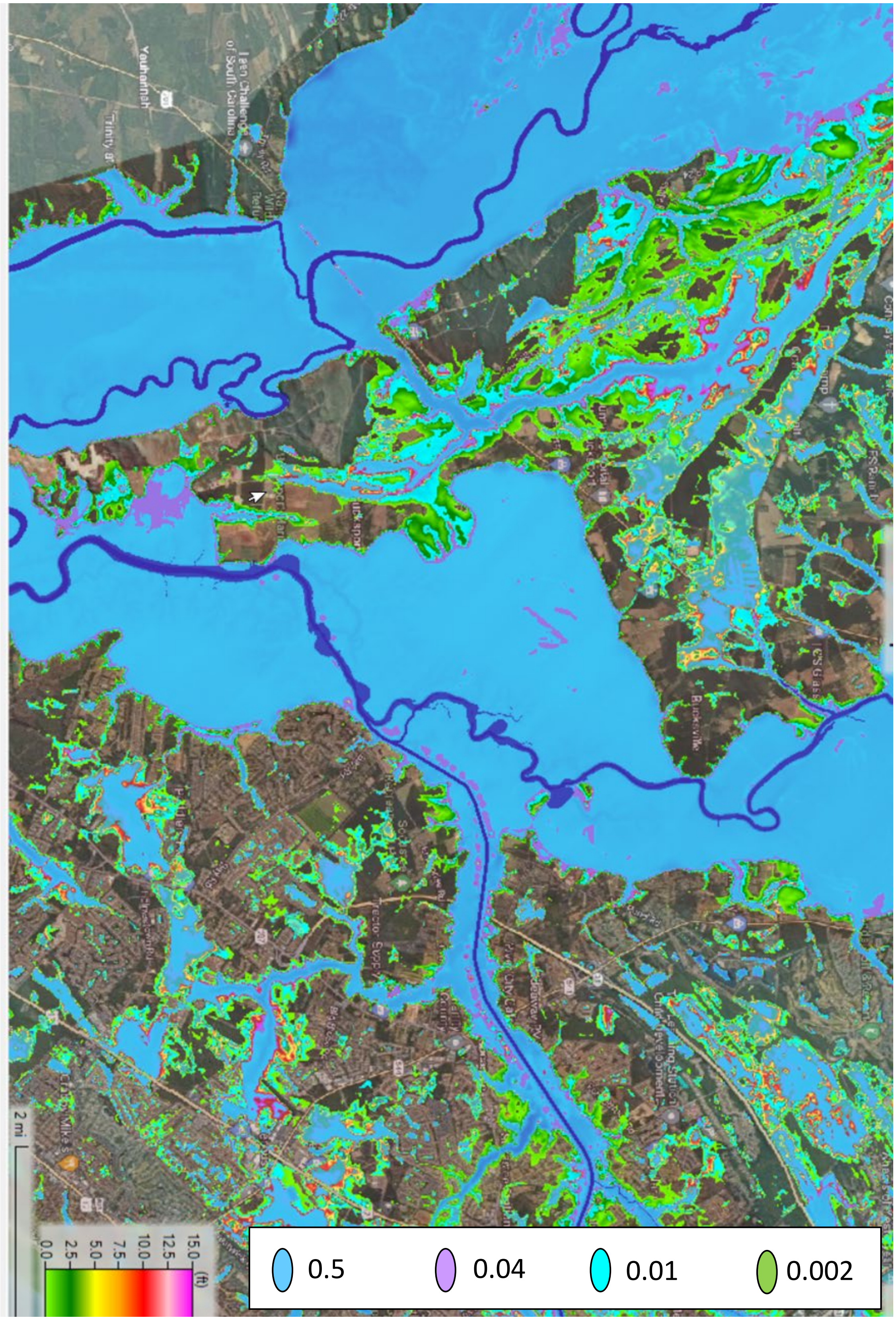


Figure F-14. Bucksport Existing Conditions

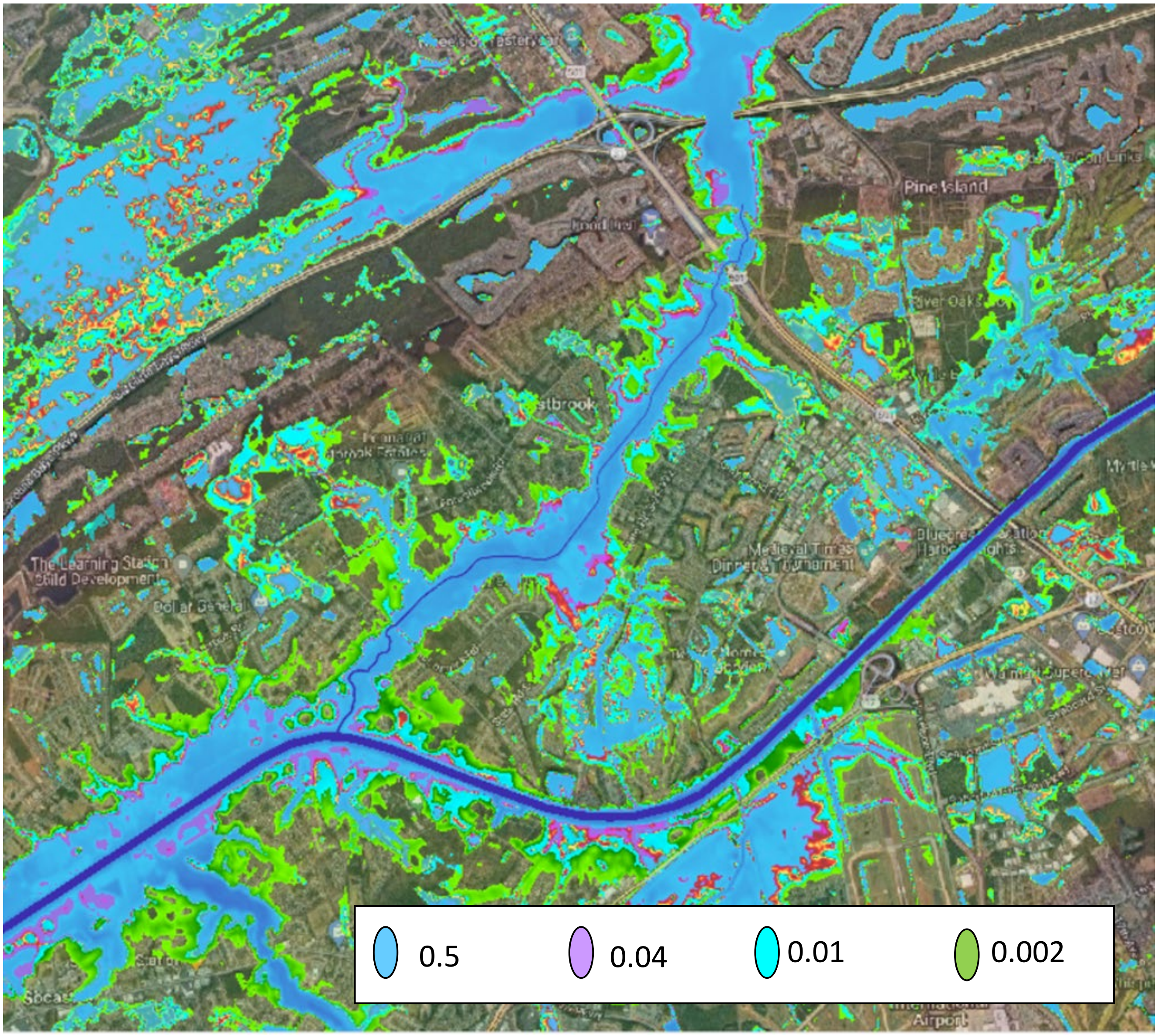


Figure F-15. Socastee Existing Conditions

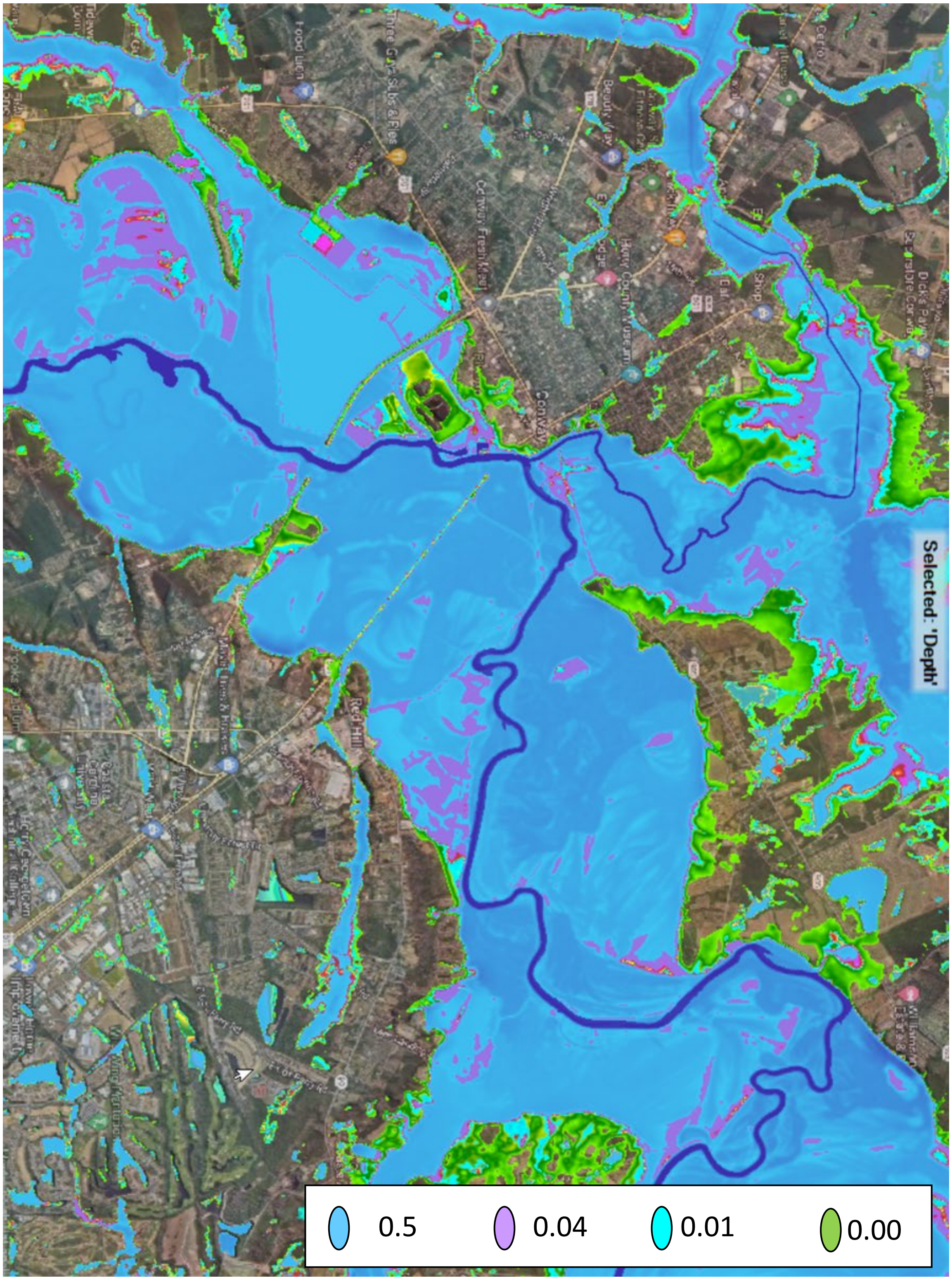


Figure F-16. Conway Existing Conditions

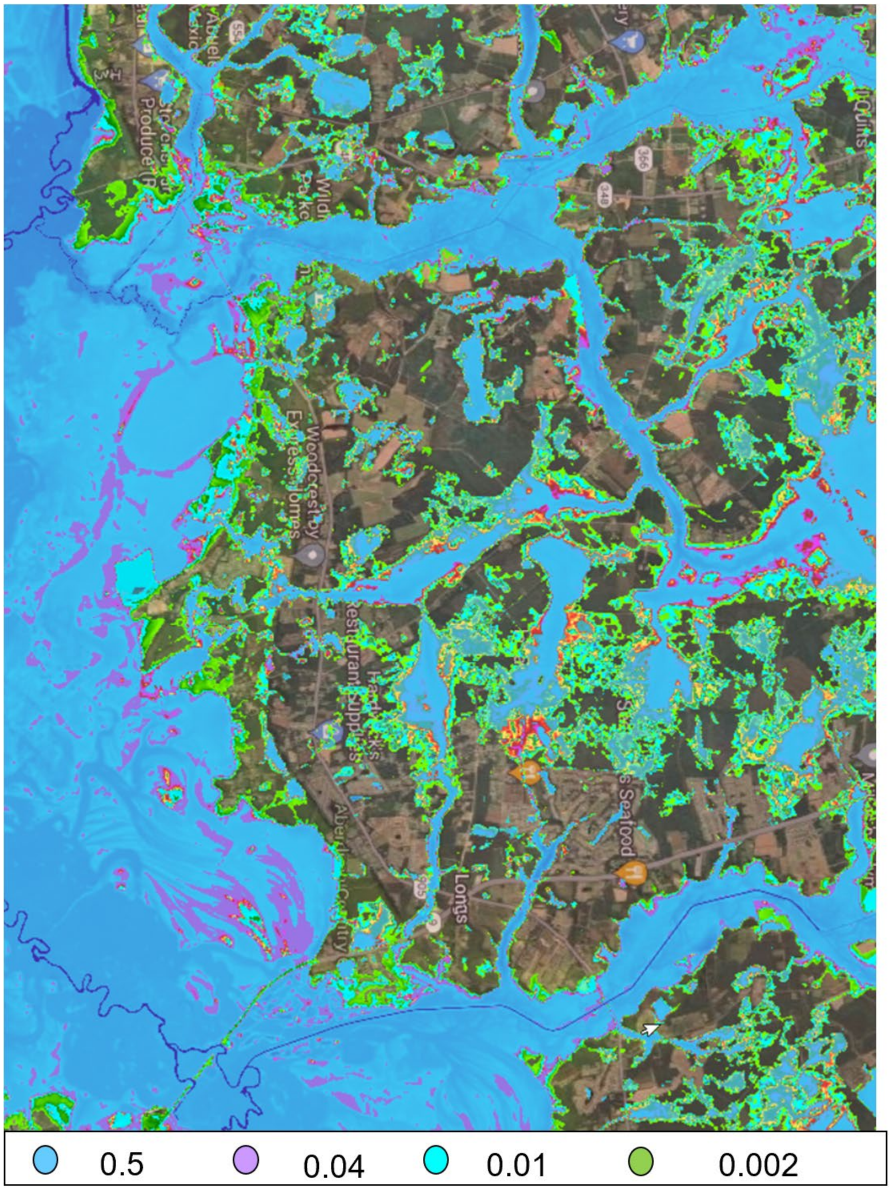


Figure F-17. Longs/ Red Bluff Existing Conditions

F.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 Changing Conditions Appendix A-2 for the assessment of the compound flooding considerations that concluded that there are some coastal and tidal impacts to the riverine flow, but 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. A sensitivity analysis was performed for all SLC scenarios, and it was determined that the magnitude of SLC will not affect the performance of any of the finalized measures. Therefore, an adaptation plan is not emphasized for this study with respect to SLC.

G FUTURE WITHOUT PROJECT CONDITIONS

G.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 areas 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.

G.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 Landing Road Raising in Bucksport and Benching along Socastee Creek in Socastee. The following are the project descriptions and FWOP results.

G.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 F-17** shows the geometric gridding for the FWOP modeling of Big Bull Landing.

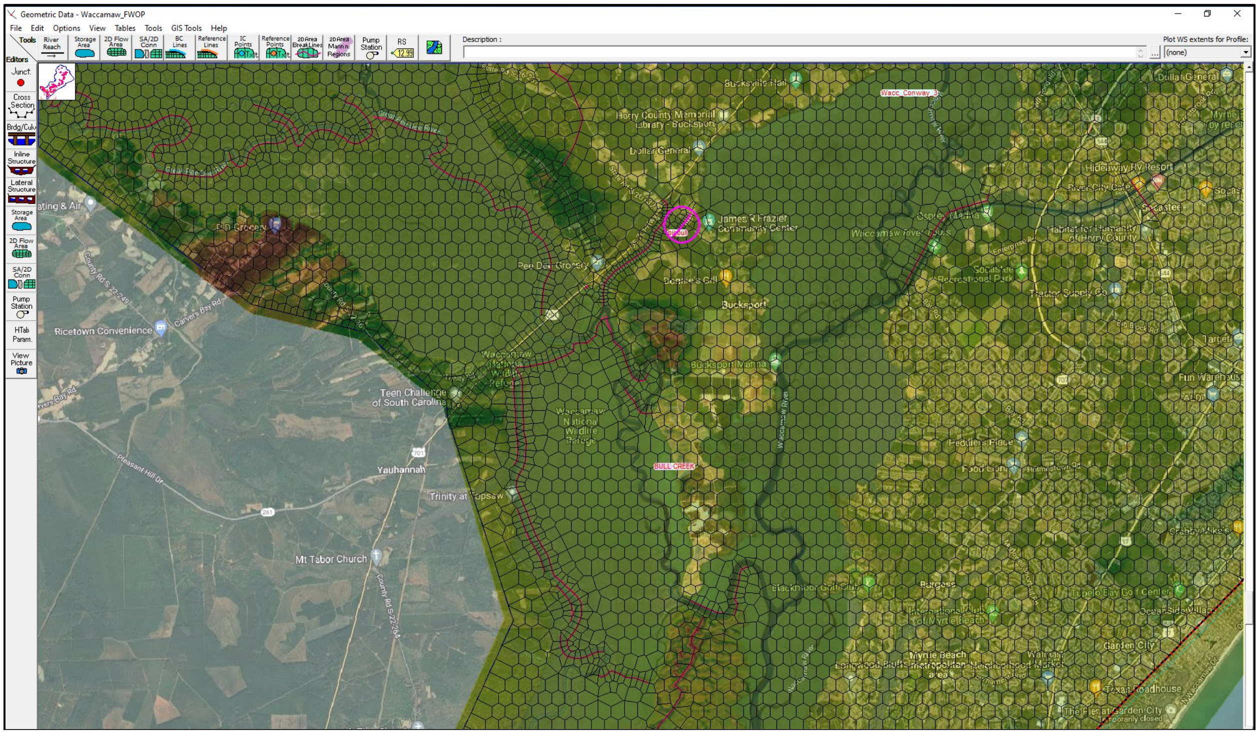
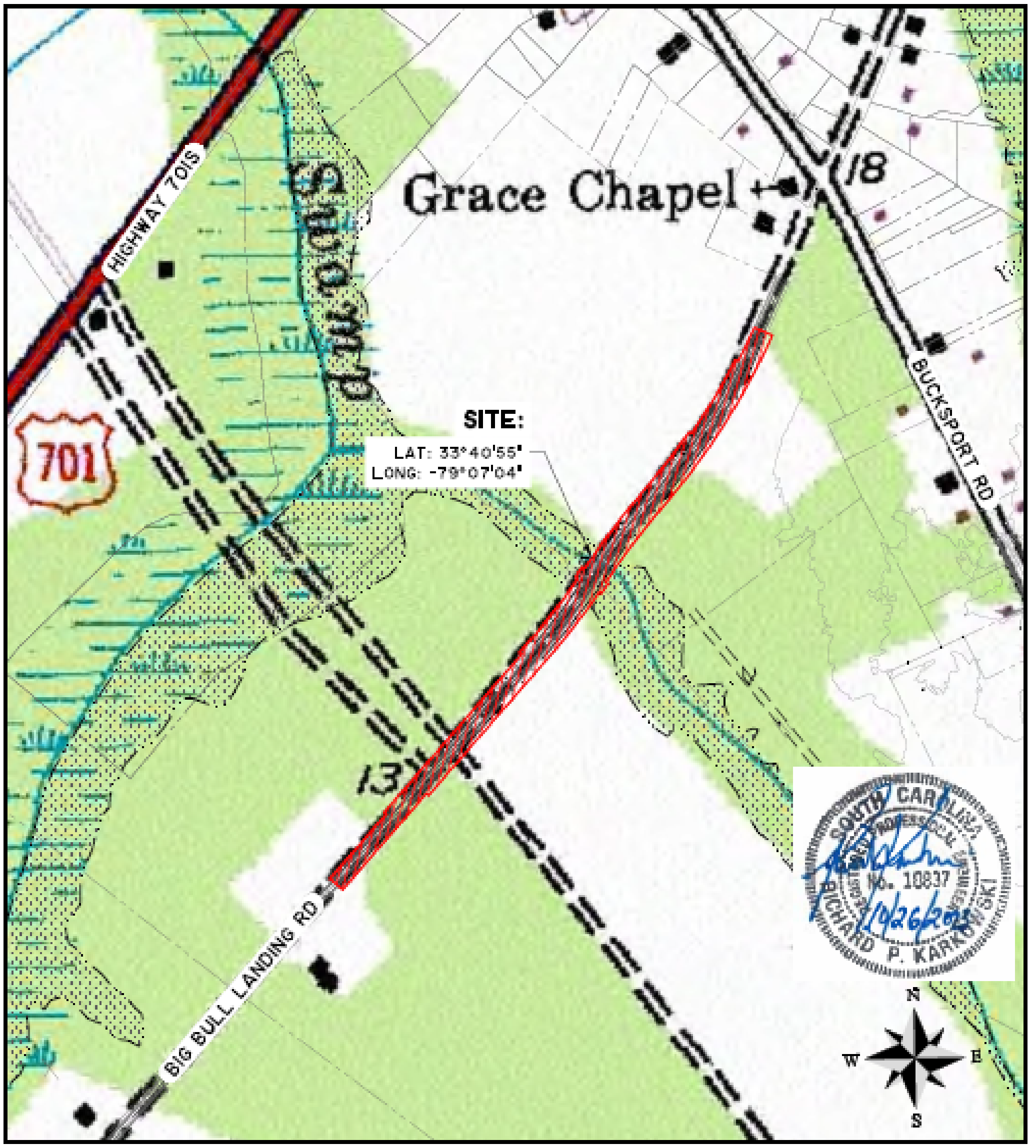


Figure G-1. HEC-RAS model geometric data for the implementation of Big Bull Landing Road Raising



Wetland Impact Exhibit - Location Map

Porject:
Cowford Swamp
CLIENT:
Horry County
LOCATION: South Carolina
DATE: October 26, 2023
JOB NUMBER: J-27636.0002

DRAWN BY: JML
REVIEWED BY: RPK

SHEET: 1 of 4
SCALE: 1" = 500'

THOMAS & HUTTON
611 Burroughs & Chapin Blvd. • Suite 202
Myrtle Beach, SC 29577 • 843.839.3545
www.thomasandhutton.com

Figure G-2. Location of Big Bull Landing Road Raising

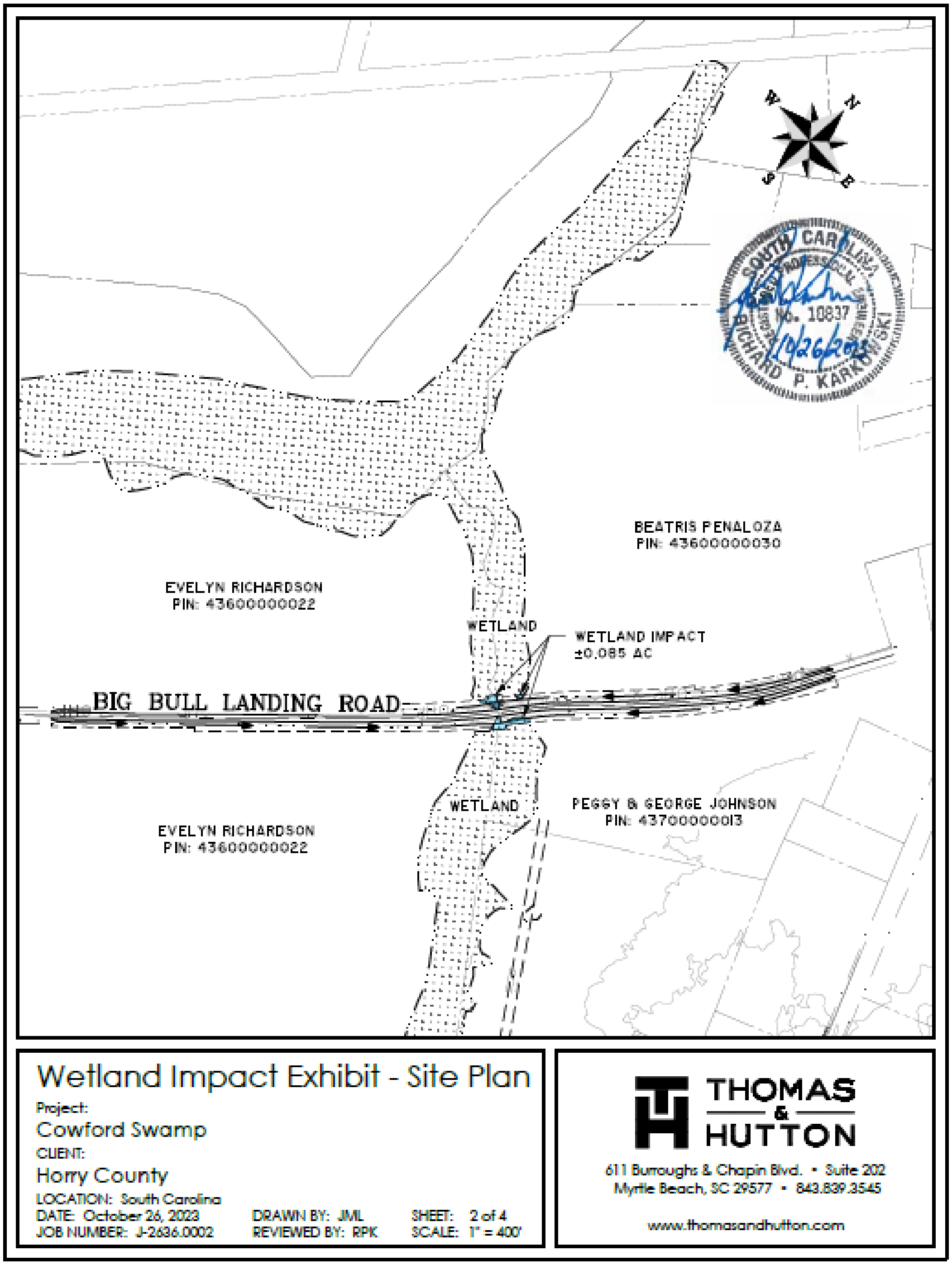


Figure G-3. Wetland Impact Exhibit Big Bull Landing Road Raising

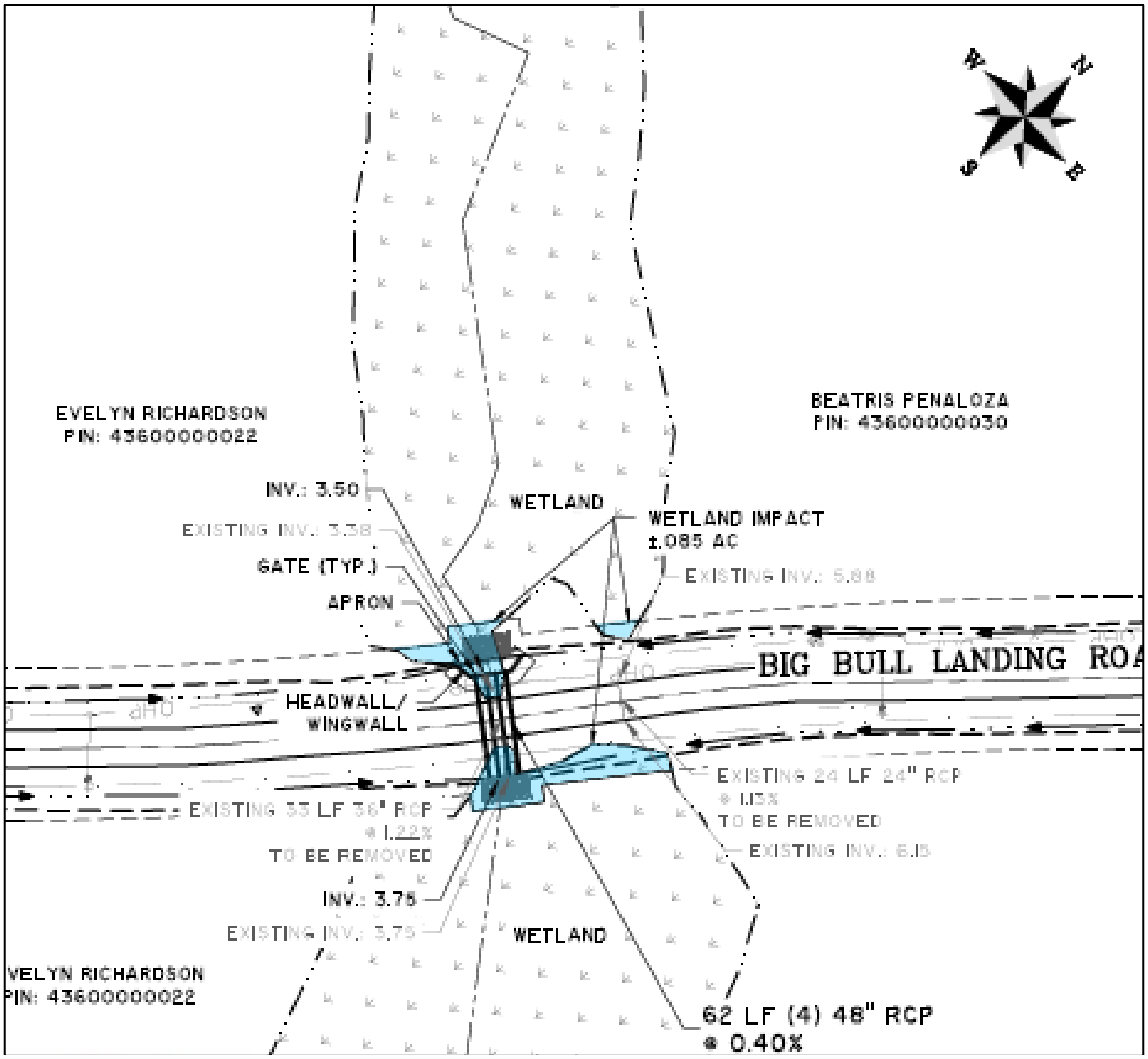


Figure G-4. Site Plans of culverts and wetlands for Big Bull Landing Road Raising

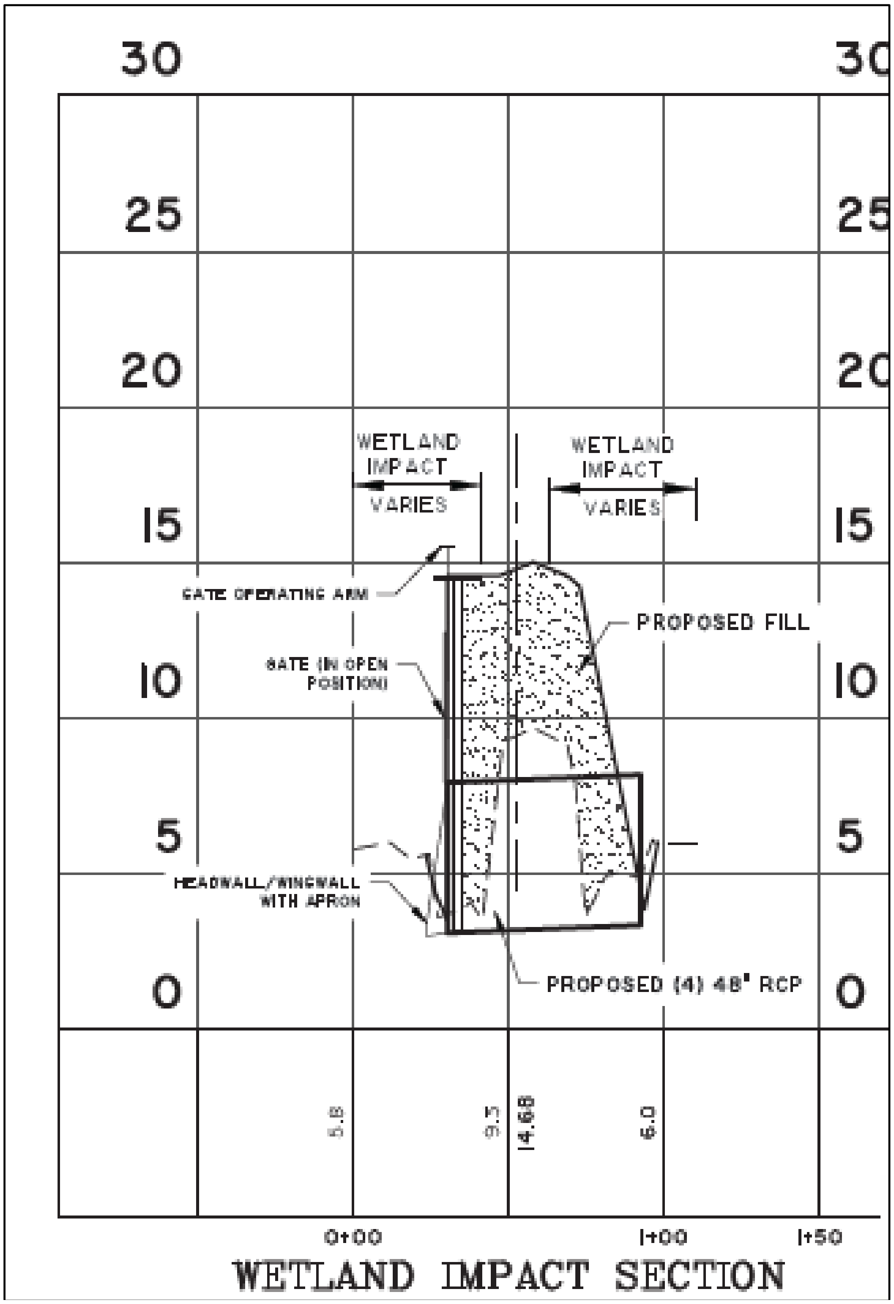
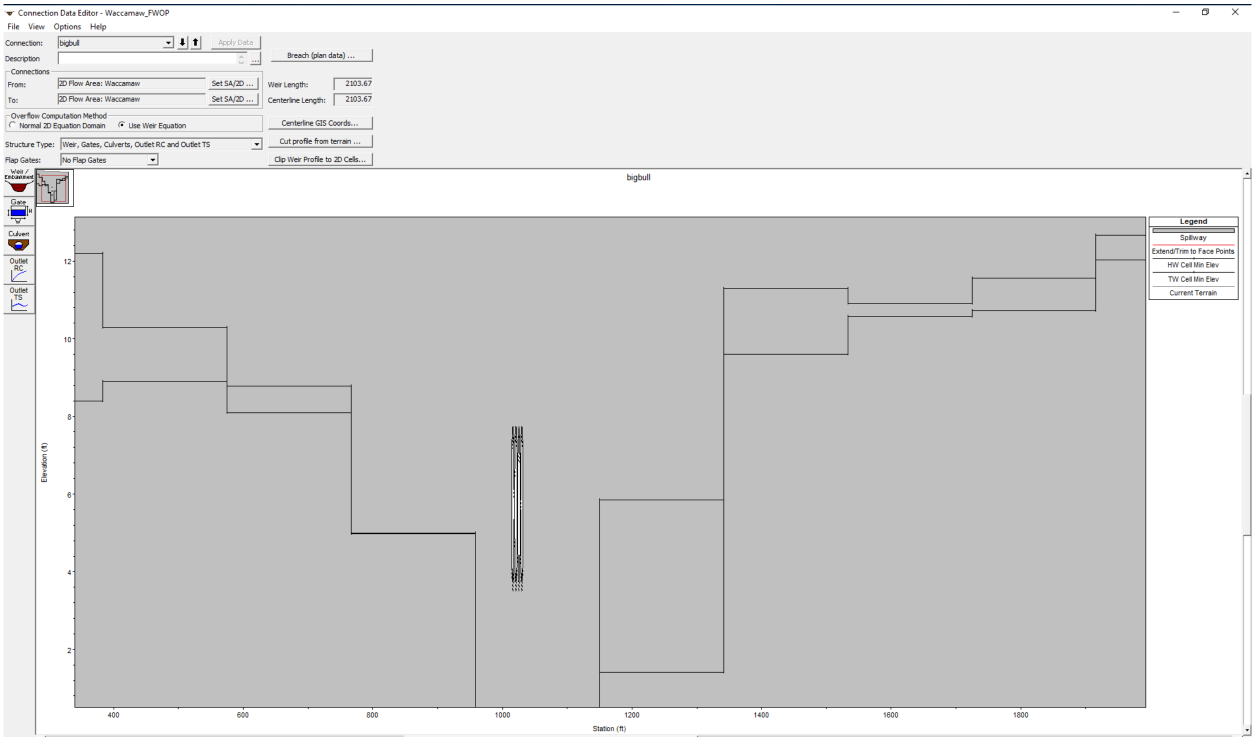


Figure G-5. Proposed Wetland Impact Section cross section for Big Bull Landing Road Raising



Connection Data Editor - Waccamaw_FWOP

Connection: bigbull

Description:

Connections

From: 2D Flow Area: Waccamaw

To: 2D Flow Area: Waccamaw

Weir Length: 2103.67

Centerline Length: 2103.67

Overflow Computation Method

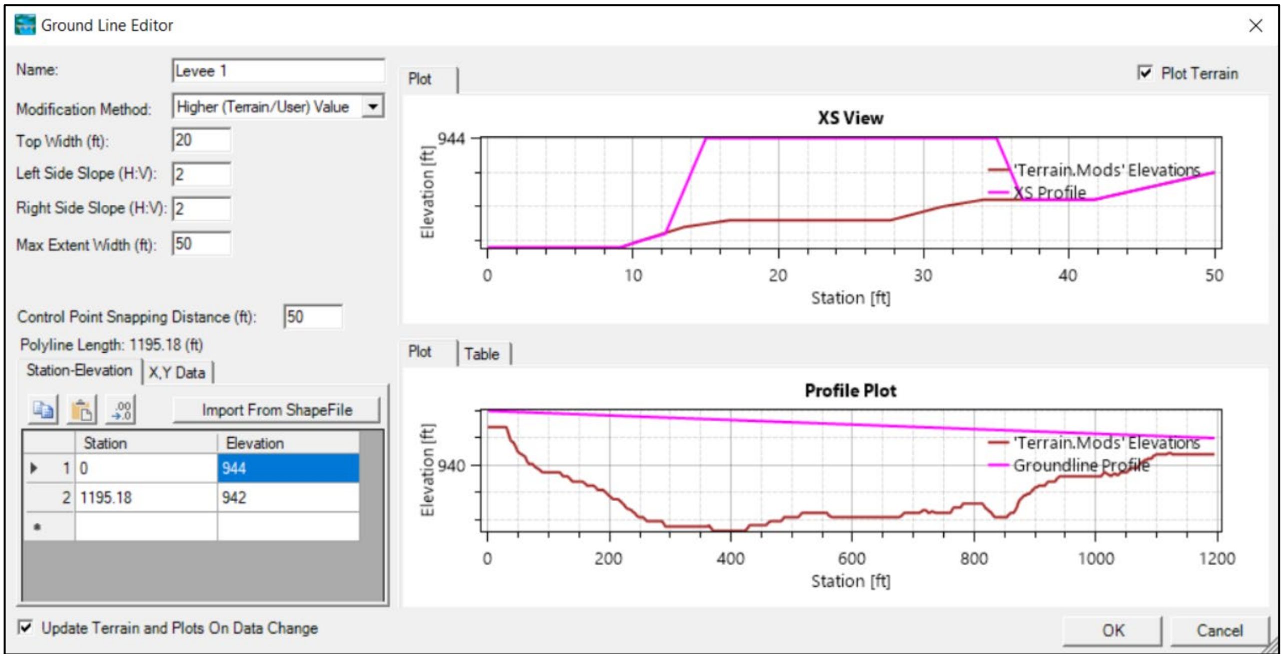
○ Normal 2D Equation Domain ● Use Weir Equation

Structure Type: Weir, Gates, Culverts, Outlet RC and Outlet TS

Flap Gates: No Flap Gates

Figure G-6. 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.



Name: Levee 1

Modification Method: Higher (Terrain/User) Value

Top Width (ft): 20

Left Side Slope (H:V): 2

Right Side Slope (H:V): 2

Max Extent Width (ft): 50

Control Point Snapping Distance (ft): 50

Polyline Length: 1195.18 (ft)

☑ Plot Terrain

	Station	Elevation
1	0	944
2	1195.18	942

☑ Update Terrain and Plots On Data Change

Figure G-7. Example of the Terrain ground line editor used to raise Big Bull landing

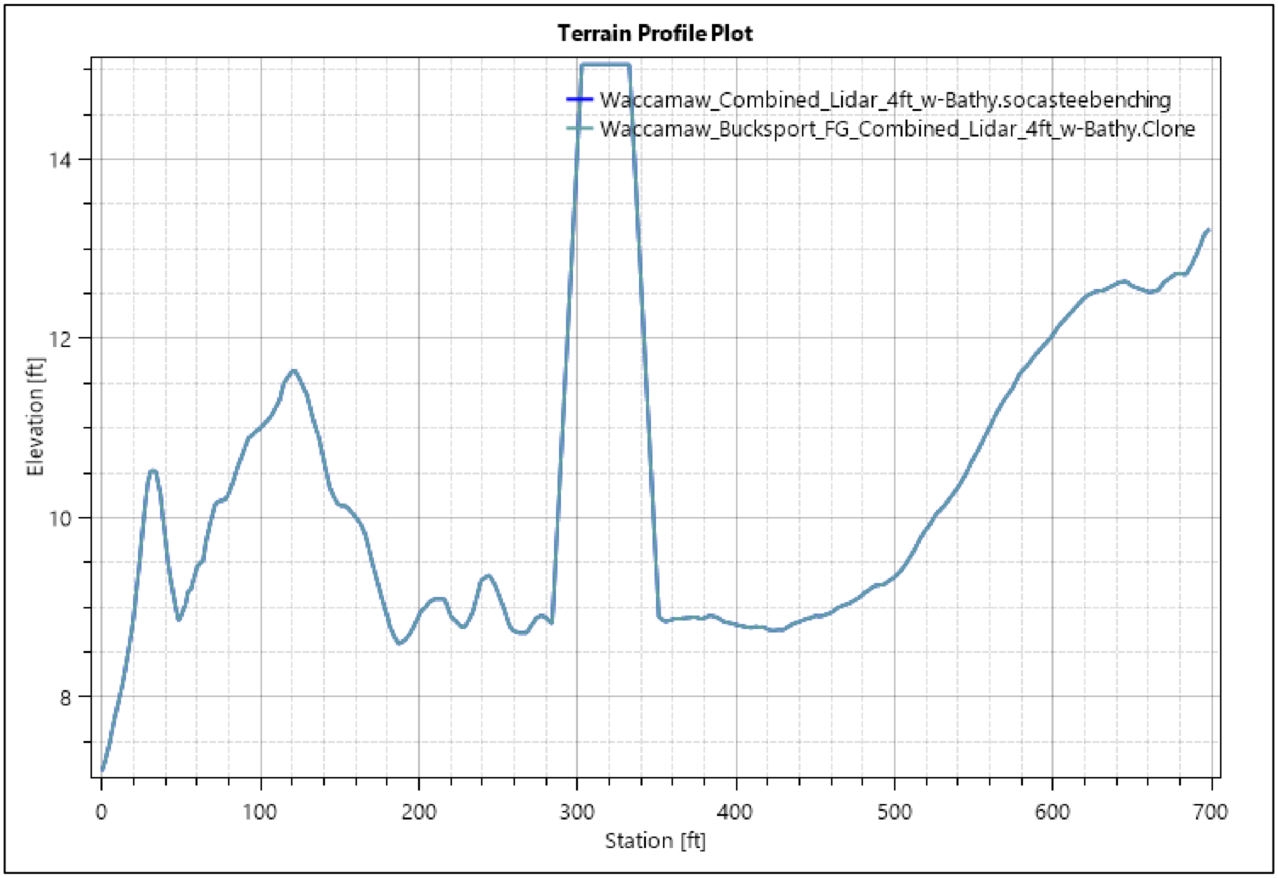


Figure G-8. Imposed terrain for Big Bull Landing Road Raising

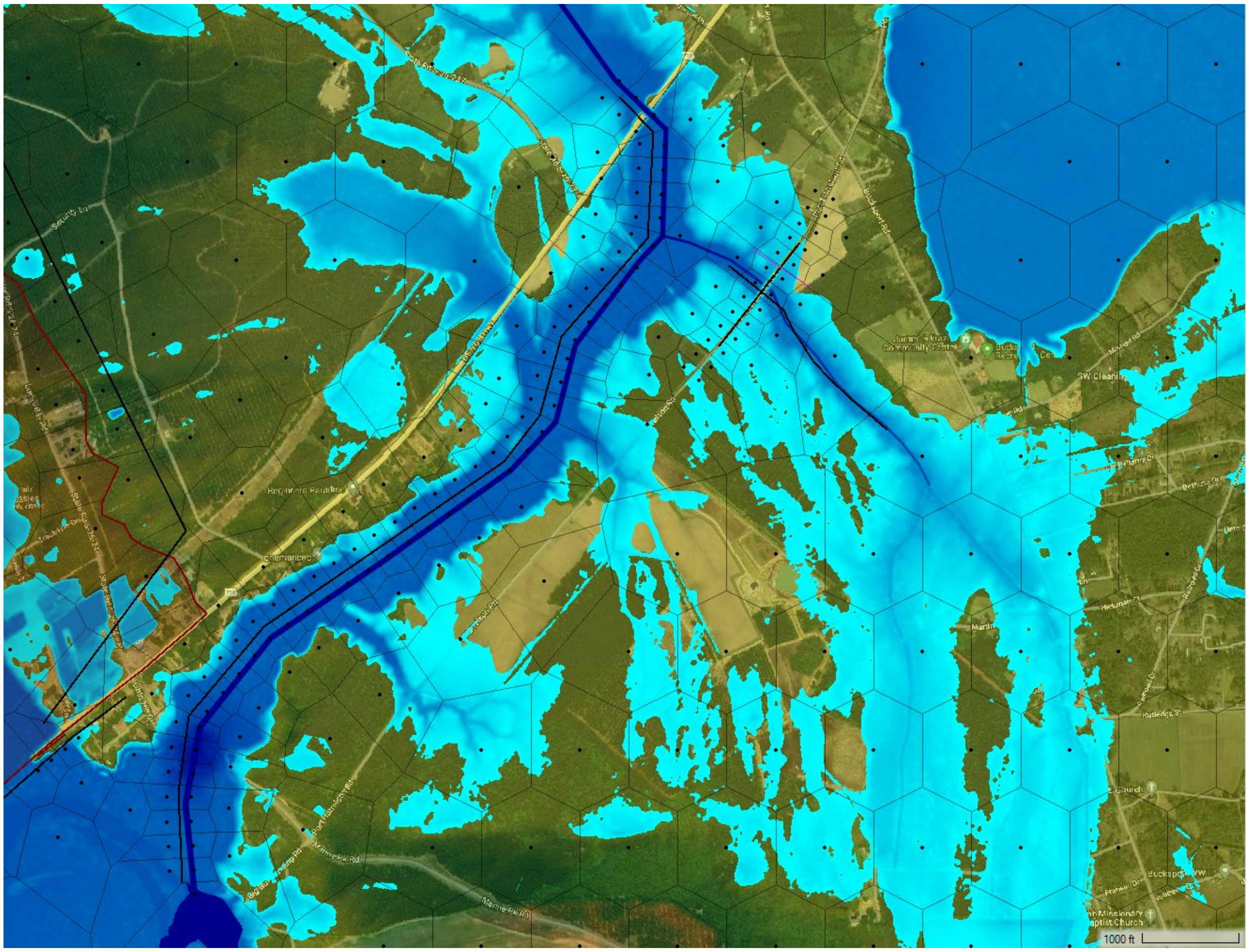
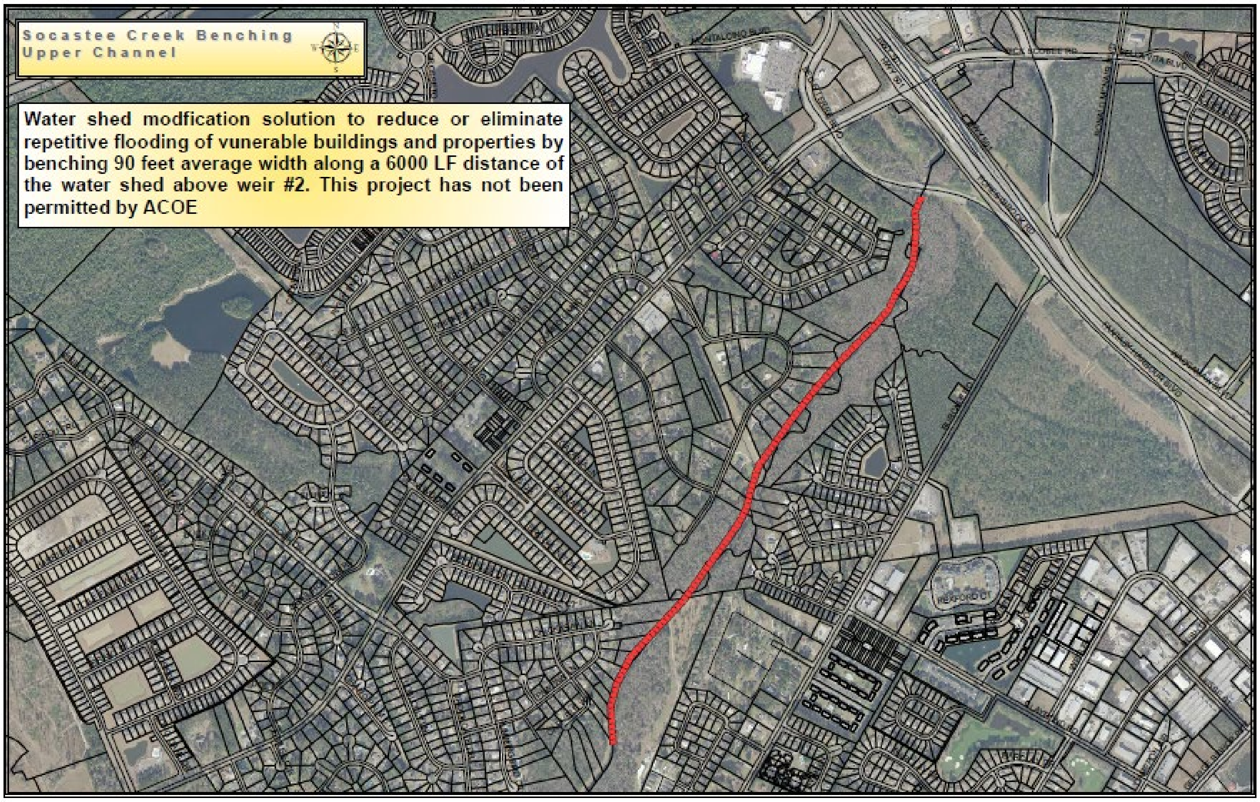


Figure G-9. 1% AEP model Run for FWOP with the implementation of Big Bull Landing

G.2.2 Socastee Benching

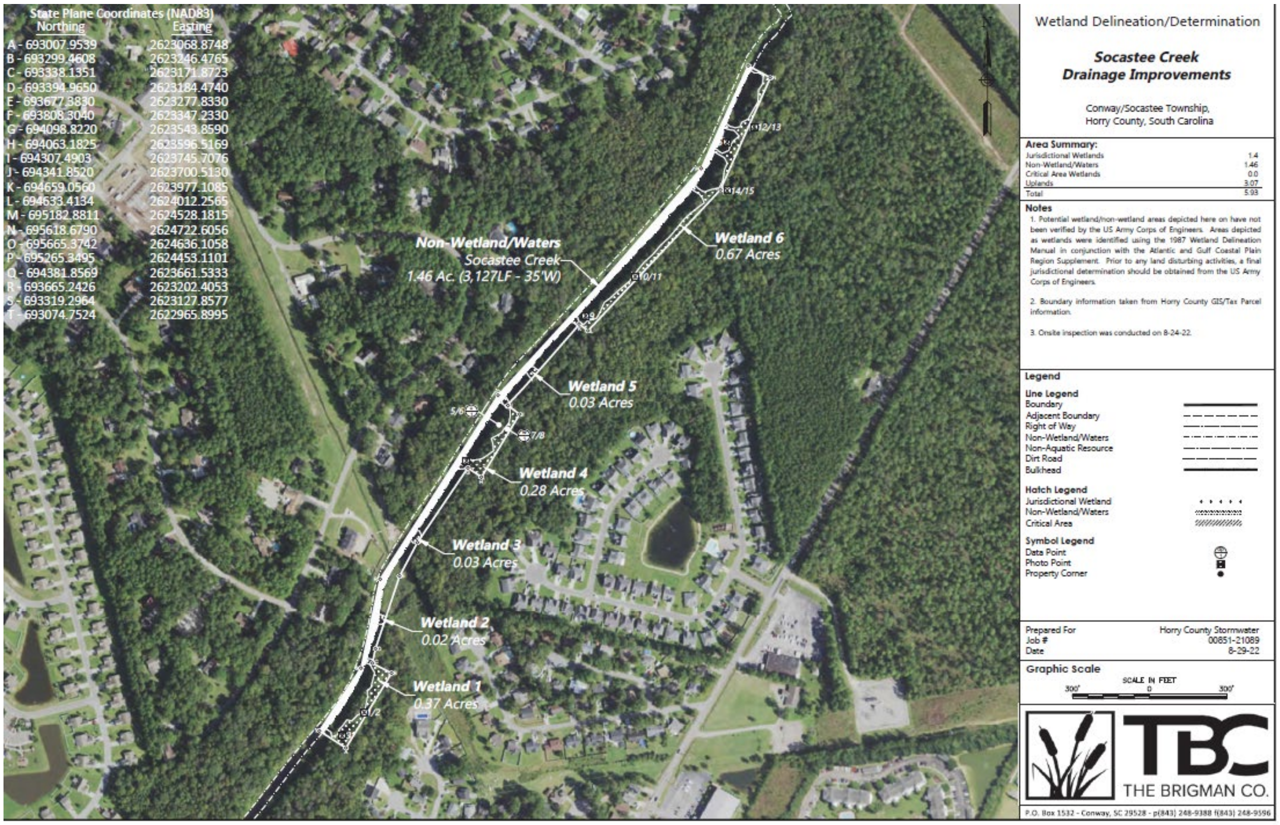
Benching will be implemented by Horry County to reduce or eliminate repetitive flooding of vulnerable buildings and properties by benching 90 ft average width along 6000LF of the watershed above weir #2. In addition, the summary table, HEC-RAS model results are conducted for all events. The results show 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 G-10** through **Figure G-14** show the geometric and terrain alterations to the models that implemented the benching occurring in Socastee and **Figure G-15** shows the WSE comparison along the length of Socastee Creek.


Socastee Creek Benching
Upper Channel

Water shed modfication solution to reduce or eliminate repetitive flooding of vunerable buildings and properties by benching 90 feet average width along a 6000 LF distance of the water shed above weir #2. This project has not been permitted by ACOE

Figure G-10. Location of Socastee Benching

Future Without Project Conditions helps floodwaters recede more quickly and can potentially reduce disruptions to access and transportation. However, benching along the Waccamaw River was not feasible due to its designation in the Nationwide Rivers Inventory, which limits alterations to the landscape.



State Plane Coordinates (NAD83)

	Northing	Easting
A	693007.9539	2623068.8748
B	693299.4608	2623246.4765
C	693338.1351	2623171.8723
D	693394.9650	2623184.4740
E	693677.3830	2623277.8330
F	693808.3040	2623347.2330
G	694098.8220	2623543.8590
H	694063.1825	2623596.5169
I	694307.4903	2623745.7076
J	694341.8520	2623700.5130
K	694659.0560	2623977.1085
L	694633.4134	2624012.2565
M	695182.8811	2624528.1815
N	695618.6790	2624722.6056
O	695665.3742	2624636.1058
P	695265.3495	2624453.1101
Q	694381.8569	2623661.5333
R	693665.2426	2623202.4053
S	693319.2964	2623127.8577
T	693074.7524	2622965.8995

Wetland Delineation/Determination

Socastee Creek
Drainage Improvements

Conway/Socastee Township,
Horry County, South Carolina

Area Summary:

Jurisdictional Wetlands	1.4
Non-Wetland/Waters	1.46
Critical Area Wetlands	0.0
Uplands	3.07
Total	5.93

Notes

1. Potential wetland/non-wetland areas depicted here on have not been verified by the US Army Corps of Engineers. Areas depicted as wetlands were identified using the 1987 Wetland Delineation Manual in conjunction with the Atlantic and Gulf Coastal Plain Region Supplement. Prior to any land disturbing activities, a final jurisdictional determination should be obtained from the US Army Corps of Engineers.

2. Boundary information taken from Horry County GIS/Tax Parcel information.

3. Onsite inspection was conducted on 8-24-22.

Legend

Line Legend

Boundary	▬▬▬
Adjacent Boundary	– – – –
Right of Way	—·——
Non-Wetland/Waters	–·–·–·–
Non-Aquatic Resource	—·——·—
Dirt Road	— — —
Bulkhead	▬▬▬

Hatch Legend

Jurisdictional Wetland	᛫ ᛫ ᛫ ᛫ ᛫
Non-Wetland/Waters	▨
Critical Area	▨

Symbol Legend

Data Point	⊕
Photo Point	■
Property Corner	●

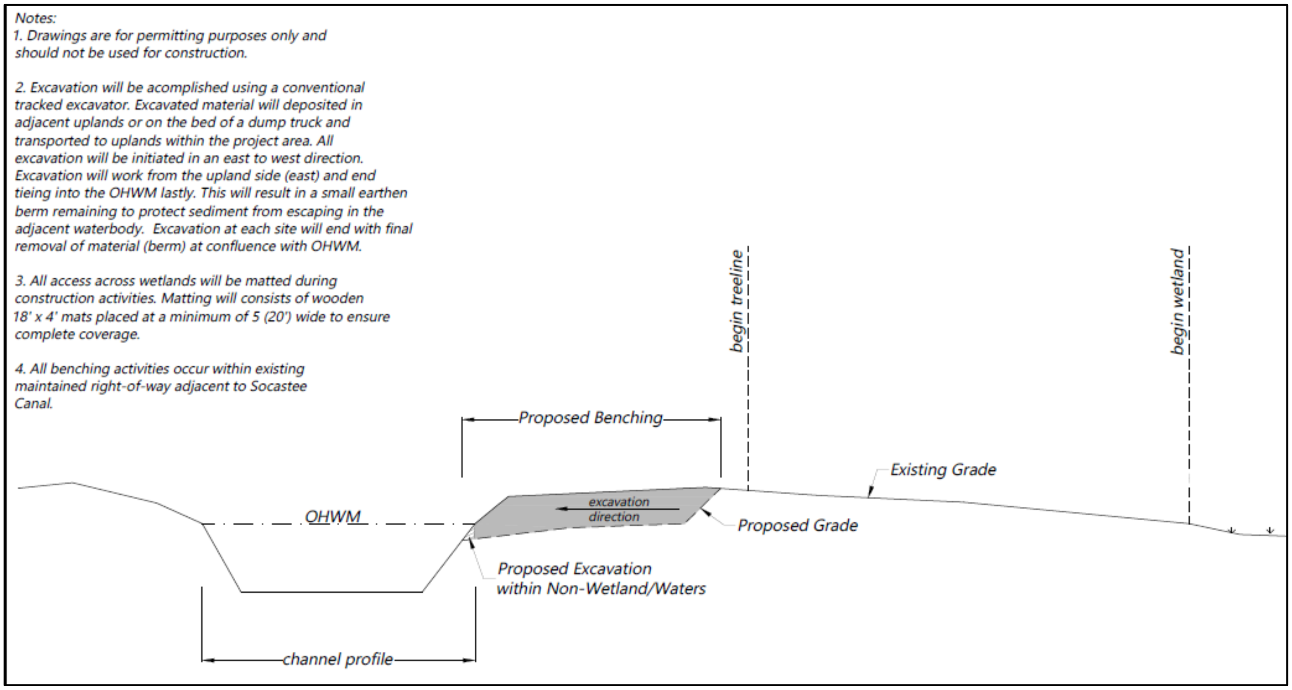
Prepared For	Horry County Stormwater
Job #	00851-21089
Date	8-29-22

Graphic Scale

SCALE IN FEET
300' 0 300'

P.O. Box 1532 - Conway, SC 29528 - p(843) 248-9388 f(843) 248-9596

Figure G-11. Socastee Benching project extents



Notes:

1. Drawings are for permitting purposes only and should not be used for construction.

2. Excavation will be accomplished using a conventional tracked excavator. Excavated material will deposited in adjacent uplands or on the bed of a dump truck and transported to uplands within the project area. All excavation will be initiated in an east to west direction. Excavation will work from the upland side (east) and end tieing into the OHWM lastly. This will result in a small earthen berm remaining to protect sediment from escaping in the adjacent waterbody. Excavation at each site will end with final removal of material (berm) at confluence with OHWM.

3. All access across wetlands will be matted during construction activities. Matting will consists of wooden 18' x 4' mats placed at a minimum of 5 (20') wide to ensure complete coverage.

4. All benching activities occur within existing maintained right-of-way adjacent to Socastee Canal.

Figure G-12. Proposed benching and channel profiles

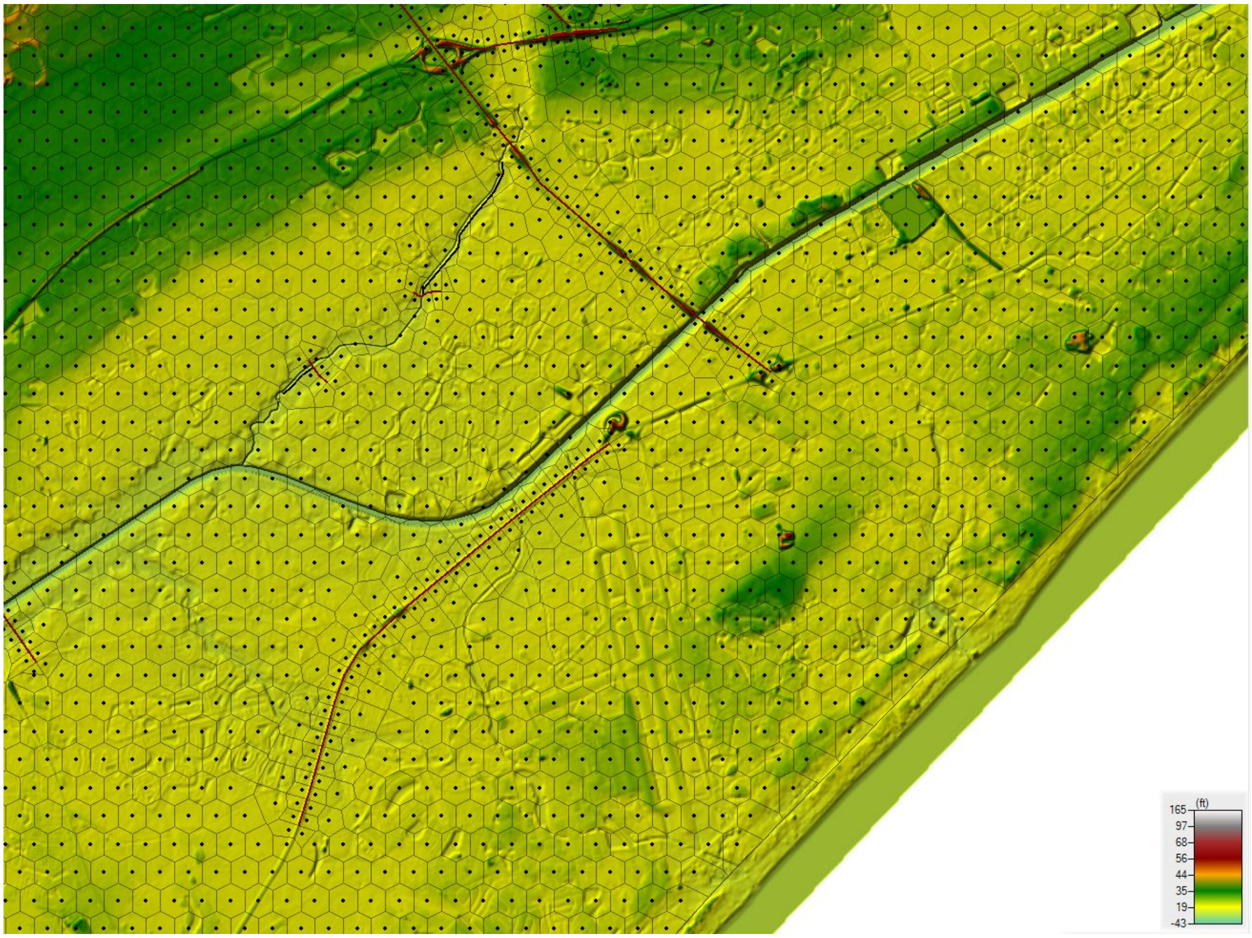


Figure G-13. HEC-RAS Gridding and terrain modification

Figure G-14. Map with proposed benching and 1% AEP event

Figure G-15.1% AEP of the Socastee Benching project

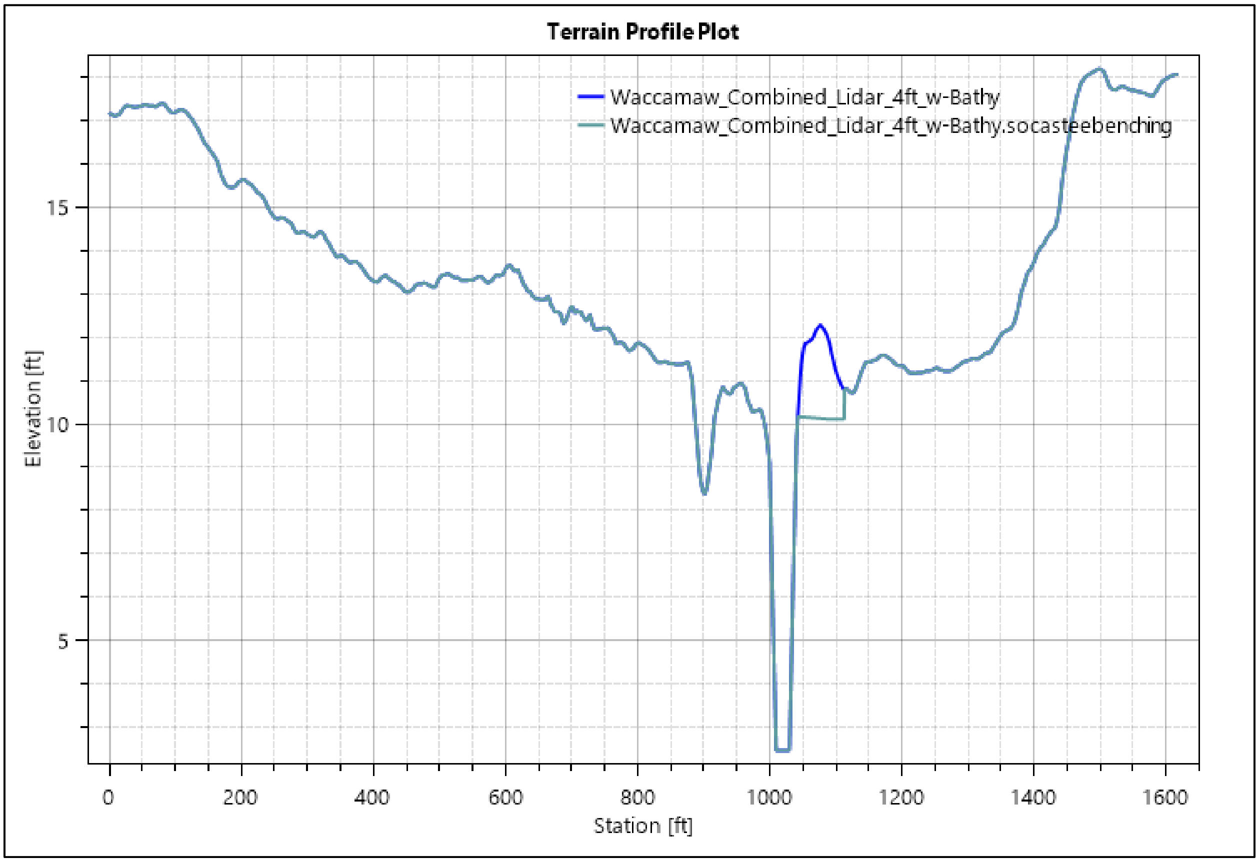


Figure G-16. Modified Terrain showing existing grade in the channel and the original bathymetry of the channel

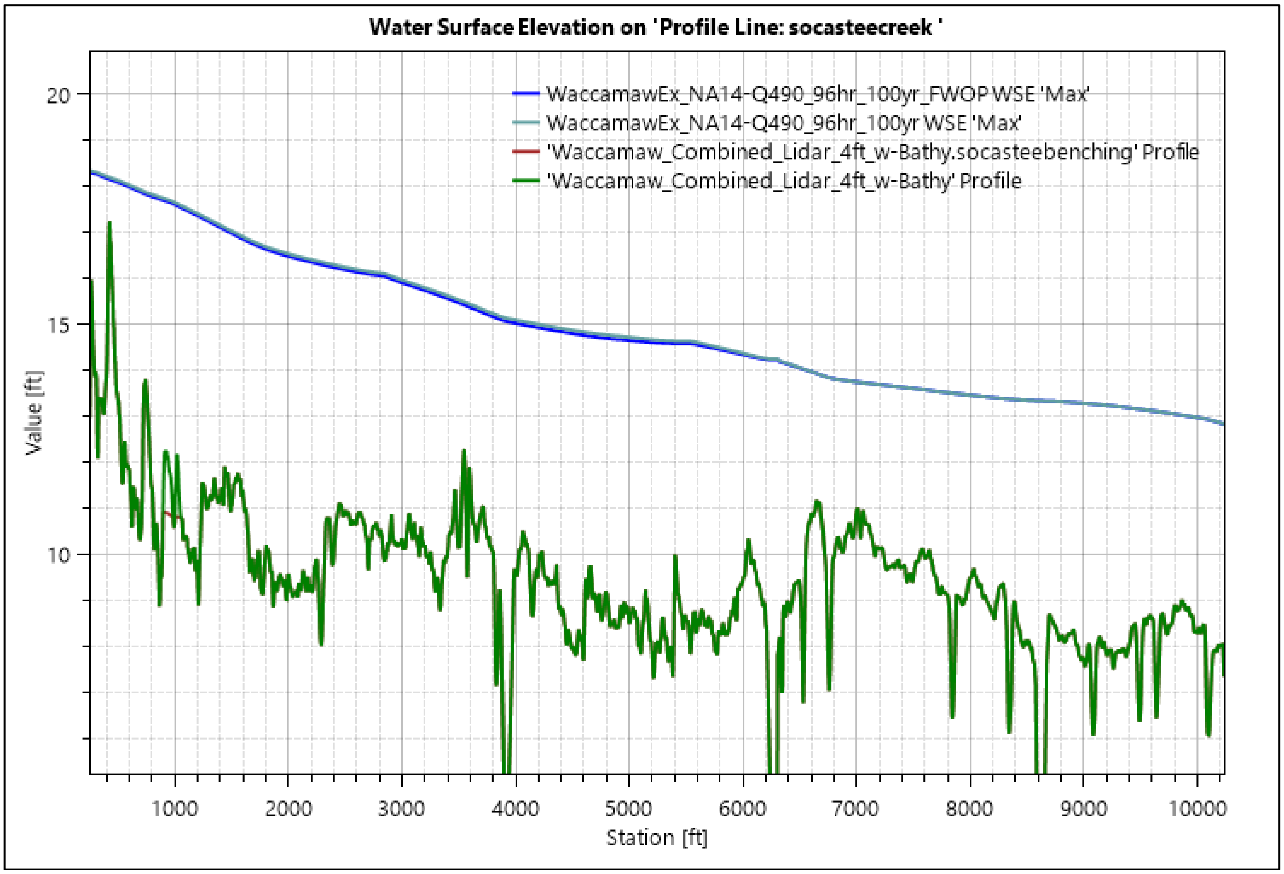


Figure G-17. WSE data comparison for existing and FWOP including Socastee Benching

G.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 areas, FWOP hydraulic simulations are shown in **Figure G-17**. 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.

H 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 ensured that all final alternatives were effective at producing hydraulic benefits with reduced risk and minimal impacts.

H.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.

H.1.1 Engineer Regulation 1165-2-21

ER 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 aforementioned 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.

H.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 not implemented because the Waccamaw is listed on the Nationwide Rivers Inventory under the Wild and Scenic Rivers Act, so alterations of the river main stem may 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.

H.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.

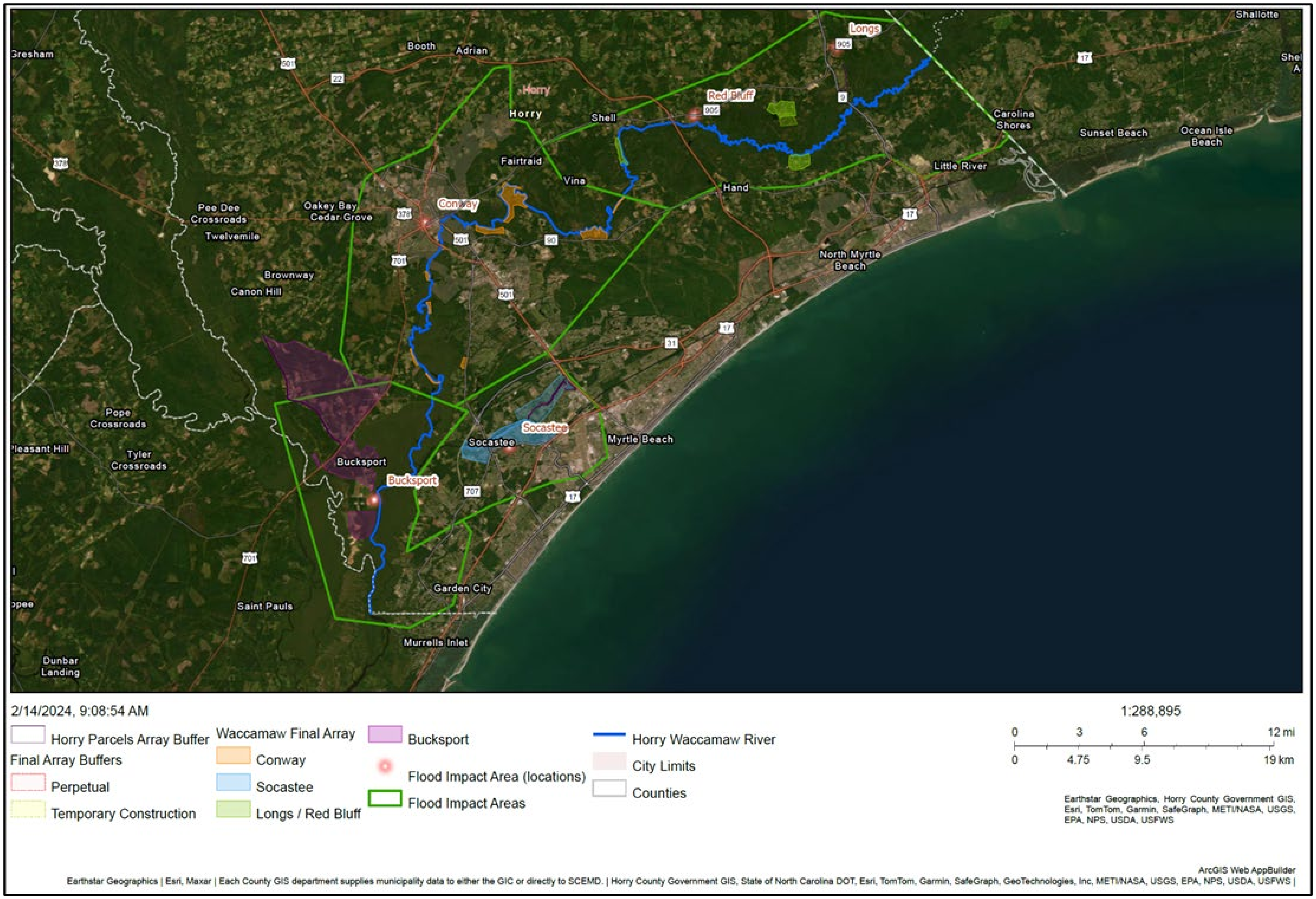


Figure H-1. Waccamaw Final Array of Evaluated Measures

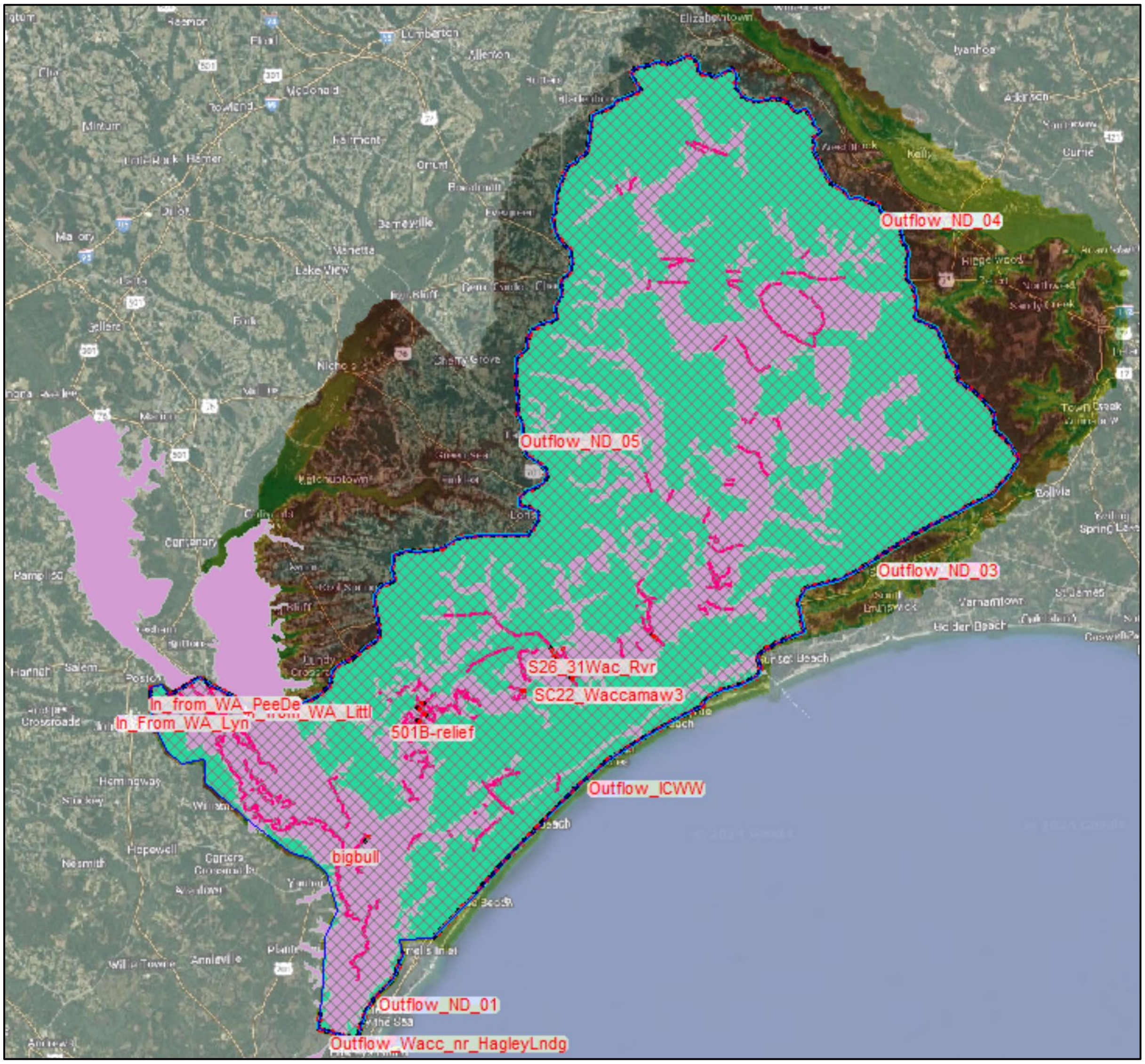


Figure H-2. HEC-RAS model showing array of evaluated measures

H.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 H-1 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 H-3** maps the evaluated measures.

Table H-1. 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

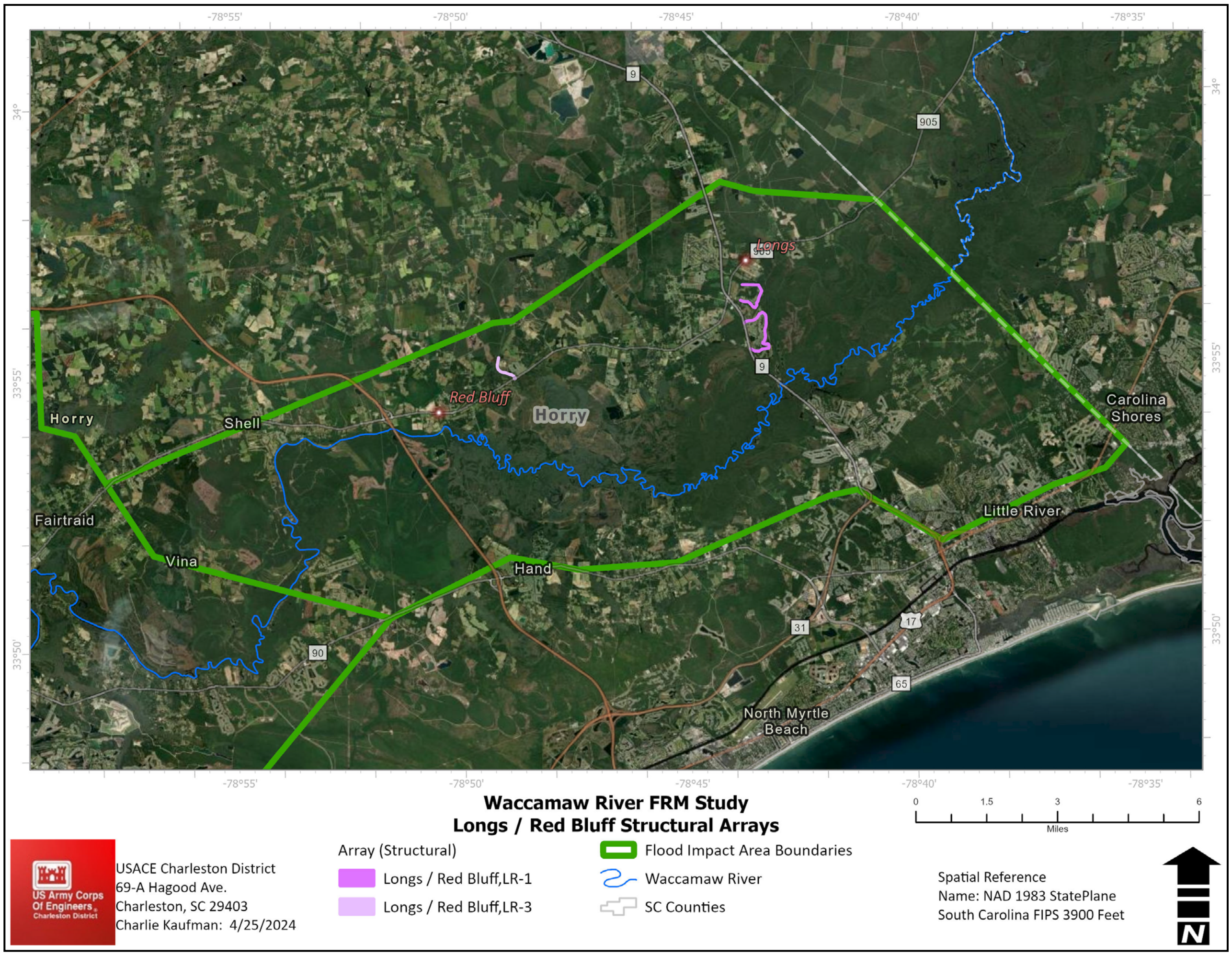


Figure H-3. Longs/Red Buff Evaluated Measures

H.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 weather-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.

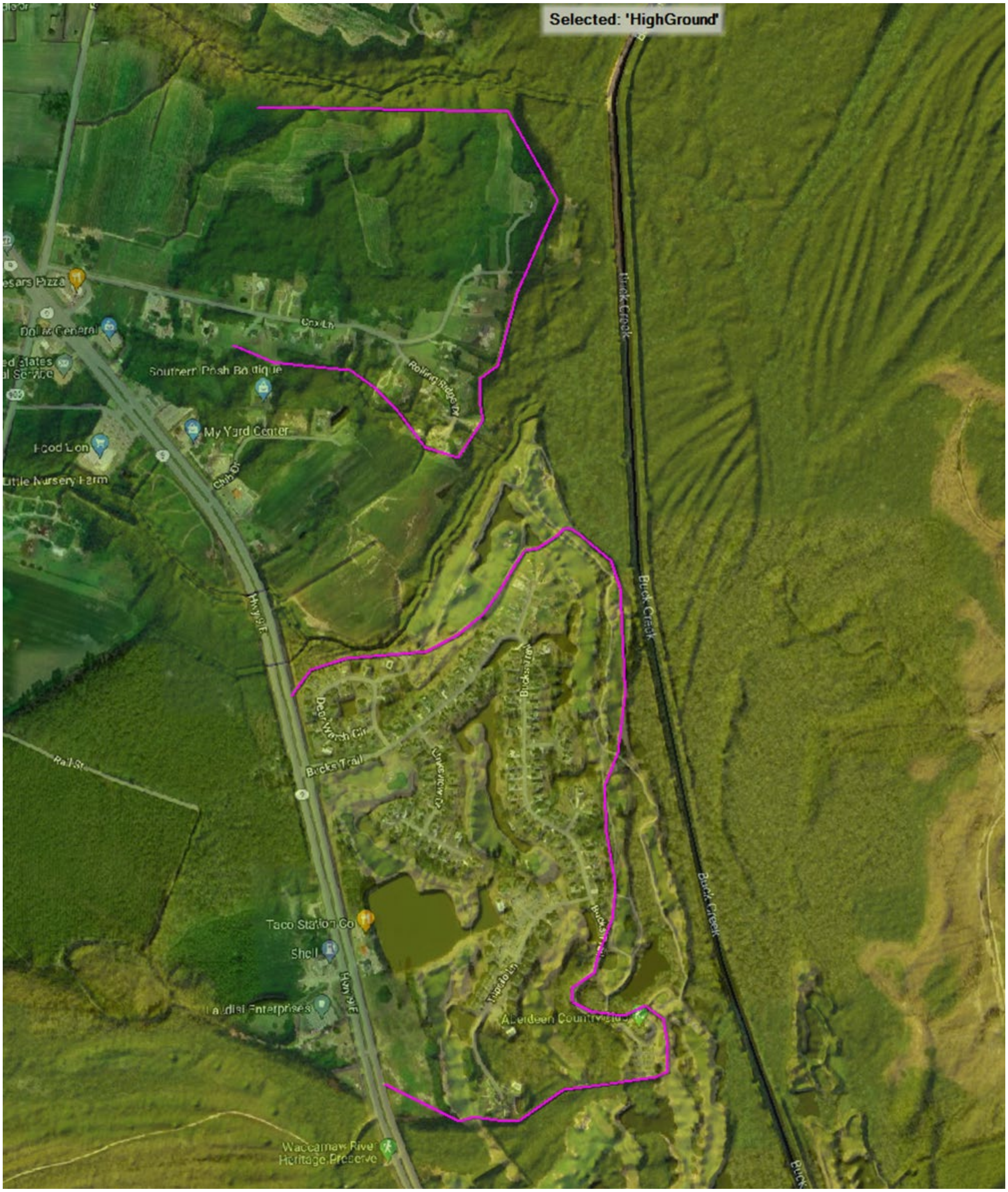


Figure H-4. Floodwall (in pink) in Longs along Buck Creek

Figure H-5 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.

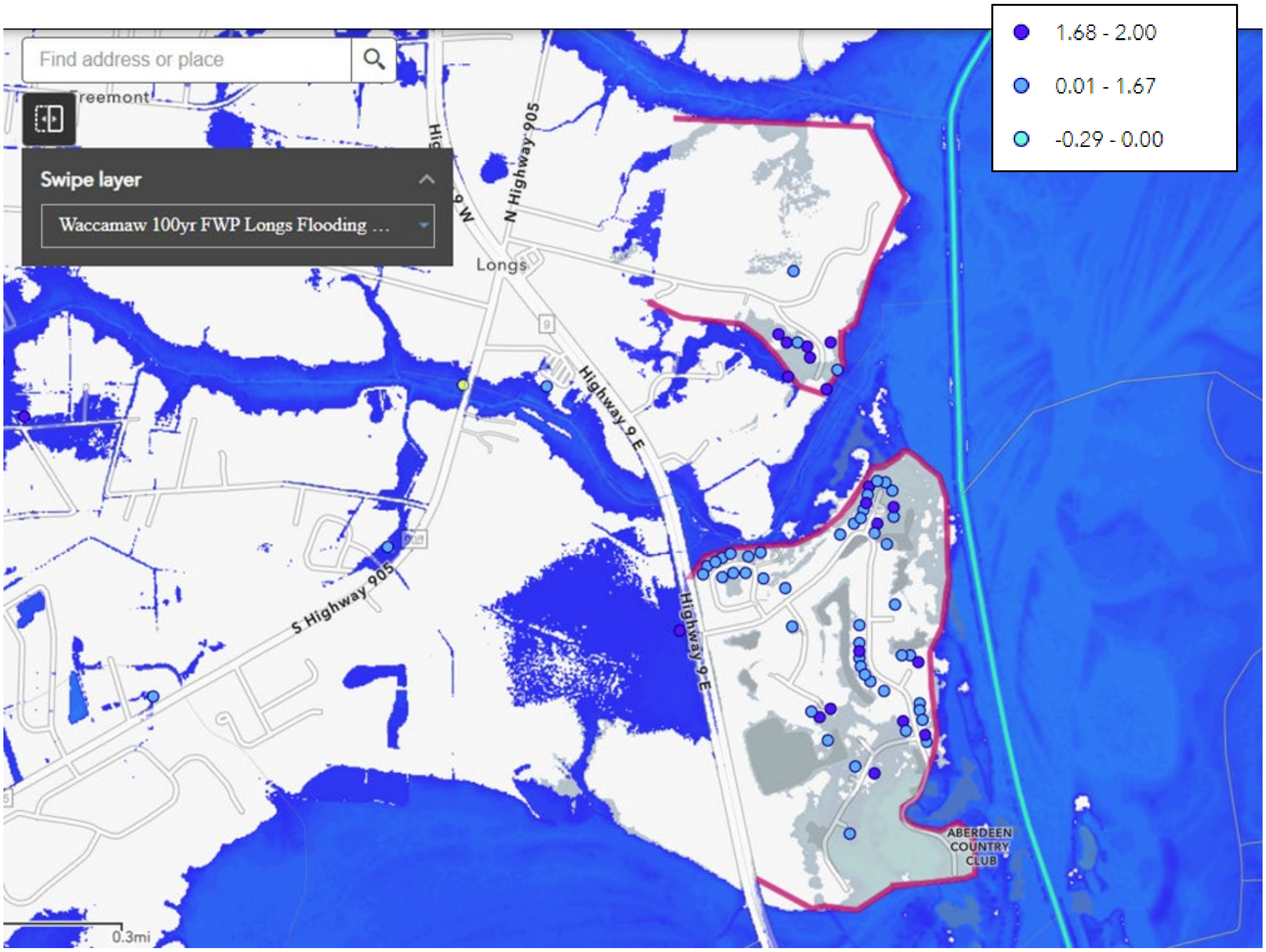


Figure H-5. FWP (Blue) and FWOP (grey) modeled Floodwall in Longs for 1% AEP. Structures are indicated with the dots with varying depths of protection

H.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 would 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.

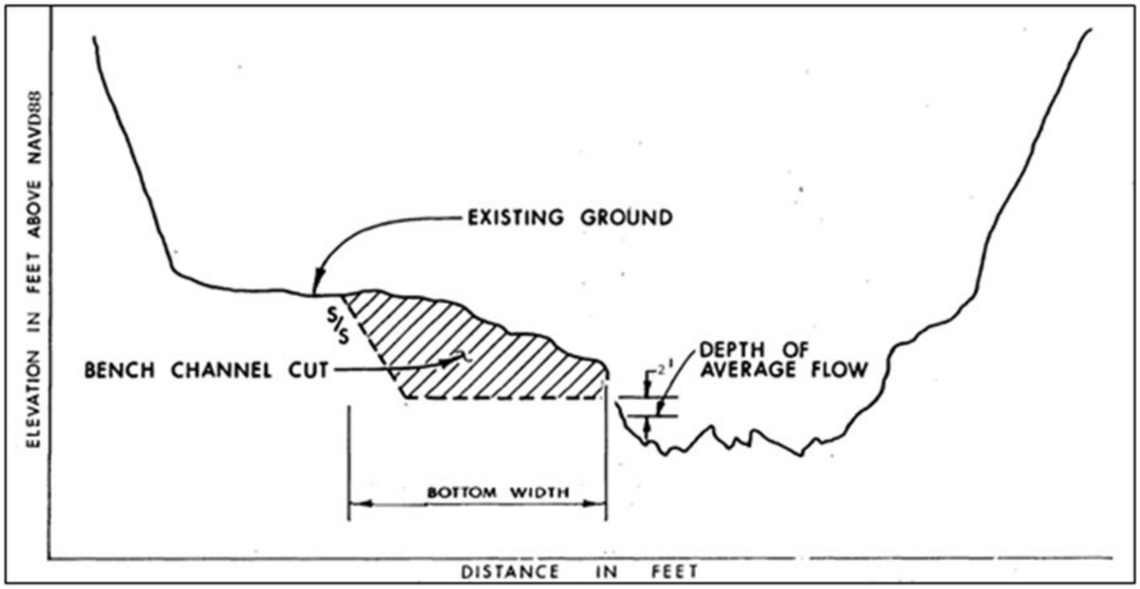


Figure H-6. Typical Benching cross section

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.

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- 200ft width with a 1:1 slope, with a total cutoff 714,373 cu yd.

Figure H-7. 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.

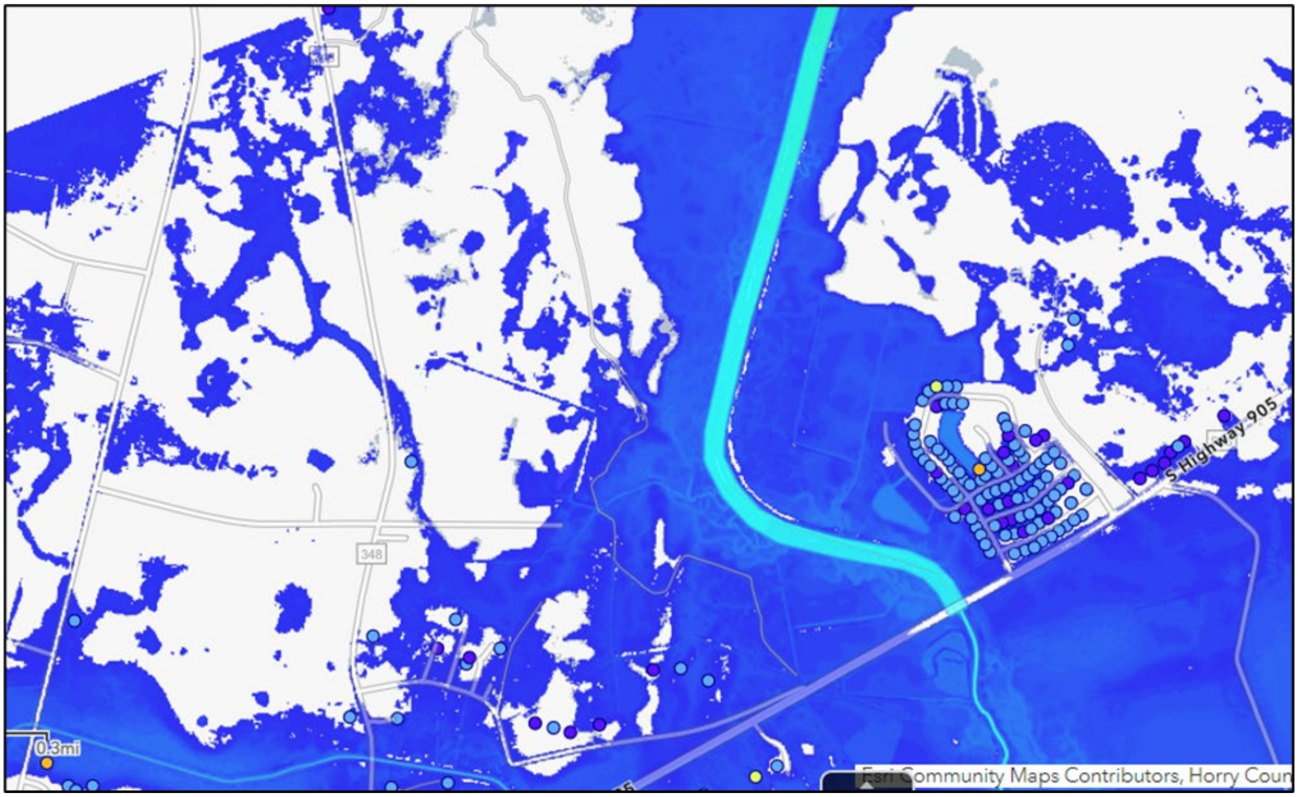


Figure H-8. Depths for FWP 1% AEP for Benching in Simpson Creek

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.

H.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 benefit cost ration (BCR).

H.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 H-9** shows the outline of the focus area on Conway and the retained and evaluated structural measures.

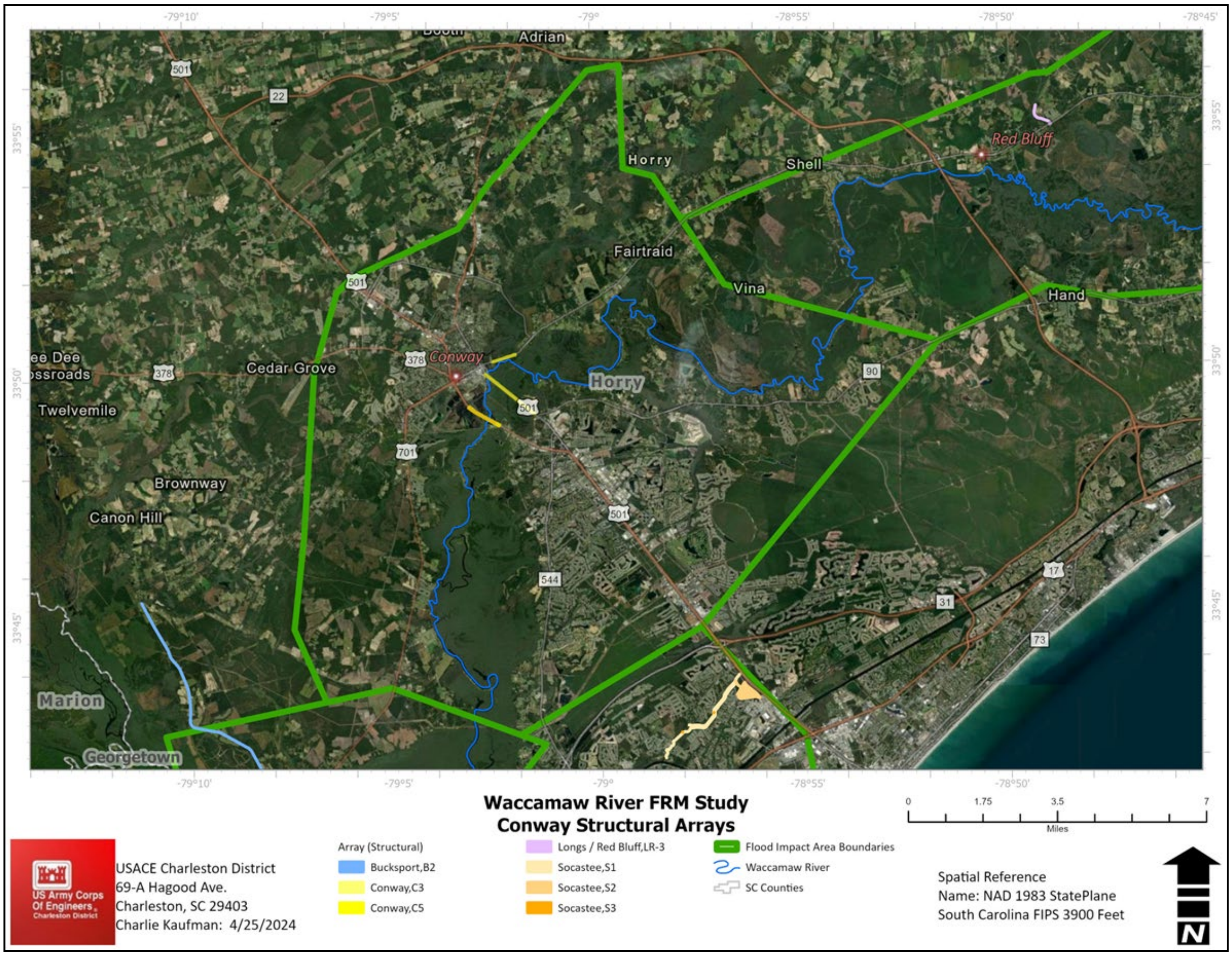


Figure H-9. Conway Structural Arrays

Table H-2 shows retained and screened measures for Conway. Floodwalls and a 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 to Lake Busbee. Relief Bridges were retained since they would not impact the Nationwide Rivers Inventory status/potential wild and scenic inclusion of the Waccamaw and would allow for the flow to convey somewhat naturally without overtopping the road.

Table H-2. 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

Conway	Screening Rationale
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 improvements 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. Relief bridges have proven successful in reducing flood durations on the lower Roanoke River following Hurricane Floyd. Key lessons learned from this experience include the effectiveness of additional "dry" bridges or culverts positioned some distance from the main bridge within the floodway. These structures helped decrease flood durations and provided improved wildlife passage.

In terms of hydraulic performance, box culverts were found to be significantly better than reinforced concrete pipes (RCPs) for managing flow, maintaining ease of maintenance, and facilitating wildlife movement, particularly for larger species like bears. These findings highlight the importance of considering both hydraulic efficiency and ecological factors in the design of relief bridges.

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.

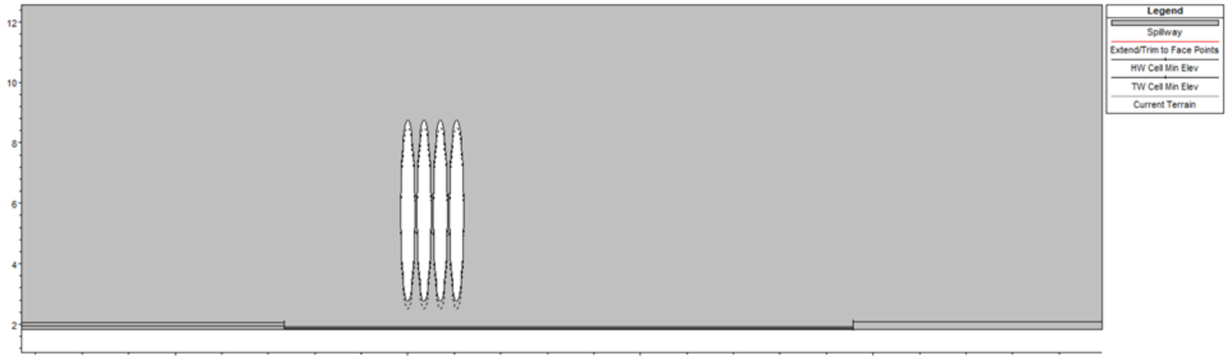


Figure H-10. Geometry in HEC RAS Cross section of the relief bridge for 501 B

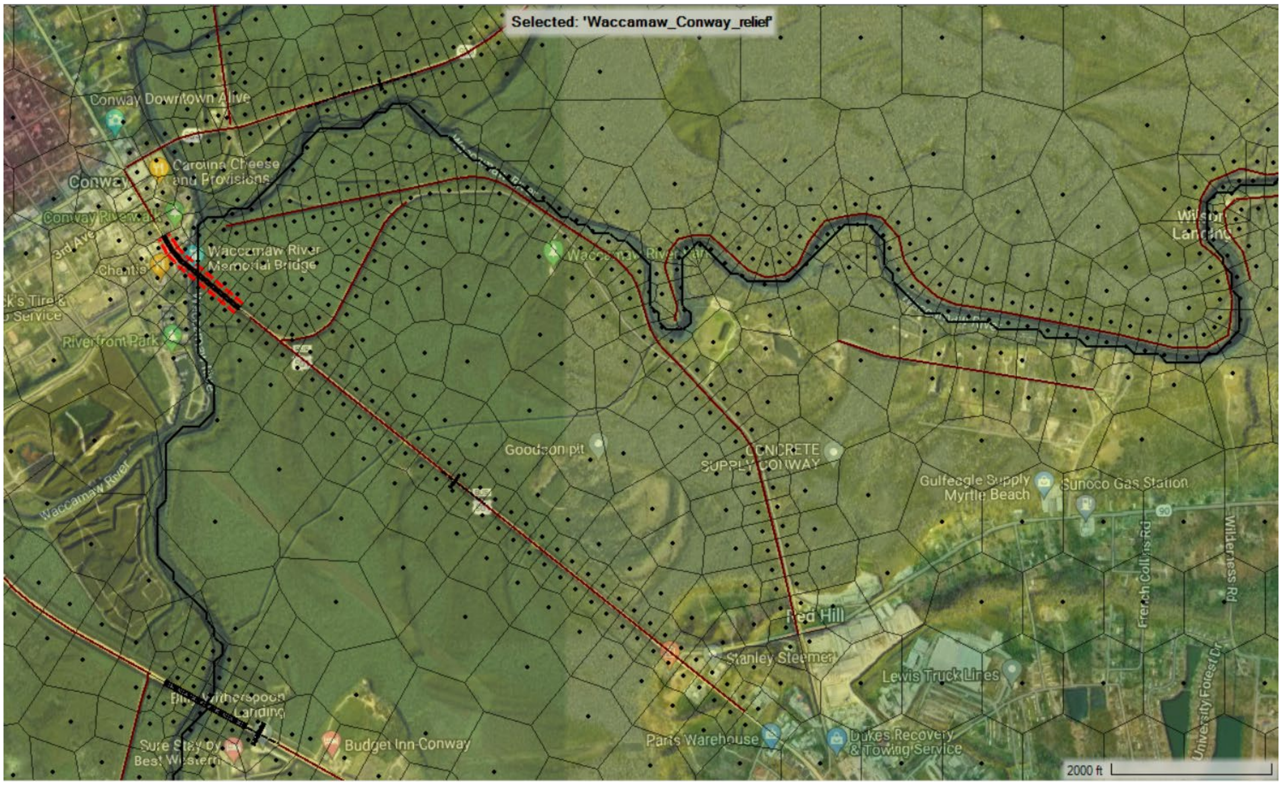


Figure H-11. Three locations of the relief bridges in Conway; 905, 501B, and 501

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 H-12 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 H-13**, with the location of the cross section in **Figure H-13**, there is a reduction in Water Surface Elevation of 1.08 ft downstream of Highway 501.

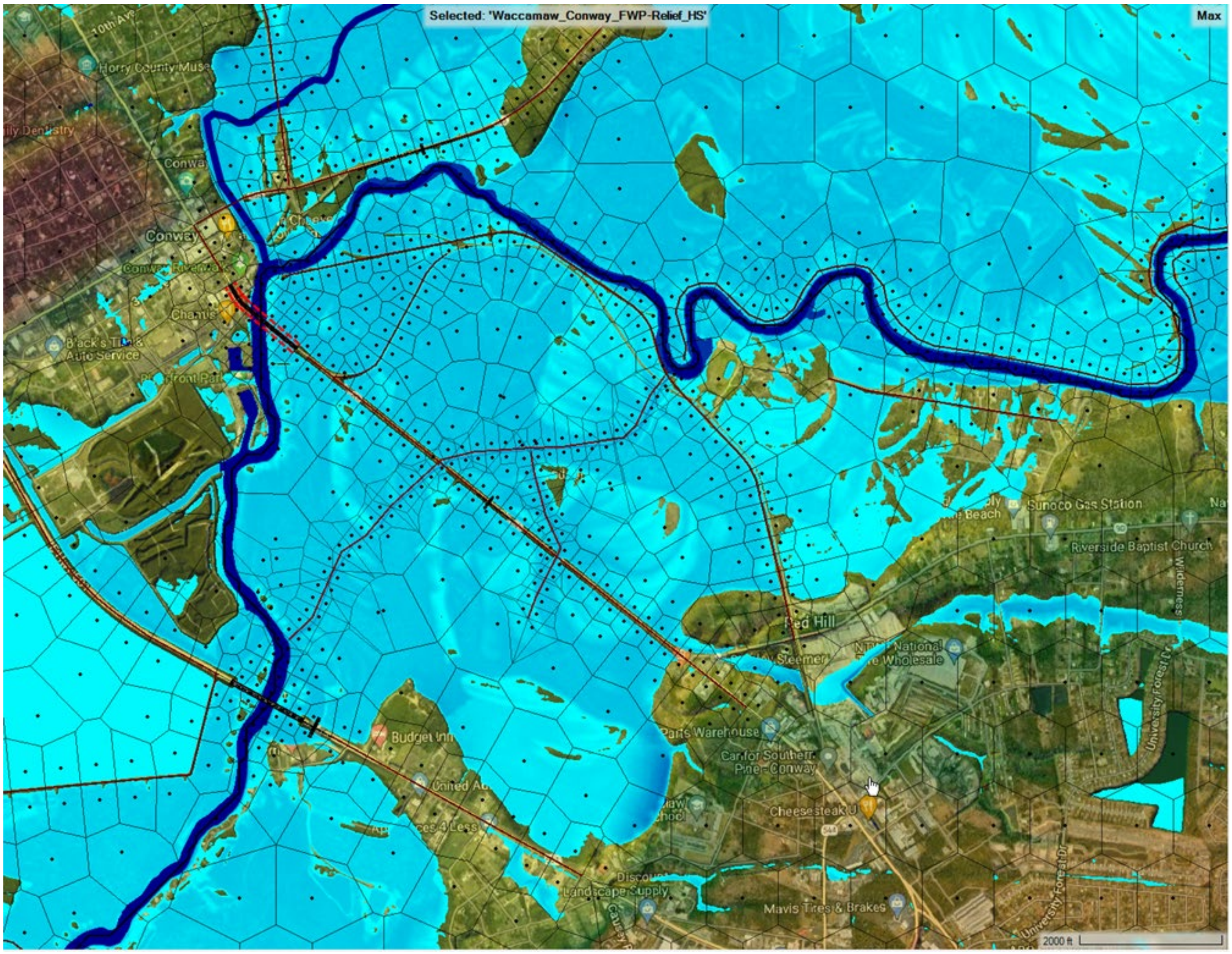


Figure H-12. 1% AEP depth of the FWP in Conway after evaluating the measure in HEC RAS

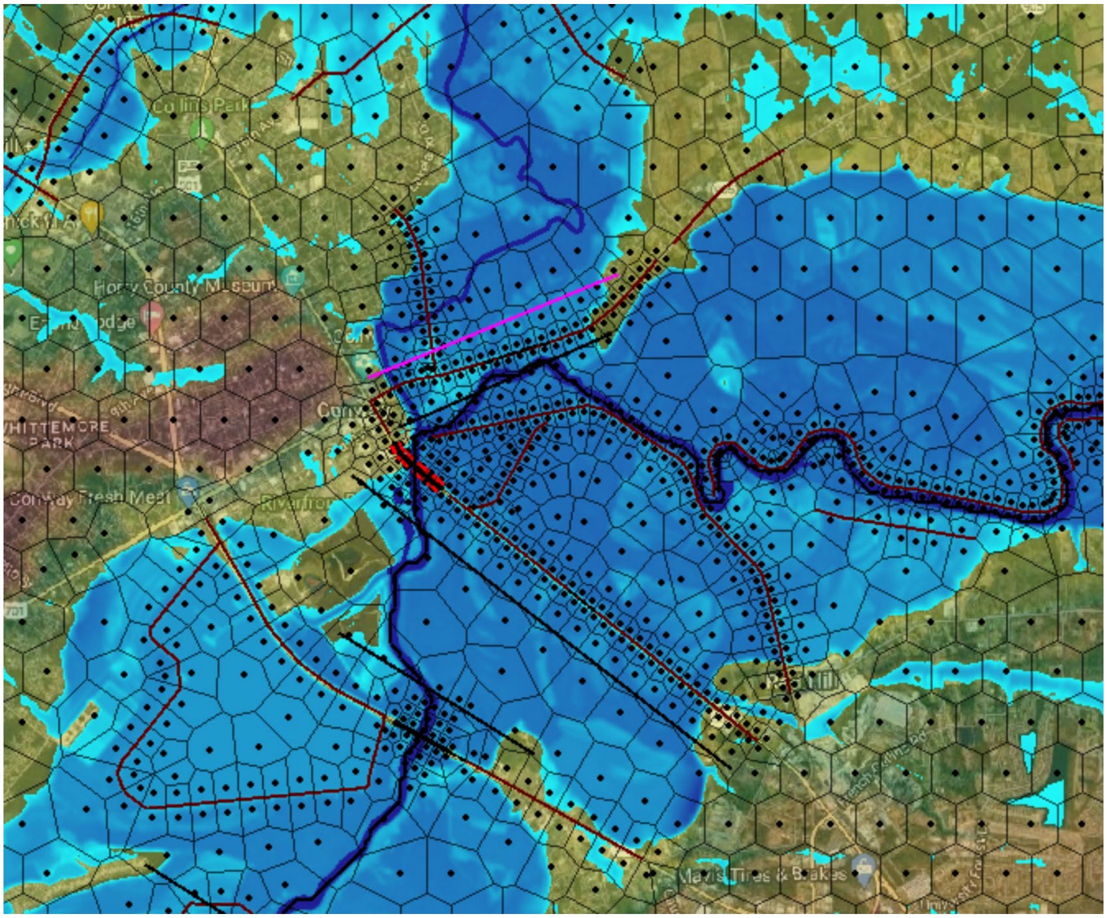


Figure H-13. Location of the cross-section water surface elevation comparison upstream of highway 905

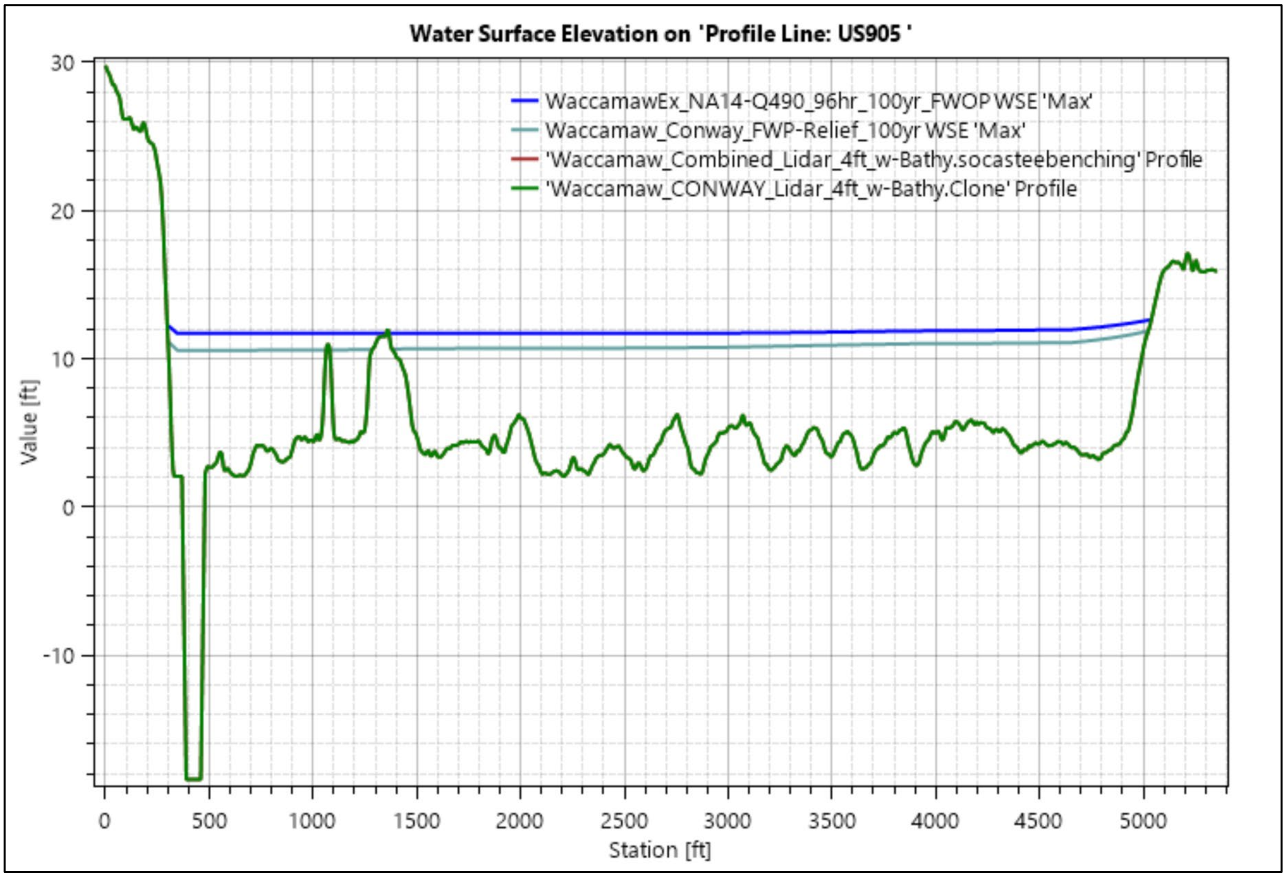


Figure H-14. Water Surface Profile cross section comparison for upstream of 905

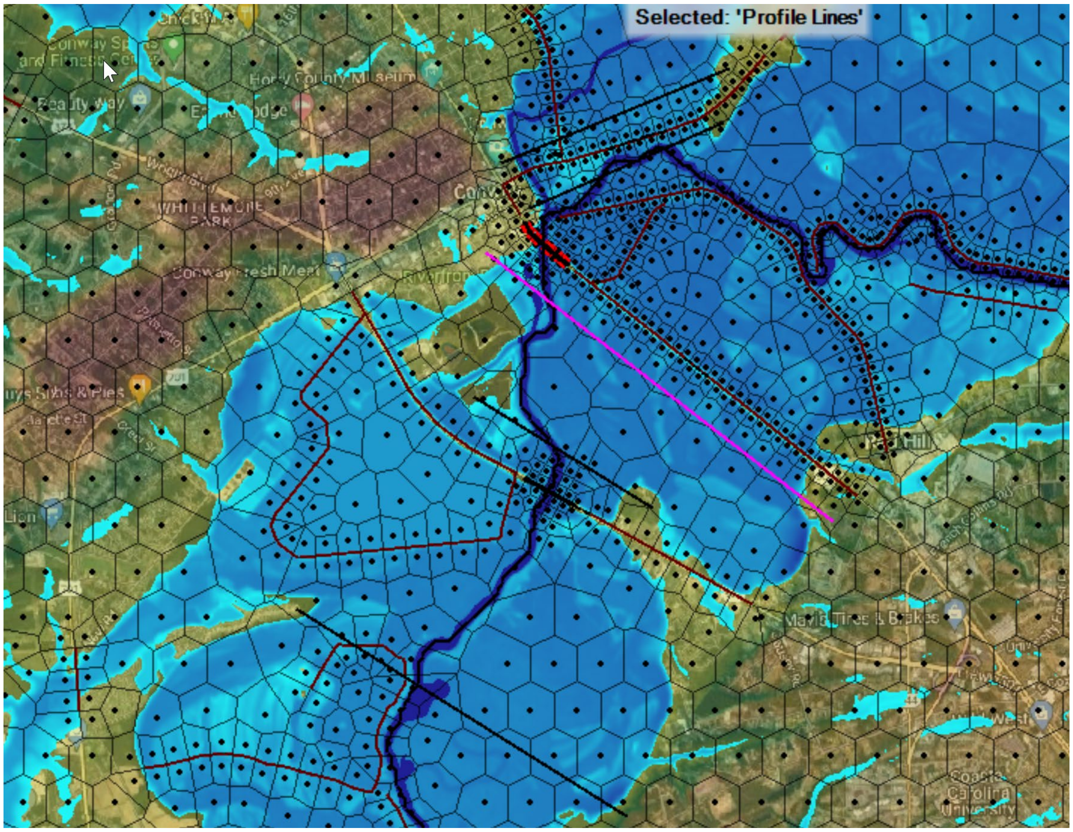


Figure H-15. Location of the cross-section water surface elevation comparison upstream of 501

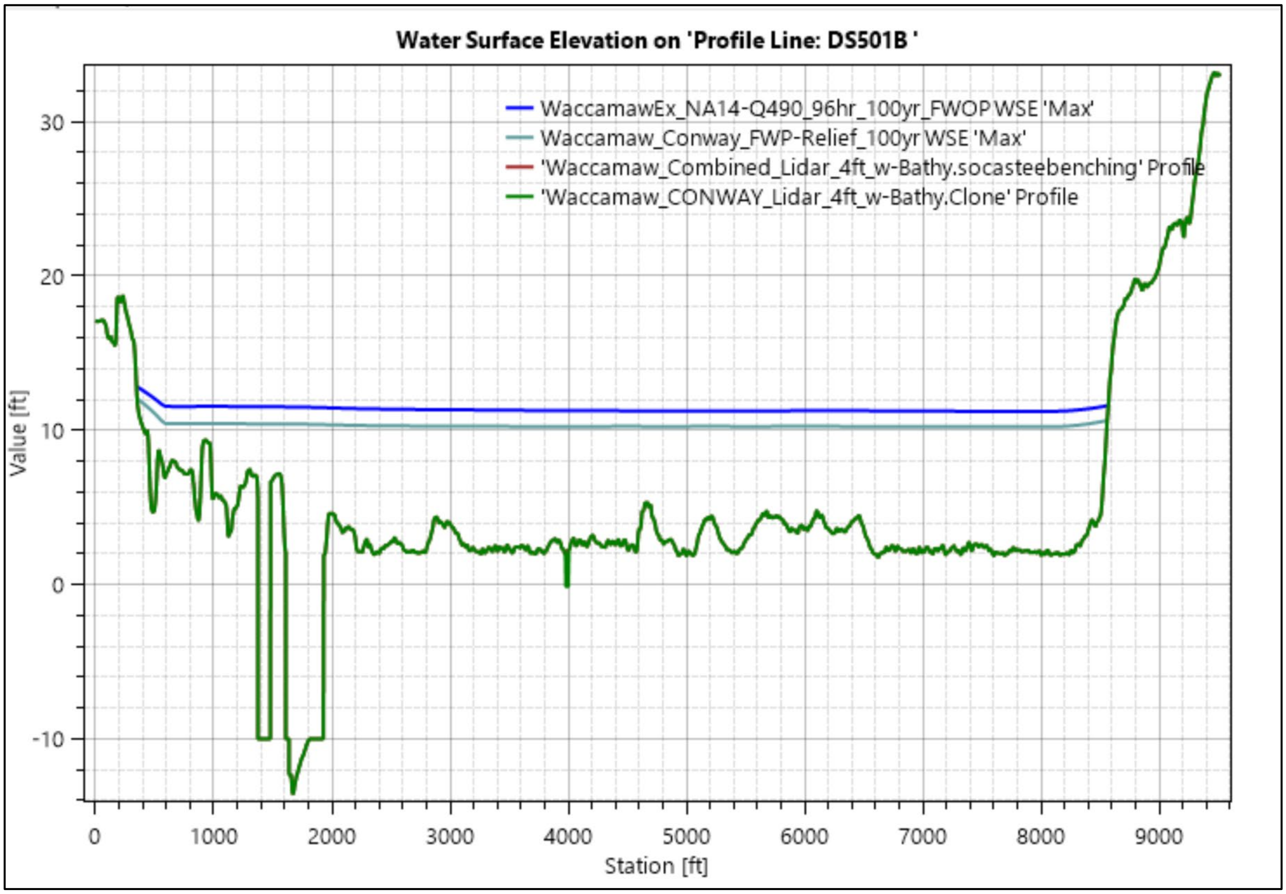


Figure H-16. Water Surface Profile cross section comparison for upstream of 501

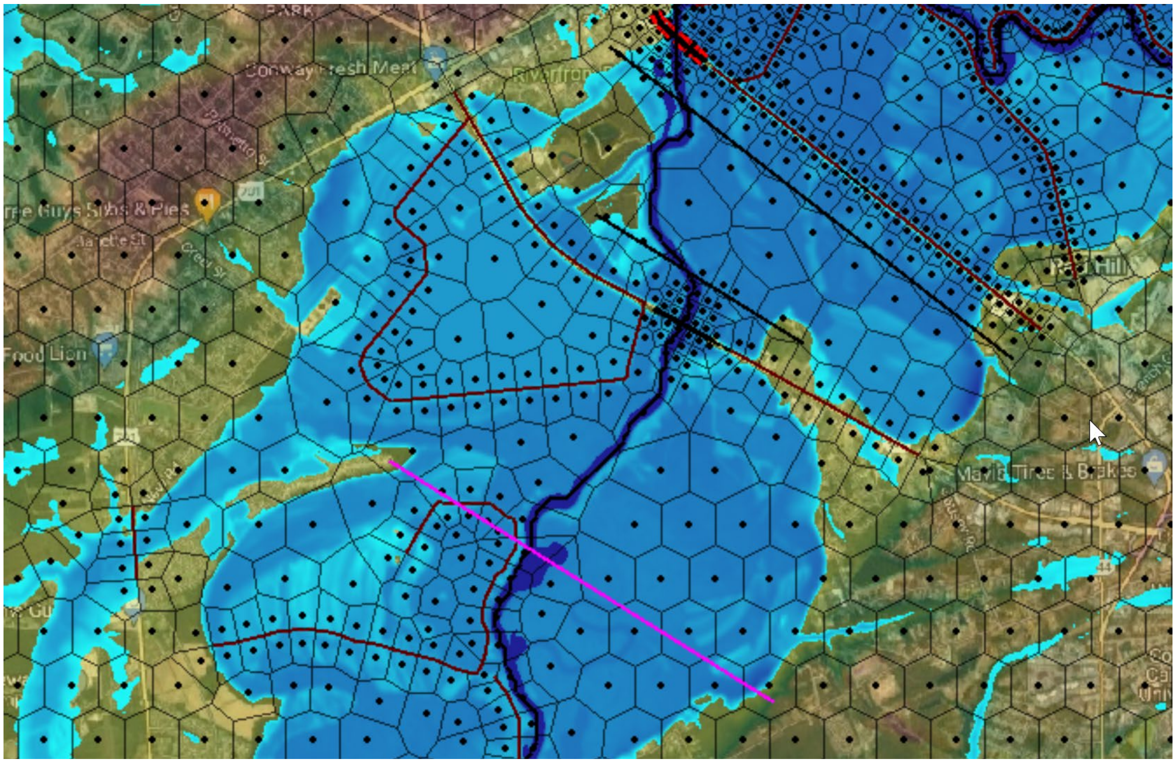


Figure H-17. Location of the cross-section water surface elevation comparison downstream of 501

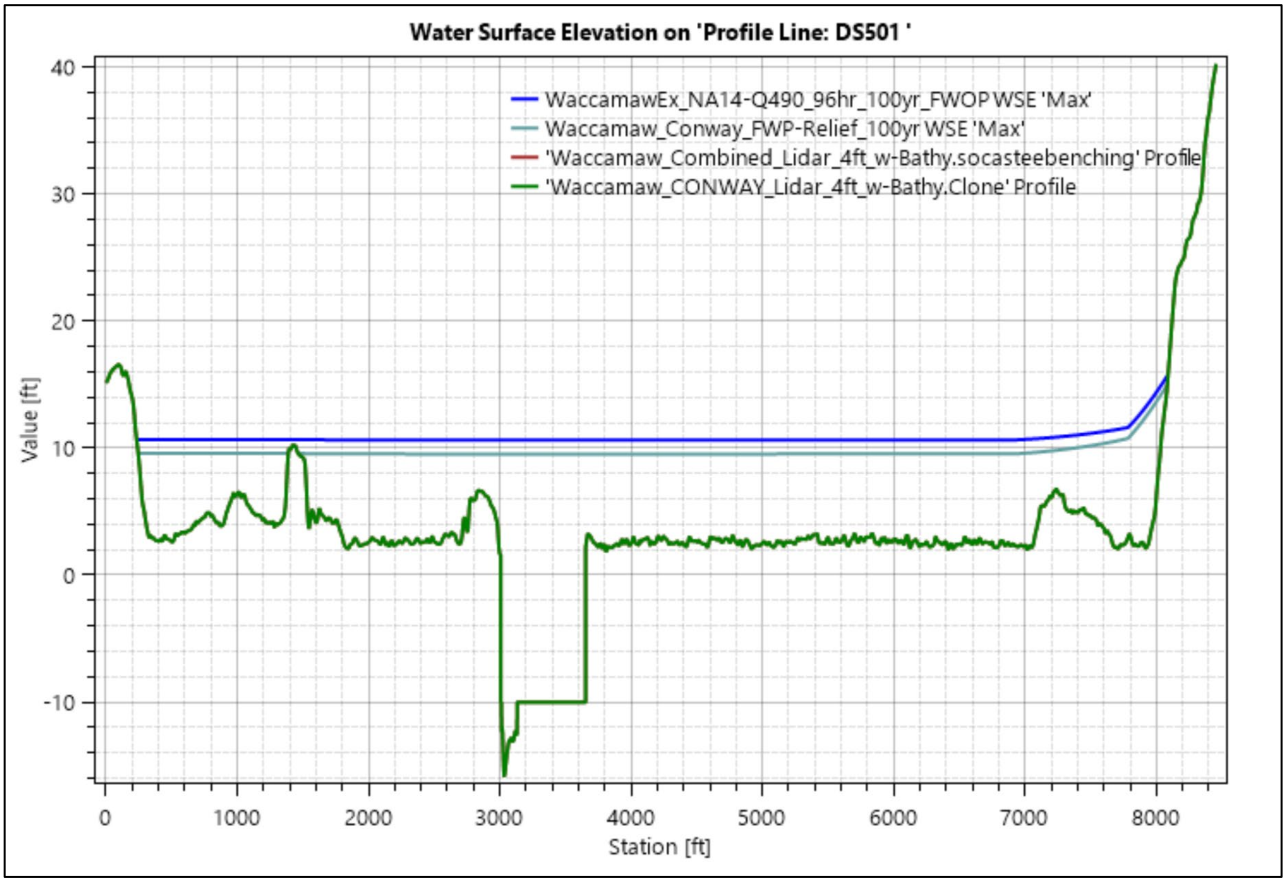


Figure H-18. Water Surface Profile cross section comparison for downstream of 501

Table H-3. 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 H-3 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.

H.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 Inspection of Completed Works (ICW)project of Socastee Creek (**Figure H-19**).

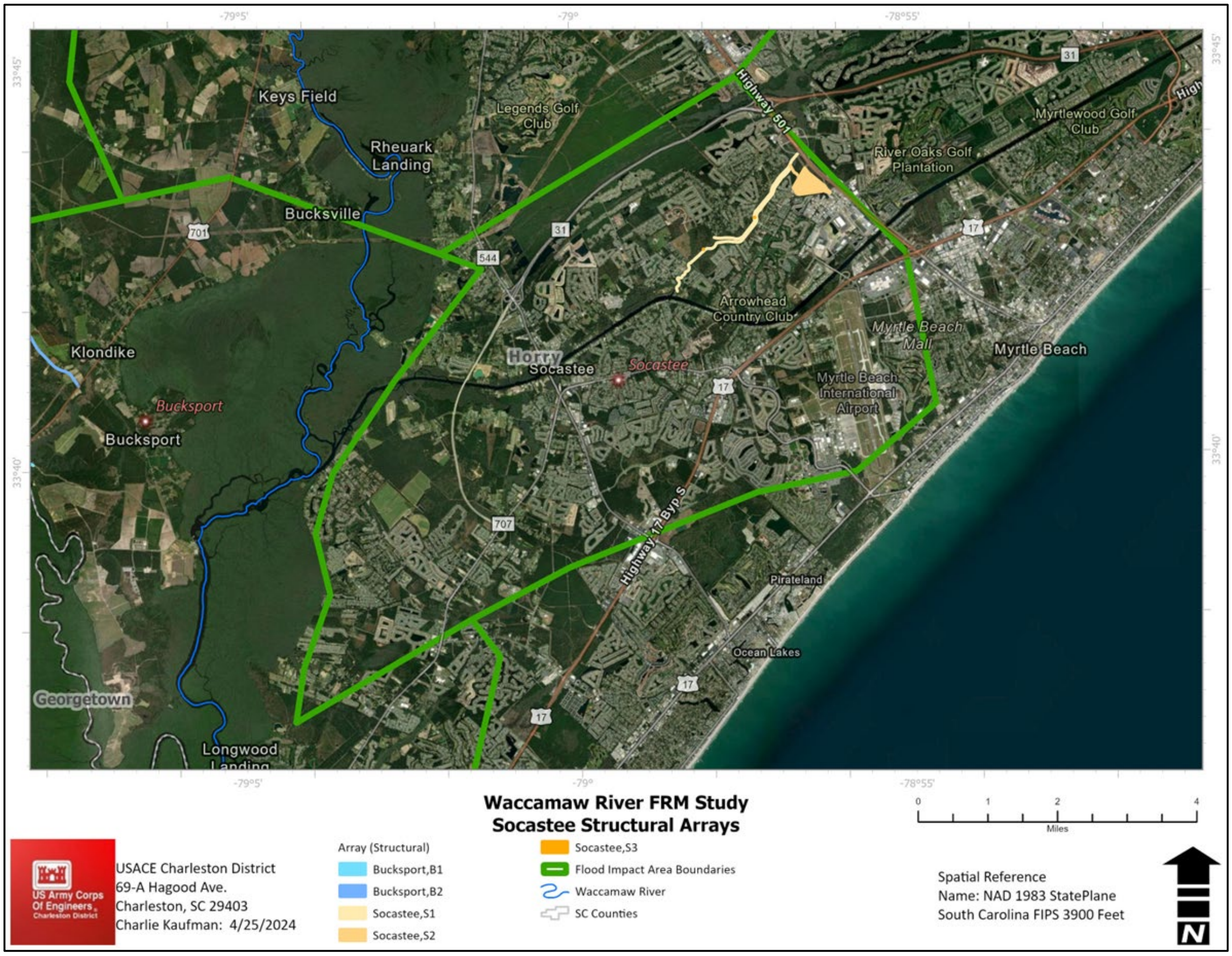


Figure H-19. Socastee Structural Arrays

Table H-4 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 H-4. 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

H.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.

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 H-20. 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 weather-related hazards. Overall, the implementation of floodwalls can contribute to more sustainable and resilient hydrological systems, benefiting both human communities and the natural environment.

H.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.

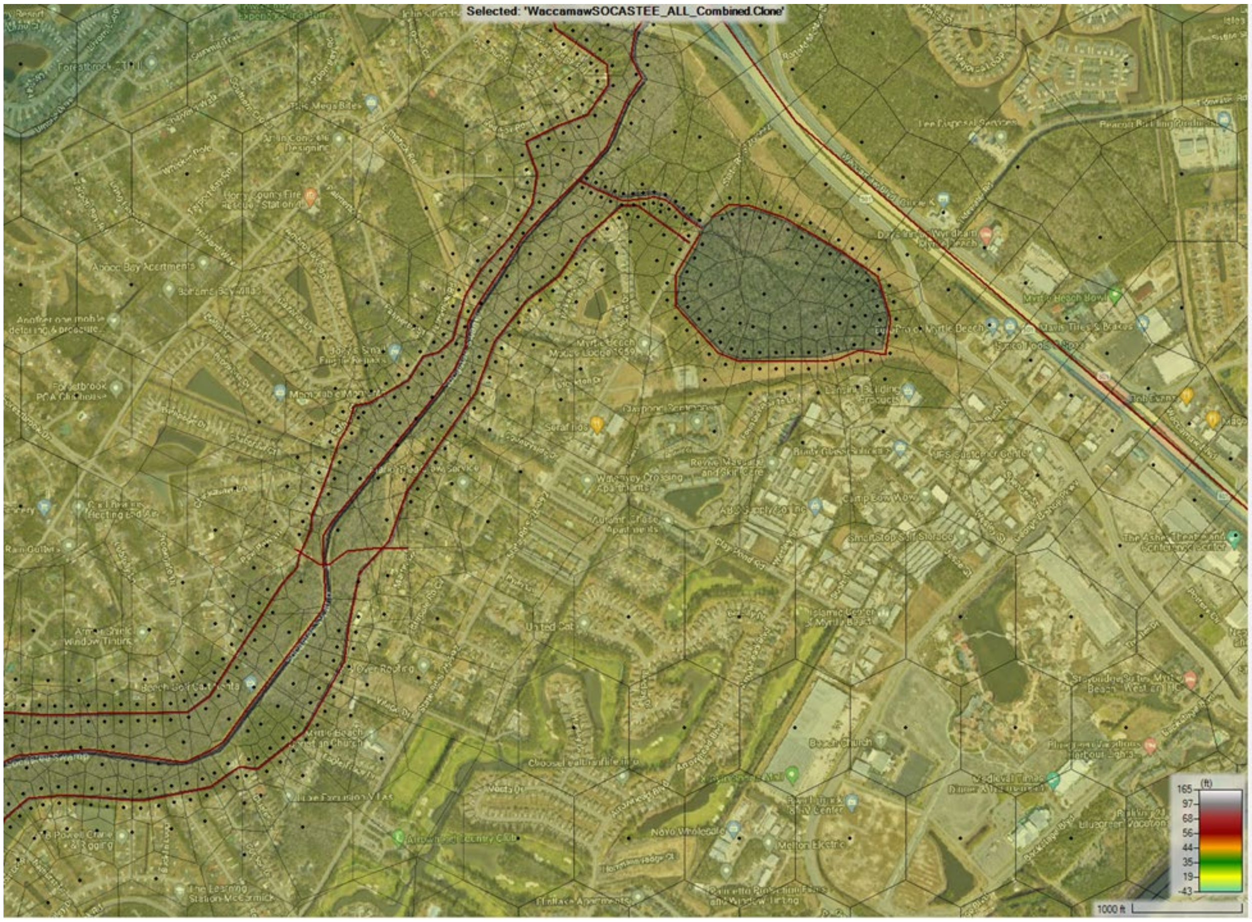


Figure H-21. Refinement of the HEC-RAS grid and topography of the detention pond and channel in Socastee

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.

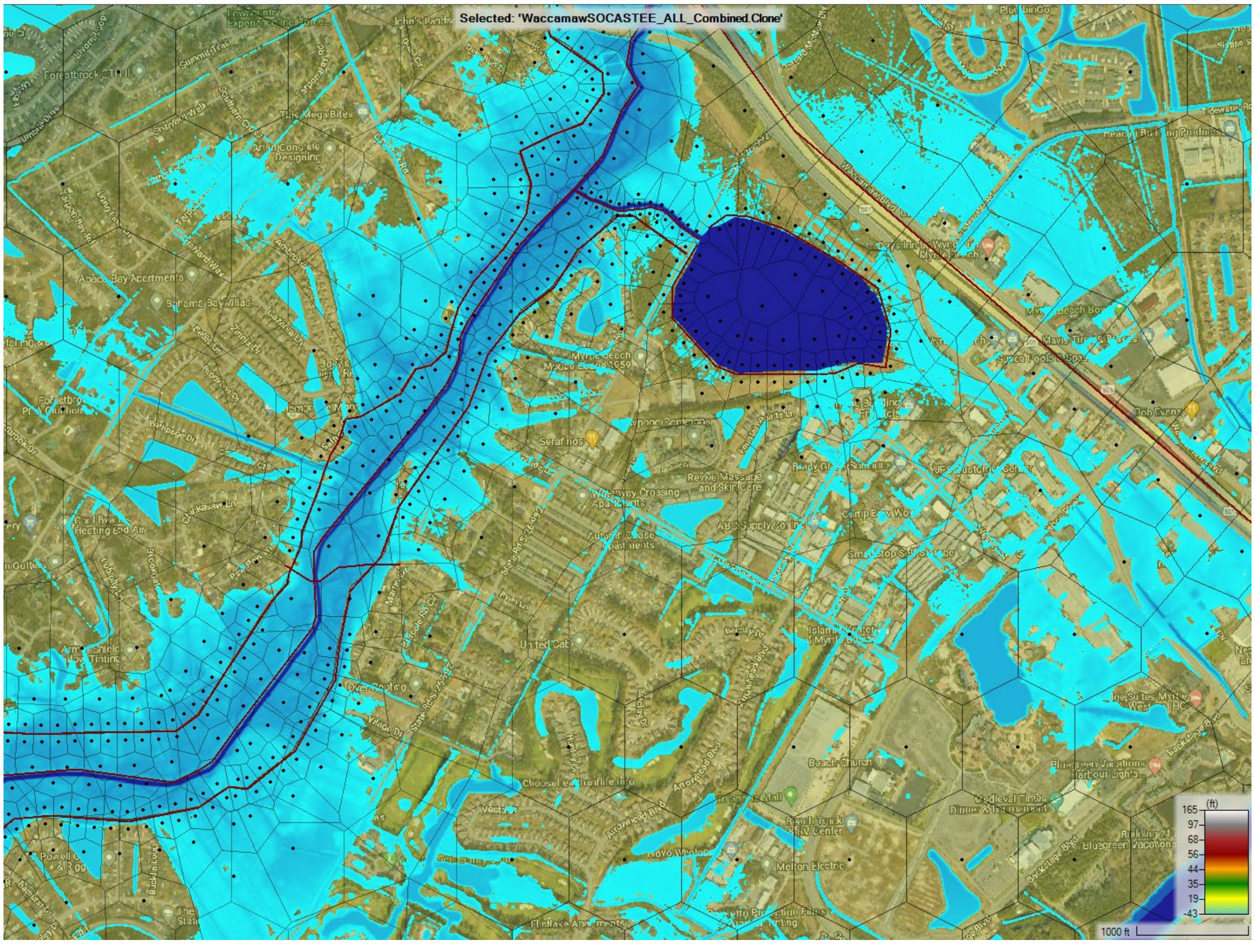


Figure H-22. 1% AEP depth with the evaluated measure of detention pond along Socastee Creek

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 (**Figure H-22**). This plan involves occupying up to 55-acres. An existing tributary would 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 (**Figure H-23**).

The proposed protection is north central Socastee. This area is land locked by Edward E Burroughs 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.

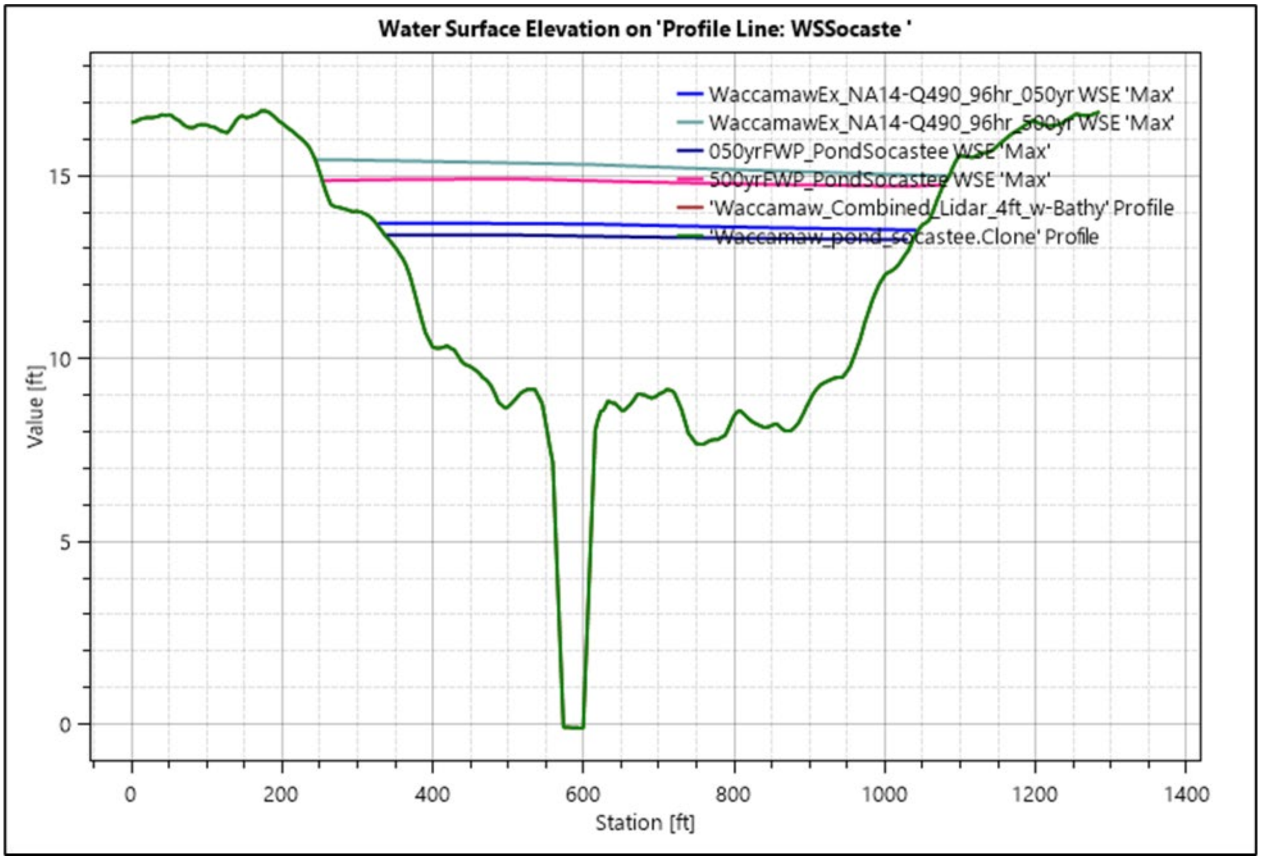


Figure H-23. Comparison of Max WSE for FWOP and FWP including the Detention Pond

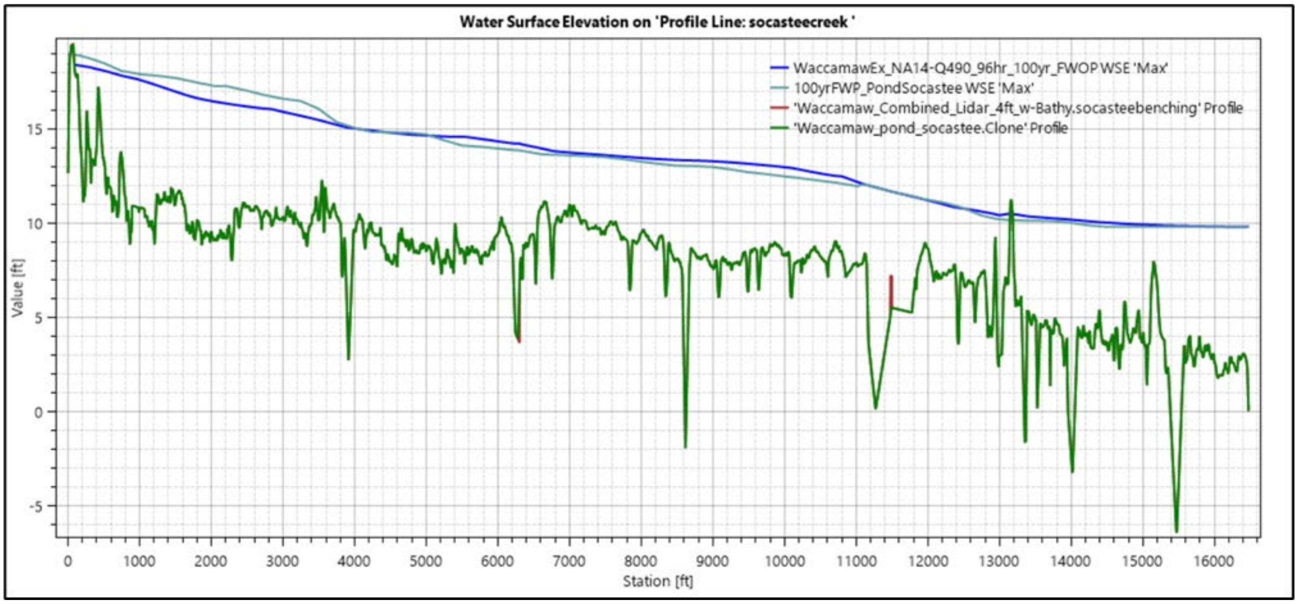


Figure H-24. Comparison of the WSE along Socastee Creek FWOP and FWP

H.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 H-25** shows the locations of the potential weir removals. The proposed measure is intended to decrease flood elevations at upstream homes along Socastee Creek (**Figure H-26**).

Figure H-25. Locations of potential weir removals along Socastee Creek

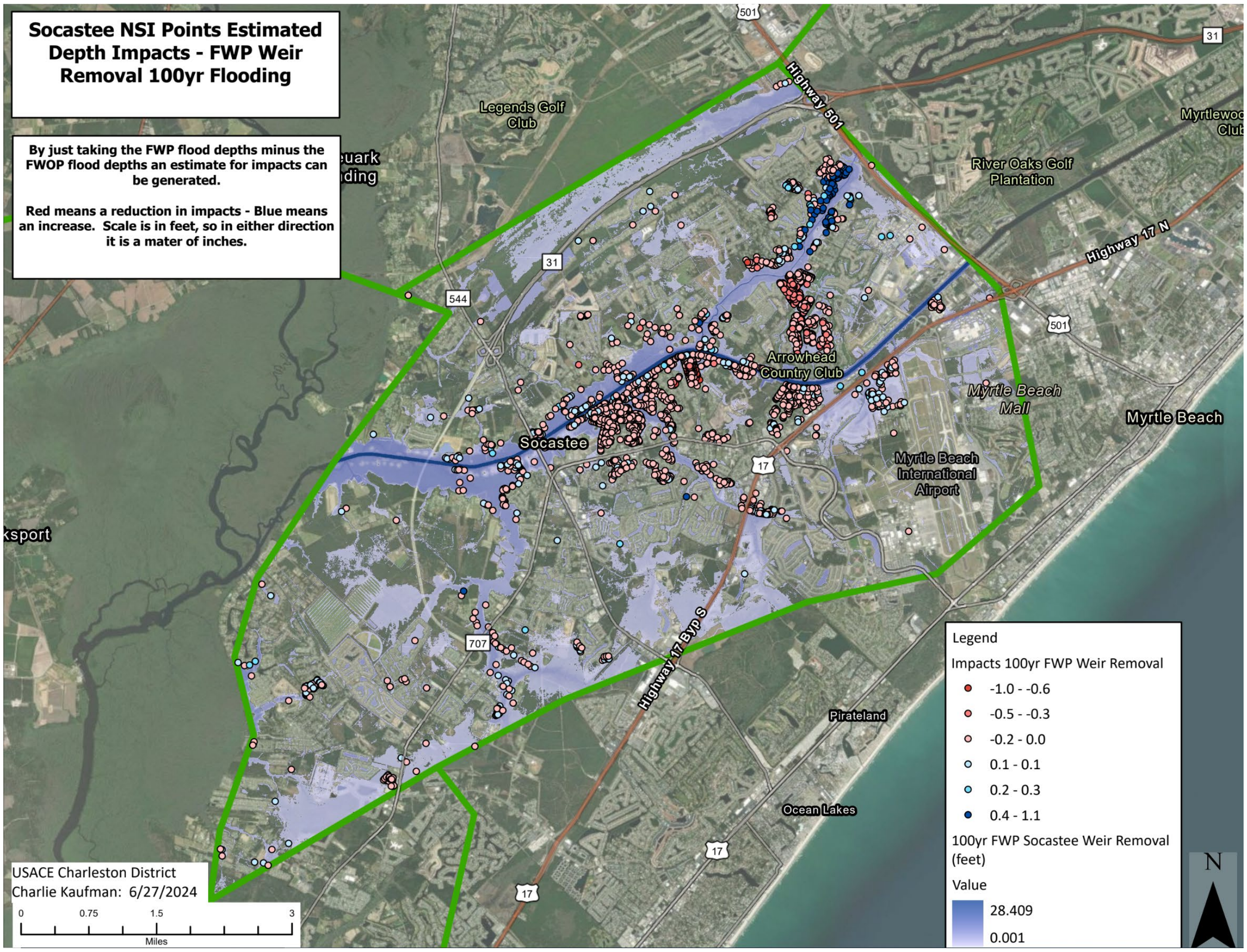


Figure H-26. Structures displayed as decreased water depths with the inclusion of the weir removal

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 substantial 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 H-27** depicts the difference in water depths at each structure.

Figure H-137. Location of the cross sections used in comparison for FWOP vs. FWP weir removal of the WSE

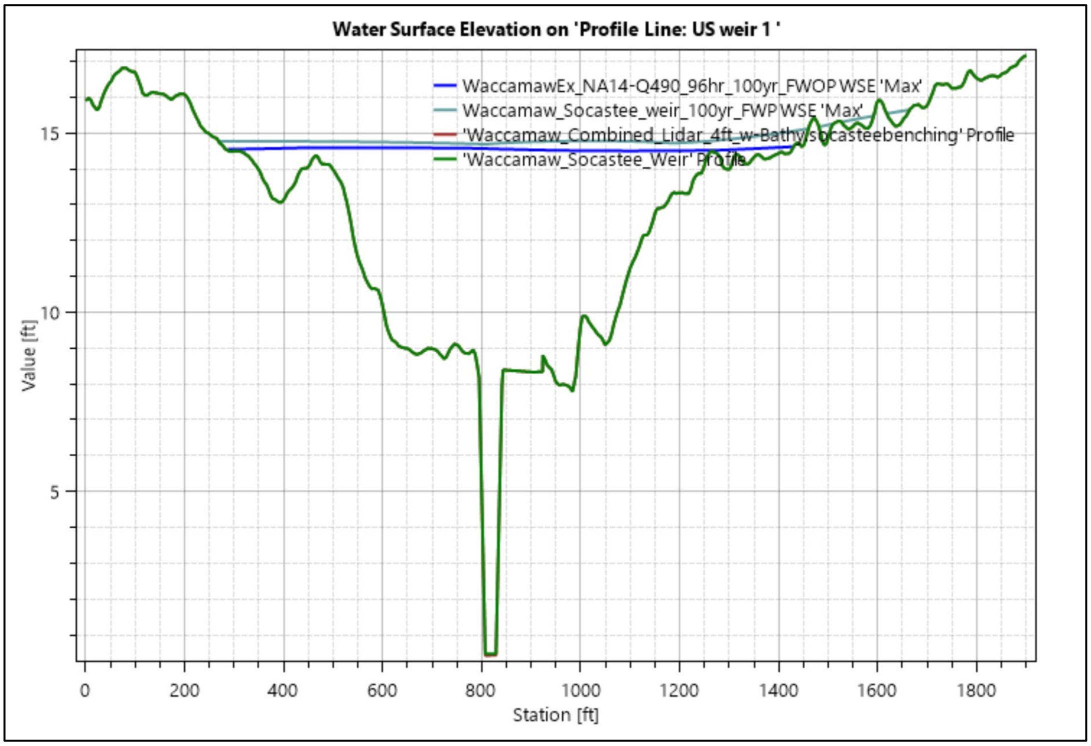


Figure H-148. Cross section WSE comparison of the FWOP and FWP weir removal U/S weir 1

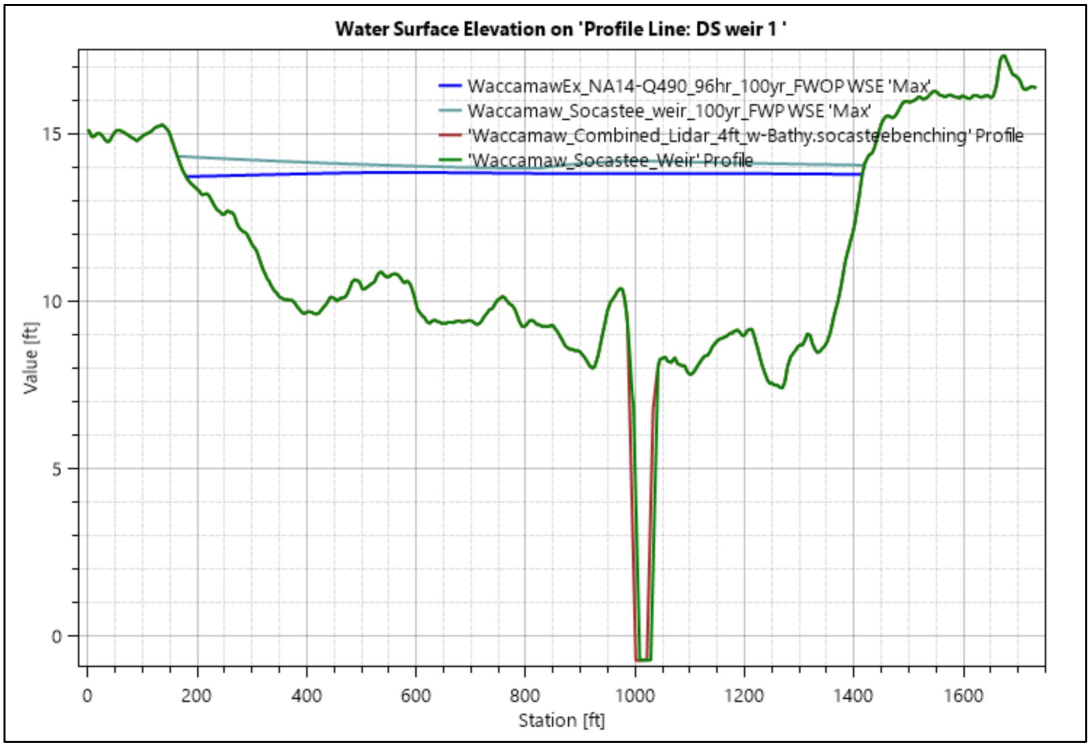


Figure H-15. Cross section WSE comparison of the FWOP and FWP weir removal D/S weir 1

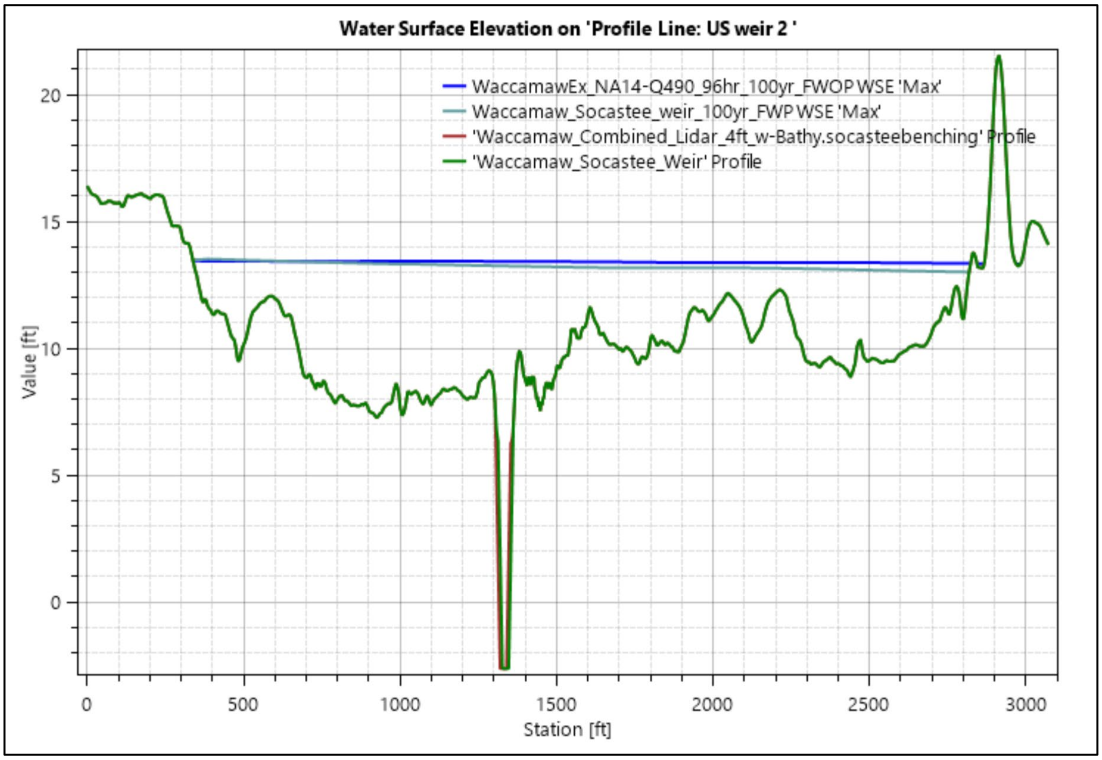


Figure H-30. Cross section WSE comparison of the FWOP and FWP weir removal U/S weir 2

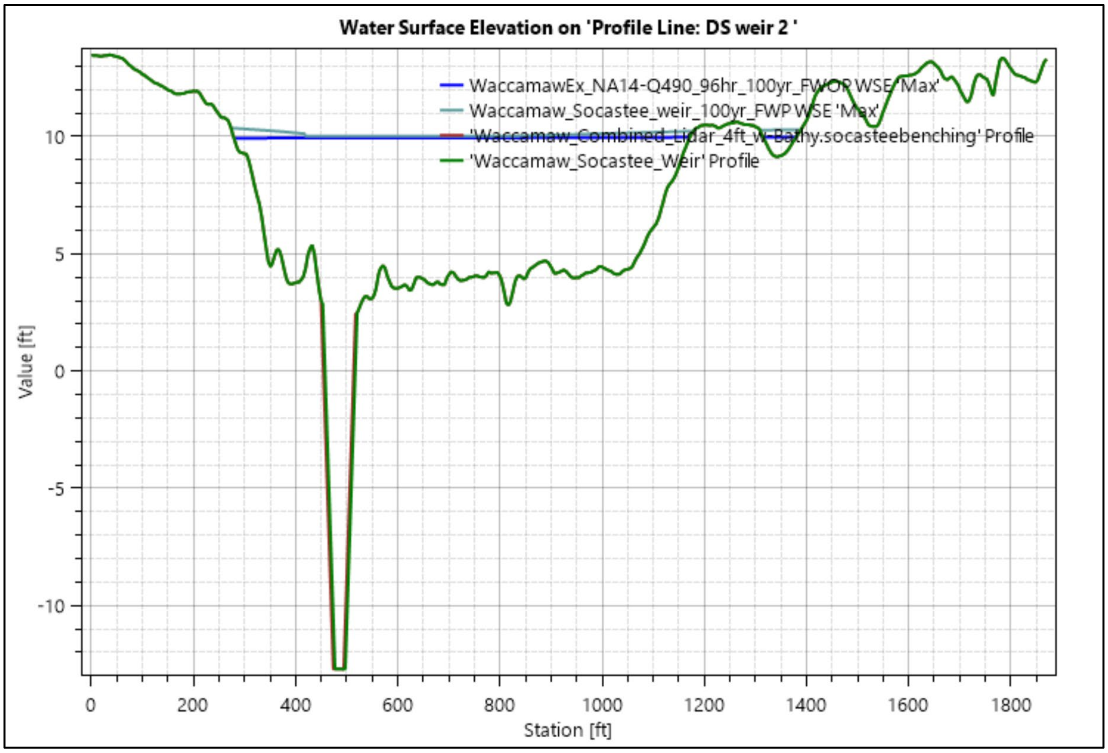


Figure H-31. Cross section WSE comparison of the FWOP and FWP weir removal D/S weir 2

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.

H.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 H-21** shows 1% AEP depths for the combined measure.

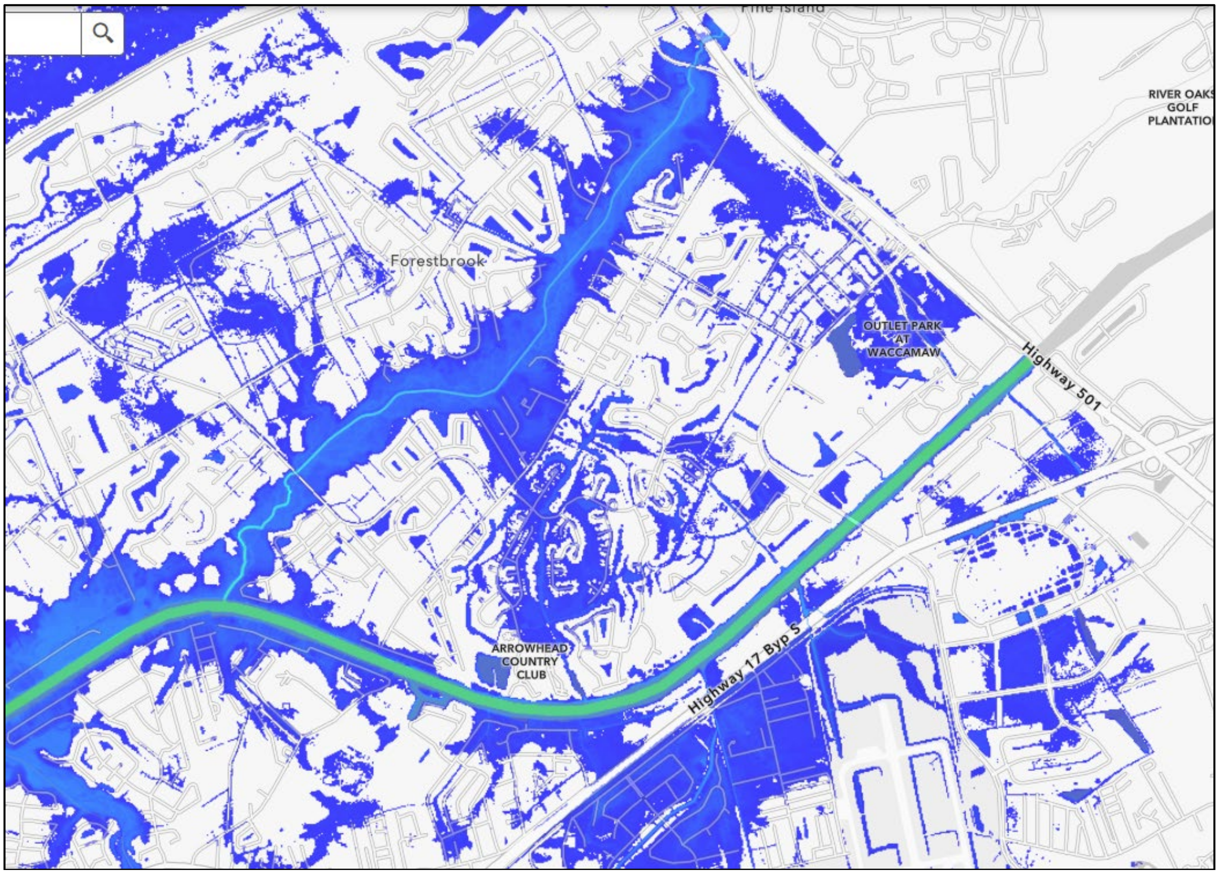


Figure H-32. Depth for 1% AEP for the combined measures

H.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 H-33**.

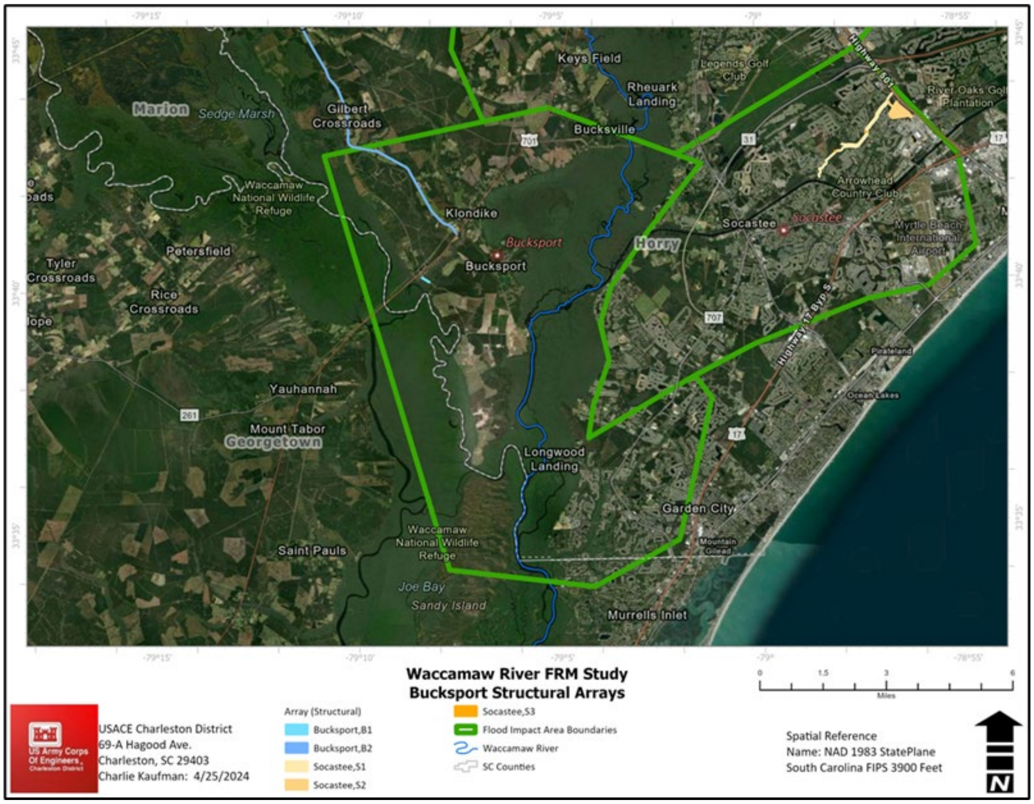


Figure H-163. Bucksport evaluated structural measures

Table H-5 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 H-5. 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 improvements that would reduce risk

H.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.

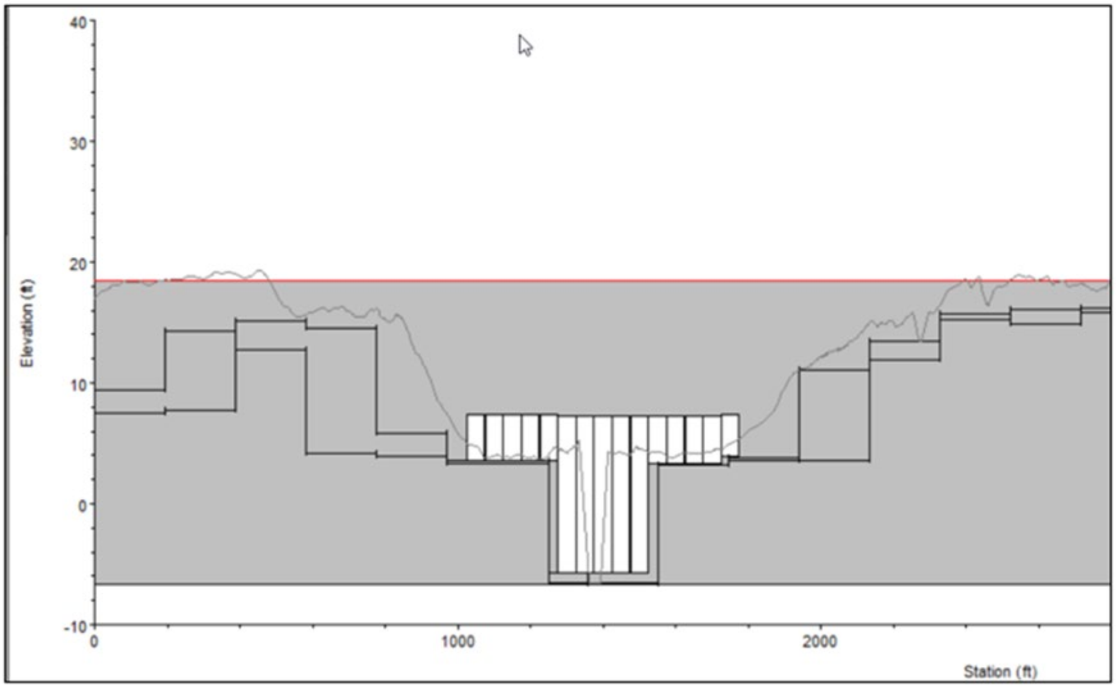


Figure H-34. Cross section of geometry input parameters of the floodgate

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 constructed and completed 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 H-35. Location of the Floodgate along Cowford Swamp

H.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 weather 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 substantial hydrologic benefits by improving drainage, reducing flooding, preserving natural watercourses, protecting aquatic ecosystems, and enhancing long-term resilience to changing conditions. 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.

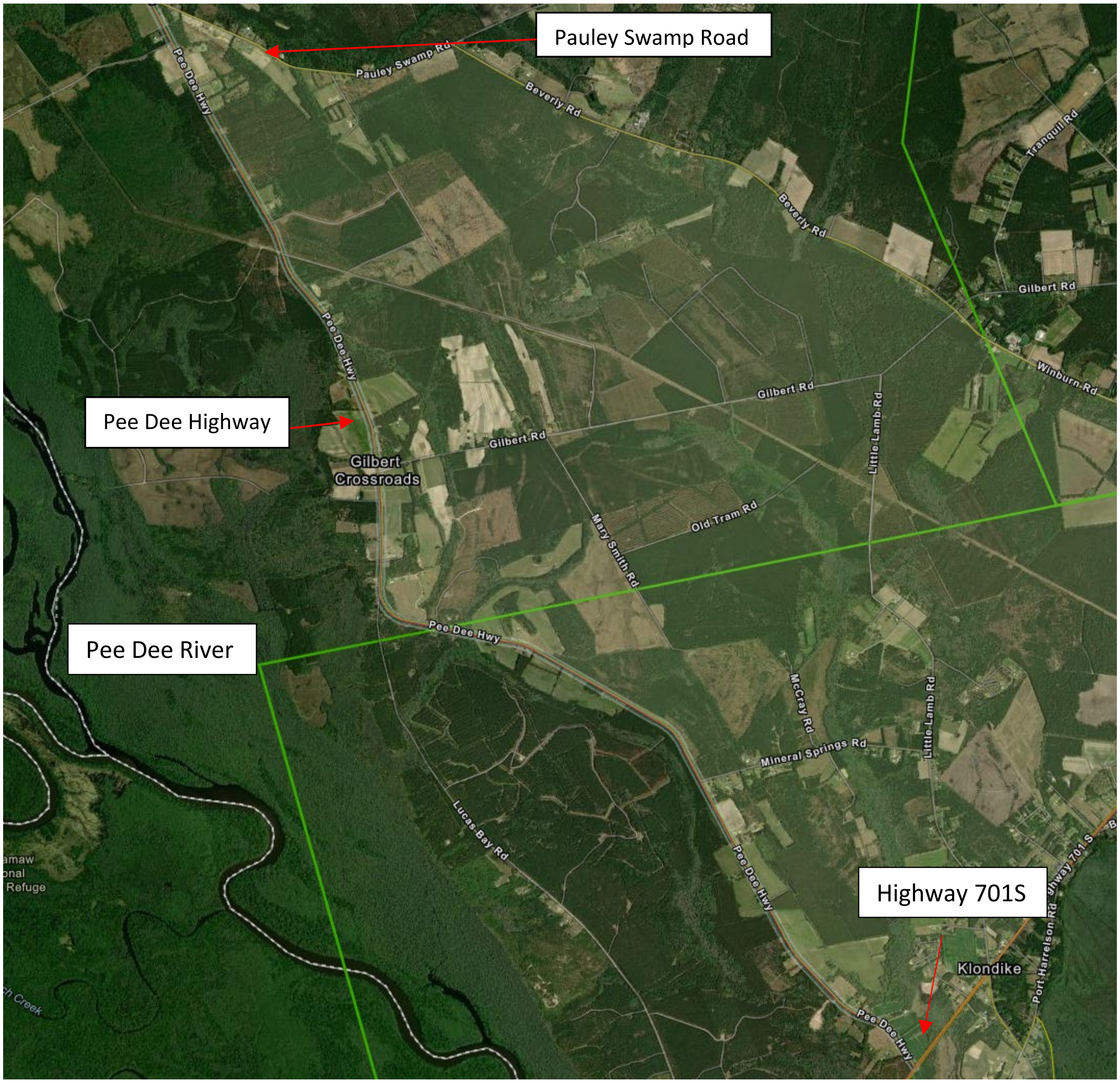


Figure H-36. Identification of the Road raising in Bucksport.

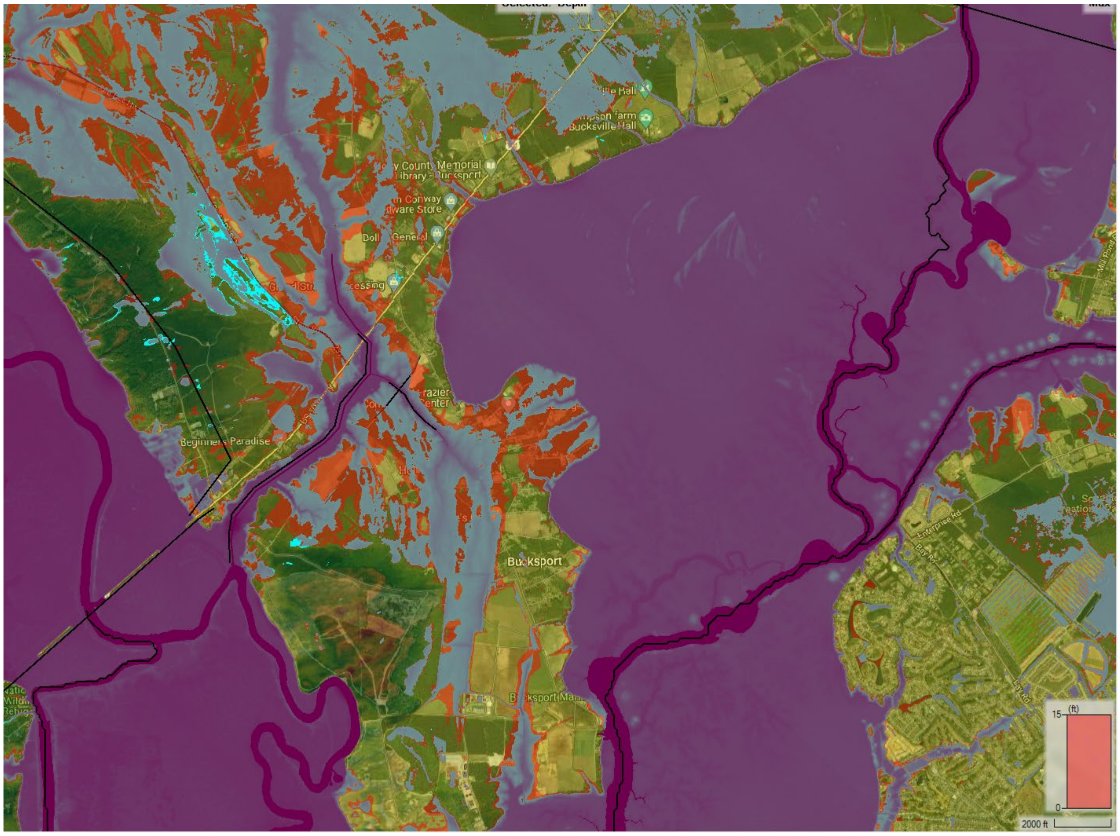


Figure H-37. Combined measures in Bucksport showing the depth FWOP and FWP

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. Changing conditions 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.

H.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.

H.5 Refined Structural Alternatives

Upon completion of the FWP economic analysis for the preliminary alternatives, it was determined that only two of the 13 structural alternatives produced a benefit-to-cost ratio (BCR) above 1.0. Specifically, overall perceived damages under FWOP conditions revealed significant challenges in the ability for structural measure refinement to elevate most alternatives to a positive BCR. To address this, the structural measures were modeled in greater detail and refined to advance toward the 35% design maturity level. This refinement process incorporated engineering assumptions, hydraulic modeling updates, and constructability considerations, ensuring that the alternatives tested reflected a more realistic and technically feasible design concept. Even after this refinement process, only two measures remained cost-effective: the relief bridges in Conway and the weir removal in Socastee. Importantly, these two measures are standalone features and do not need to be combined in order to achieve a positive BCR. At 35% design maturity, the BCR for the Relief Bridges was 5.5 and for the Weir Removal was 8.2, clearly demonstrating their viability as economically justified alternatives.

H.5.1 Conway Relief Bridges (Cross-Drains)

In order to fully capture the design of the cross drains, H&H modeling was conducted to verify sizing, spacing, and hydraulic capacity. The initial modeling assumed circular culverts; however, once the geotechnical analysis was completed, the design was updated to incorporate box culverts instead. The geotechnical investigation provided critical information on subsurface conditions, including soil bearing capacity, settlement potential, lateral earth pressures, and groundwater conditions. These factors directly influence the type and geometry of culvert that can be structurally supported without long-term performance issues. For example, soft or compressible soils with limited bearing strength may not provide adequate support for concentrated point loads from circular culverts, whereas box culverts distribute loads more evenly across a larger footprint. Additionally, high groundwater levels and lateral pressures can affect culvert stability and durability, necessitating a reinforced box design that resists uplift, buoyancy, and soil movements.

As a result, Highway 905, 501, and 501B were all remodeled in the H&H analysis to confirm that the redesigned box culverts maintain equivalent or improved hydraulic performance while satisfying geotechnical stability requirements. The updated modeling ensured that the revised cross-drain structures can transfer the same flow capacity while also meeting long-term structural and geotechnical performance standards.

Table H-6: Relief Bridges dimensions and quantities.

Bridge Location	No. of Culverts	Length (ft)	Height (ft)	Width (ft)	Mannings n	Slope (%)	Inlet elev. (ft.NAVD88)	Outlet elev. (ft.NAVD88)	Freeboard 2% AEP ft
905	3	83	3	6	0.012	0.4	3.6	3.2	2.17
501B	2	83	7	8	0.012	0.4	2.9	2.5	3.89
501	2	127	7	8	0.012	0.4	3.0	2.5	3.76

Bridge Location	No. of Culverts	Length (ft)	Height (ft)	Width (ft)	Mannings n	Slope (%)	Inlet elev. (ft.NAVD88)	Outlet elev. (ft.NAVD88)	Freeboard 2% AEP ft
501 (trenchless installation)	3	127	4	10	0.012	0.4	3.0	2.5	2.38

Overall, the use of box culverts with flared wing walls and uniform spacing allows the project to maintain hydraulic capacity equivalent to or greater than the original design assumptions while aligning with the subsurface conditions and long-term stability requirements.

H.5.1.1 Highway 905

Highway 905 will include three box culverts, each measuring 6 feet (72-in) wide by 3 feet (36in) tall and extending 83 feet in length. With a Manning's n of 0.012 and a slope of 0.4%, this configuration provides the necessary hydraulic capacity while maintaining stability in the relatively shallow profile dictated by the site's geotechnical conditions.

Figure H-178: Aerial view of 905 relief bridge location

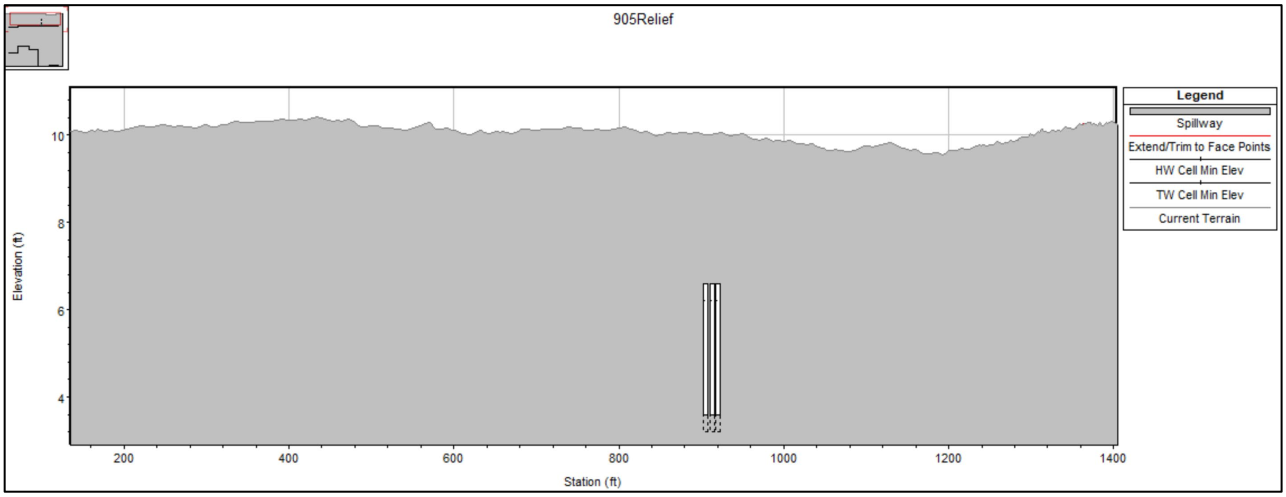


Figure H-39: HEC-RAS cross section at highway 905

Figure I-2 presents the cross-sectional profile of the Highway 905 relief crossing, showing the roadway embankment and the proposed culvert configuration. The gray shaded area represents the existing ground terrain, with elevations ranging from approximately 4 to 11 feet. The embankment crest is generally consistent around elevation 10 to 11 feet across the section.

At approximately Station 900, three reinforced concrete box culverts are shown extending through the embankment. Each culvert measures 6 feet in width, 3 feet in height, and 83 feet in length, with flared wing walls and a 1.5-foot spacing between barrels. The vertical placement of the culverts ensures that the invert is positioned below the surrounding ground to allow continuous conveyance of flows.

The culvert barrels are identified in the legend as “Spillway,” with associated headwater and tailwater control elevations used in the hydraulic computations. This figure demonstrates the embedment of the culverts within the embankment and verifies their hydraulic functionality relative to the terrain profile. The cross-sectional representation confirms that the culvert system can provide the necessary conveyance capacity under modeled flow conditions while also meeting structural and geotechnical requirements for long-term stability.

H.5.1.2 Highway 501B

Highway 501B will utilize two larger culverts, each 8 feet wide by 7 feet tall with an 83-foot length. These dimensions accommodate a higher conveyance capacity at this location, reflecting both the modeled hydrology and the deeper embankment. The larger cross section ensures resilience to long-duration flow events and accounts for the soft foundation soils that benefit from the wider load distribution of the box structure.

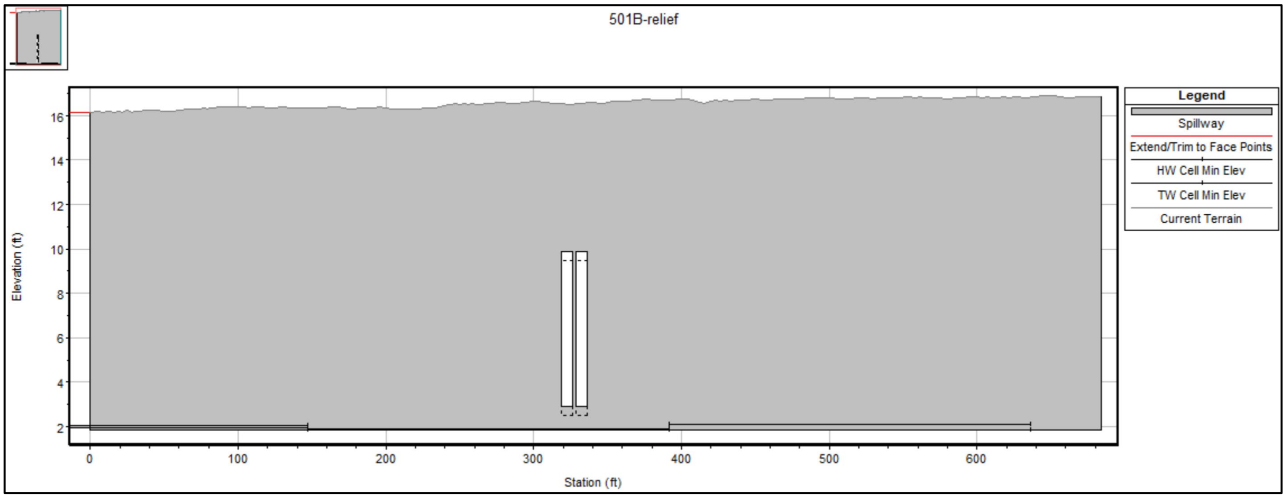


Figure H-18: Cross drain cross section Highway 501B

Figure H-40 illustrates the cross-sectional profile of the Highway 501B relief crossing. The gray shaded area represents the current terrain, with ground elevations ranging from approximately 2 feet at the channel bottom to 16 feet along the roadway embankment crest. The embankment spans the cross section consistently near elevation 15 to 16 feet.

At approximately Station 320, two reinforced concrete box culverts are shown embedded within the embankment. Each culvert measures 8 feet in width, 7 feet in height, and 83 feet in length. The barrels are spaced 6-in apart, with flared wing walls designed to reduce inlet losses, improve approach flow conditions, and provide stability against lateral soil pressures. The culvert invert elevation is placed below the surrounding ground, ensuring adequate conveyance of channel flow beneath the roadway.

The twin-box culvert configuration at Highway 501B ensures sufficient flow capacity while addressing geotechnical considerations, including load distribution and long-term settlement potential.

H.5.1.3 Highway 501

There are two different culvert design dimensions for the relief culvert for Highway 501. Depending on the SCDOT regulations on the open-cut installation or trenchless installation there are two presented designs. For the open cut installation, two (2) box culverts each 8 feet in width and 7 feet in height, 127 ft in length. For the trenchless installation, three (3) box culverts each 10 feet wide by 4 feet tall, extending 127 feet in length. The extended length reflects the broader roadway embankment at this location, while the cross-sectional area and twin-barrel arrangement provide the required conveyance under modeled flood conditions. The box design also minimizes long-term settlement risks associated with the geotechnical findings at this crossing.

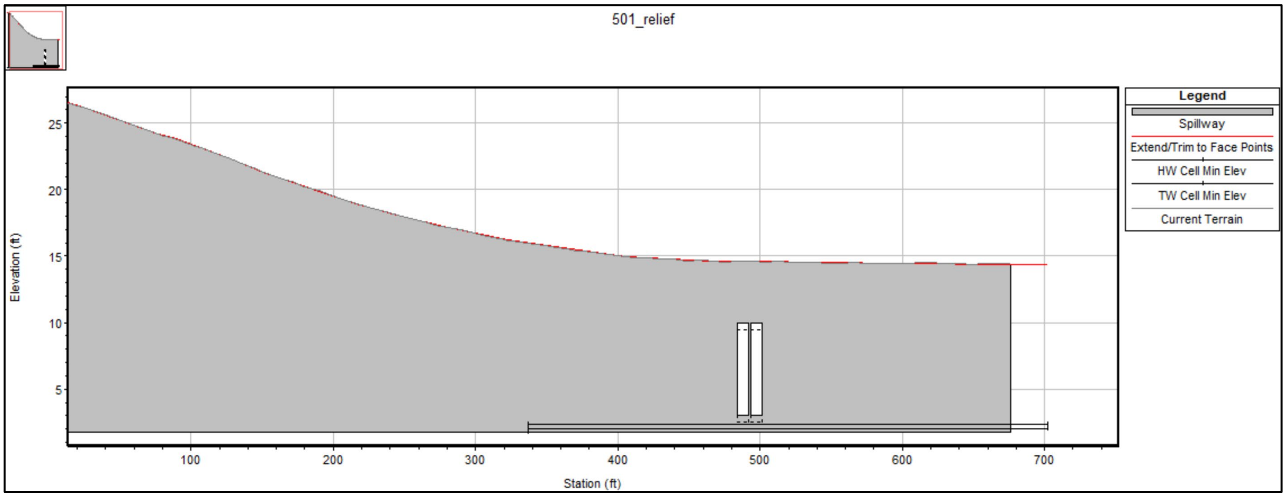


Figure H-41: Cross sectional view Highway 501.

Figure H-41 shows the cross-sectional profile of the Highway 501 relief crossing. The existing terrain ranges from approximately elevation 3 feet at the channel bottom to over 25 feet at the embankment crest.

At approximately Station 500, two reinforced concrete box culverts are placed within the embankment. Each culvert measures 8 feet in width, 7 feet in height, and 127 feet in length, with a 0.5-foot spacing between barrels and flared wing walls to improve hydraulic efficiency. The embedment of the culverts below grade provides adequate conveyance capacity while maintaining roadway stability.

This profile confirms that the twin-box culvert configuration satisfies the hydraulic requirements while addressing geotechnical constraints for the larger Highway 501 crossing.

H.5.1.4 Construction Sequence for Relief Bridge Installation

The proposed measure includes the installation of three relief bridges along the project corridor. These structures are intended to increase conveyance capacity, reduce backwater conditions, and alleviate localized flooding by allowing flows to pass more efficiently through natural floodplain areas. The following sequence outlines the anticipated construction approach.

Pre-Construction Preparations

- Conduct detailed site surveys and staking of bridge footprints, including limits of disturbance, staging areas, and access routes.
- Establish staging and laydown areas outside of flood-prone zones to minimize environmental impacts.
- Install erosion and sediment control measures (silt fencing, turbidity curtains, check dams) at each site prior to ground disturbance.

Access and Temporary Diversion

- Construct temporary access roads and working platforms using stone or matting to support heavy equipment.

- Install temporary cofferdams, diversion channels, or pump-around systems as necessary to manage creek flows and provide a safe, dry work zone.
- Maintain minimum flow downstream to protect aquatic habitat during construction.

Excavation and Foundation Preparation

- Excavate approach embankments and channel sections at each bridge location to subgrade elevation.
- Install foundation support systems, which may include driven piles, drilled shafts, or spread footings depending on site-specific geotechnical conditions.
- Place bedding layers and install scour protection stone at abutments and piers.

Bridge Superstructure Installation

- Construct abutments, wing walls, and piers using reinforced concrete.
- Set precast or cast-in-place bridge beams or girders using cranes or launching systems.
- Install bridge decking, barriers, and parapets according to design specifications.

Channel and Roadway Tie-Ins

- Regrade and stabilize channel approaches to smoothly connect with the new bridge openings, ensuring proper conveyance.
- Place riprap or other armoring around piers and abutments for scour protection.
- Reconstruct roadway approaches and transition embankments, compacting to design elevations.
- Install guardrails, signage, and striping for traffic safety.

Restoration and Demobilization

- Remove temporary cofferdams, access roads, and diversion structures once the bridge and approaches are complete.
- Restore disturbed areas by seeding with native vegetation and implementing bio-stabilization techniques where appropriate.
- Conduct final inspection to confirm structural integrity, hydraulic performance, and compliance with environmental commitments.

The installation of relief bridges at the three identified locations will increase hydraulic capacity along the corridor, reduce upstream flood elevations, and mitigate roadway overtopping during high-water events. In addition, these bridges will improve long-term resilience of transportation infrastructure by reducing maintenance demands associated with repetitive flood damage.

H.5.2 Socastee Creek Weir Removal

The Socastee Creek Federal Project currently incorporates two existing weir structures along Socastee Creek. Each structure measures approximately 40 feet in width and 10 feet in height, constructed from reinforced concrete supported by sheet pile. To provide stability and scour resistance, both the upstream and downstream faces of the weirs are armored with a riprap apron approximately 2 feet thick and extending 50 feet in width.

The original purpose of these weirs was to regulate groundwater levels and preserve wetland functions by maintaining hydrologic conditions similar to those that existed prior to construction. At the time, this approach was expected to mitigate wetland loss and protect natural habitat in the surrounding area. However, over the past several decades, the project area has experienced extensive development. Increased impervious cover, altered runoff patterns, and changes in land use have reduced the natural habitat originally targeted for preservation, diminishing the ecological benefit of the structures.

At present, the weirs no longer function as intended and have introduced new challenges to the hydraulic system. Flow is bypassing the structures, routing around their ends instead of passing directly over them. This diversion has created localized erosion, degrading the natural banks and undermining sections of the weir foundation. If unaddressed, the erosion threatens long-term channel stability and accelerates deterioration of the structures.

Given these conditions, removal of the weirs has been identified as a beneficial measure. Eliminating the hydraulic control points would restore conveyance capacity along Socastee Creek, allowing flows to move more efficiently downstream. This increase in capacity is expected to reduce backwater effects and lower flood elevations that currently impact residential properties upstream. **Figure H-14** and **Figure H-15** illustrate the location of the existing weirs and the anticipated reduction in flood elevations resulting from their removal.

H.5.2.1 Construction Sequence

The weir removal would follow a sequenced approach designed to minimize environmental disturbance, maintain safe site access, and protect adjacent infrastructure. The major construction steps are anticipated to include:

Pre-Construction Preparations

- Conduct site surveys and establish staging areas outside of the floodway to minimize impacts.
- Install temporary access roads or mats as needed to reach both upstream and downstream work zones.
- Implement erosion and sediment control measures, such as silt fencing, turbidity curtains, and temporary cofferdams, to prevent sediment migration downstream during demolition activities.

Water Management and Flow Diversion

- A temporary bypass channel or pump-around system may be installed to reroute flow away from the demolition area, depending on streamflow conditions at the time of construction.

- Low-flow cofferdams or sheet pile barriers would isolate work areas around each weir, creating dry conditions for safe removal.

Demolition of Weir Structures

- Riprap armoring on the upstream and downstream faces would be carefully removed and stockpiled for potential reuse in stabilization.
- Concrete sections of the weir would be saw-cut or broken down mechanically using excavators with hydraulic hammers.
- Sheet piles would be extracted where feasible or cut down below grade to eliminate obstructions to flow.
- All debris would be transported off-site for disposal or recycling in compliance with environmental regulations.

Channel Stabilization and Restoration

- Following demolition, the channel banks and bed would be graded to tie smoothly into adjacent creek sections, ensuring a stable transition.
- Suitable riprap or native stone would be reinstalled in targeted locations to provide scour protection at the new channel alignment.
- Where appropriate, bioengineering techniques (such as live staking or coir logs) may be employed to stabilize banks and encourage vegetative regrowth.

Site Restoration and Demobilization

- Disturbed areas would be backfilled, graded, and reseeded with native species to restore vegetation and minimize erosion.
- All temporary access roads, cofferdams, and erosion controls would be removed once the site is stabilized.
- A final inspection would confirm proper conveyance, bank stability, and compliance with environmental commitments.

The removal of the existing weirs is expected to provide multiple benefits. Hydraulically, conveyance along Socastee Creek will be restored, reducing upstream flood elevations during high-flow events. Structurally, long-term maintenance concerns associated with deteriorating weir infrastructure will be eliminated. Environmentally, the restored channel will promote more natural flow patterns and sediment transport, while stabilization measures will protect against future erosion. Overall, the measure represents a balance of flood risk management and ecological restoration within the project corridor.

H.6 Residual Risk

Residual risk remains high for the proposed weir removal and cross drains in the Waccamaw FRM project due to the region’s flat topography and extensive floodplain area. The Waccamaw River Basin is characterized by broad, low-relief terrain with minimal elevation change over long distances, which results in slow drainage and long periods of inundation following storm events. In such low-gradient systems, hydraulic interventions such as cross drains or weir removal provide only limited benefit, as there is insufficient slope to generate meaningful redistribution of flow. While these measures may improve localized connectivity and reduce ponding in specific areas, their impact is quickly overwhelmed during large-scale flood events when the entire floodplain is activated.

This challenge is not unique to the region. The Tar-Pamlico and Neuse River feasibility studies in North Carolina faced similar issues with flat, coastal-plain terrain, where large-scale channel modifications or structural conveyance improvements failed to demonstrate system-wide reductions in flood risk. Instead, those studies emphasized the importance of nonstructural strategies, such as floodproofing, elevation of structures, and buyouts, along with nature-based solutions to restore floodplain storage and connectivity. Likewise, the Lumber River feasibility study encountered limitations in applying conventional structural measures due to minimal topographic relief and the presence of vast wetland complexes, where even modest structural changes produced negligible improvements in water surface elevation.

For the Waccamaw River FRM project, the residual risk is compounded by the sensitivity of such flat systems to small changes in elevation. Even a few tenths of a foot can significantly alter flow distribution and inundation patterns across the broad floodplain. This sensitivity increases uncertainty in hydrologic and hydraulic modeling results and complicates the evaluation of potential structural solutions. Larger interventions, such as floodwalls and floodgates, were also tested during the alternative’s evaluation. While these provided localized reductions in WSE, they also induced adverse effects by raising water levels at other nearby structures due to backwater effects and altered flow pathways, thereby limiting their feasibility and system-wide benefit.

As a result, structural interventions alone are unlikely to provide significant flood risk reduction in the Waccamaw Basin. The project must recognize that residual risks will remain due to the region’s flat topography, hydrologic sensitivity, and vast floodplain storage. These risks are consistent with lessons learned from other regional feasibility studies and are further emphasized under the changing conditions criteria outlined in ECB 2018-14. Additional detail on these considerations is provided in Appendix A2.

H.7 Induced Flooding H&H Analysis

The induced flooding evaluation documents the hydraulic modeling considerations applied in the Waccamaw River Flood Risk Management (FRM) Study and characterizes the uncertainties associated with modeled water surface elevations (WSE), flood durations, and flood frequencies. The purpose of this assessment is to support interpretation of hydraulic differences between without-project (FWOP) and with-project (FWP) conditions and to inform evaluation of potential project-induced effects.

The Waccamaw River FRM study requires a reliable and updated hydraulic analysis to evaluate both the impacts and benefits of proposed flood risk management measures. A HEC-RAS 2D model was selected because it provides robust simulation of floodplain hydraulics, including flow distribution, storage interactions, and hydraulic connectivity across the low-gradient riverine system. The modeling supports floodplain mapping, impact assessment, benefit-cost analysis, engineering design considerations, and real estate evaluations. Particular attention was given to potential induced flooding effects in unprotected areas, where changes in flood elevations, durations, or flow patterns could influence flood risk.

To characterize flood durations and hydrograph timing, synthetic hydrographs were developed at key tributary inflows. Historic flood events, including storms from the 2015, 2016, and 2018 hurricane years, were used to shape representative hydrographs and were calibrated against USGS peak flow frequency estimates to maintain consistency with regional hydrology.

The model incorporated best available terrain data, including high-resolution LiDAR and surveyed channel geometry. Land cover classifications derived from the National Land Cover Database (NLCD), supplemented by aerial imagery and local datasets, were translated into Manning's roughness coefficients within the 2D computational mesh. Routing hydrographs across this terrain produced WSE, depth, velocity, and flood duration estimates for existing (FWOP) conditions, establishing the baseline for comparison.

Proposed FRM measures, including relief bridges, weir removals, and associated structural or mitigation features, were incorporated into with-project (FWP) simulations. Multiple design iterations were performed to refine layouts and minimize adverse hydraulic consequences, including potential induced flooding and loss of floodplain storage. Terrain modifications representing project features included smoothing and transition grading consistent with realistic construction practices.

Uncertainty in the hydraulic modeling arises from terrain accuracy, survey precision, hydrologic inputs, roughness parameterization, boundary condition specification, and computational mesh resolution. These inherent uncertainties were considered when evaluating modeled differences between FWOP and FWP conditions.

Comparative analyses were performed to quantify changes in water surface elevations, flood durations, and hydraulic parameters between without-project (FWOP) and with-project (FWP) conditions for all evaluated flood events, including the 50%, 20%, 10%, 4%, 2%, 1%, 0.5%, and 0.2% annual exceedance probabilities. This assessment included explicit evaluation of the 0.5% AEP event to ensure induced flooding considerations address the full range of reasonably foreseeable flood conditions.

Modeled differences were evaluated based on magnitude, spatial extent, and potential influence on flood risk. While localized variations in hydraulic results are inherent to numerical modeling and reflect uncertainties associated with terrain accuracy, hydrologic inputs, roughness parameterization, and computational resolution, all predicted differences between FWOP and FWP conditions were retained and assessed. Following detailed structure-level review of modeled water surface elevations and flood durations, no properties were identified that exhibited an increase in flood risk at the structure level under with-project conditions.

Uncertainty in model results was considered qualitatively to inform interpretation of hydraulic outputs but was not used to screen, adjust, or dismiss predicted project effects. This approach ensures that induced flooding evaluations remain consistent, transparent, and aligned with USACE guidance.

From a real estate and takings perspective, modeled differences between FWOP and FWP conditions were evaluated in accordance with applicable policy and legal standards.

I FLOOD RISK MANAGEMENT UNCERTAINTY

I.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.*

I.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 3 and 4 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 I-1** shows equivalent record lengths for selected rivers in the Waccamaw basin.

Table I-1. Equivalent Record Lengths

Hydrologic Study Model	Equivalent Record Length (yr)
Waccamaw River Mainstem	50
Buck Creek	30
Pee Dee River	30

Stage-discharge uncertainty was evaluated independently from hydrologic uncertainty. Variability in hydraulic parameters, including Manning's roughness coefficients, was assessed consistent with EM 1110-2-1619 guidance. Standard deviations associated with roughness coefficients were estimated using reference relationships and applied within sensitivity analyses to evaluate the influence of hydraulic parameter variability on computed water surface elevations. These analyses produced bounding water surface profiles used to quantify hydraulic model uncertainty at the reach scale. Standard deviations of hydraulic roughness coefficients used in the study models were determined from reference Figure 5-4 in **Figure I-1** below.

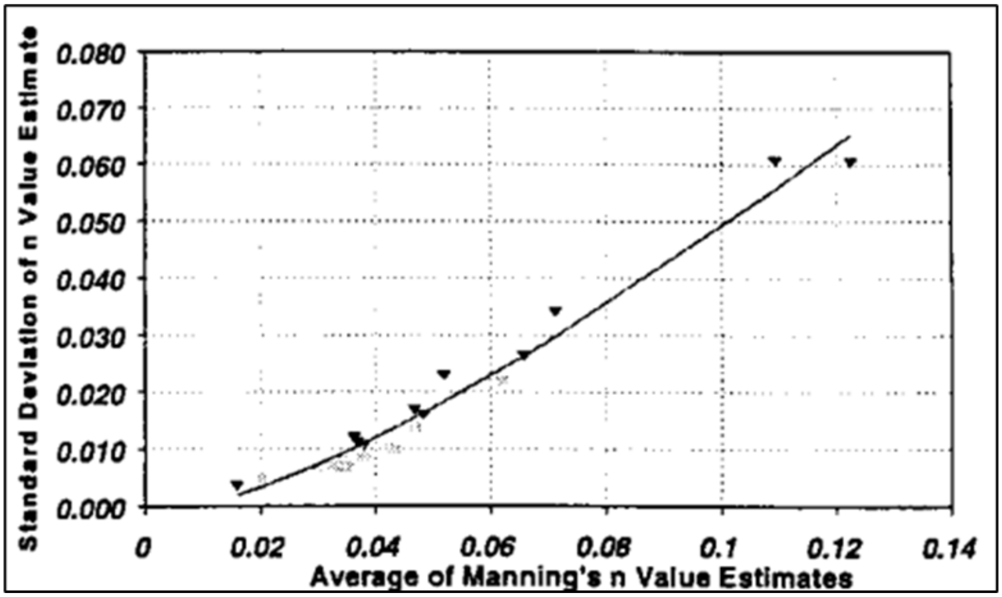


Figure I-1. EM 1110-2-1619 Figure 5-4

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 I-1**. 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.

Model uncertainty (S_{model}) was computed for each HEC-RAS reach using EM 1110-2-1619 procedures. Reach-level results were subsequently aggregated to align with HEC-FDA damage reaches. Natural variability

($S_{natural}$) was estimated using available USGS streamflow gage data where present. For reaches lacking representative gages, $S_{natural}$ was estimated using EM 1110-2-1619-recommended methods appropriate for data-limited conditions. Total stage uncertainty ((S_{total}) was calculated as the combination of S_{model} and $S_{natural}$.

The 1% AEP (0.01) event served as the reference condition for uncertainty characterization. For more frequent events, uncertainty was adjusted proportionally to reflect reduced hydraulic variability at lower flow magnitudes. For less frequent events, uncertainty was held constant to avoid overstating precision beyond the calibration and data support range.

The resulting uncertainty estimates were incorporated into the HEC-FDA analysis to represent variability in stage-frequency relationships. Total uncertainty values for each HEC-FDA reach at the 1% AEP event are presented in **Figure I-2** through **Figure I-5**.

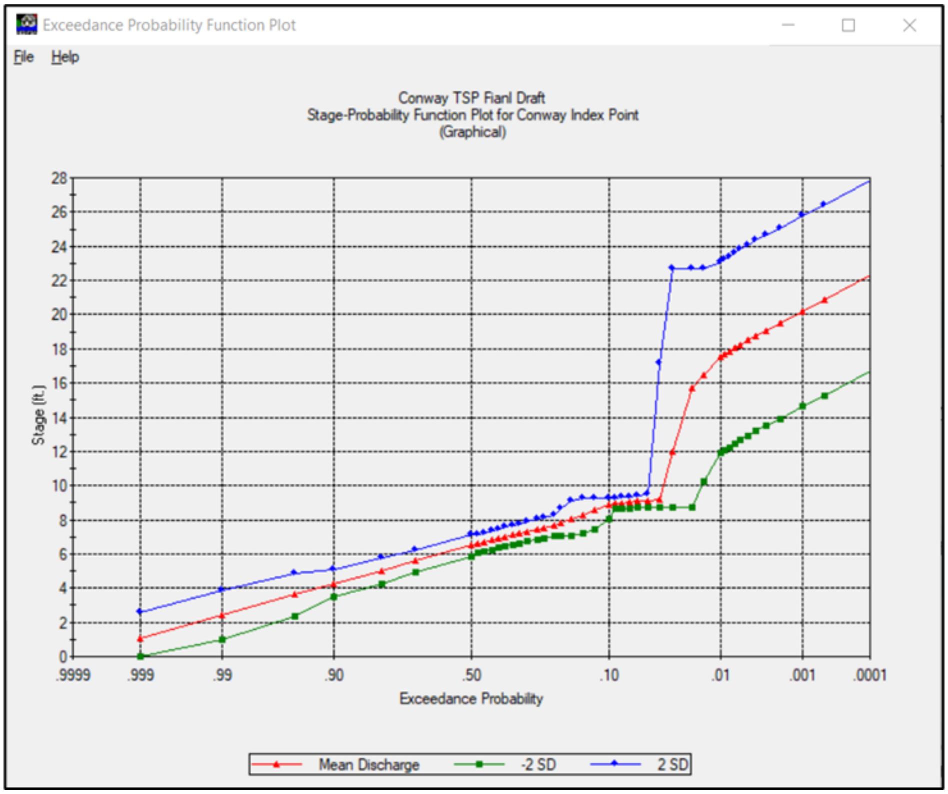


Figure I-2. Stage Probability function for Conway

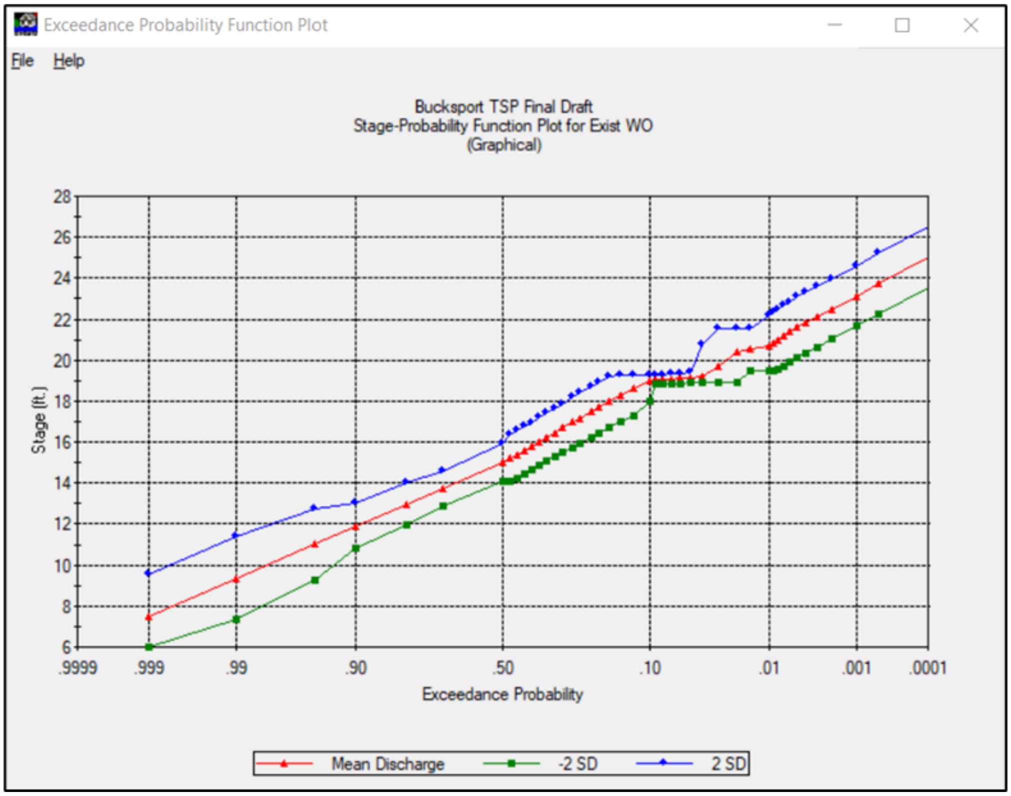


Figure I-3. Stage Probability function for Bucksport

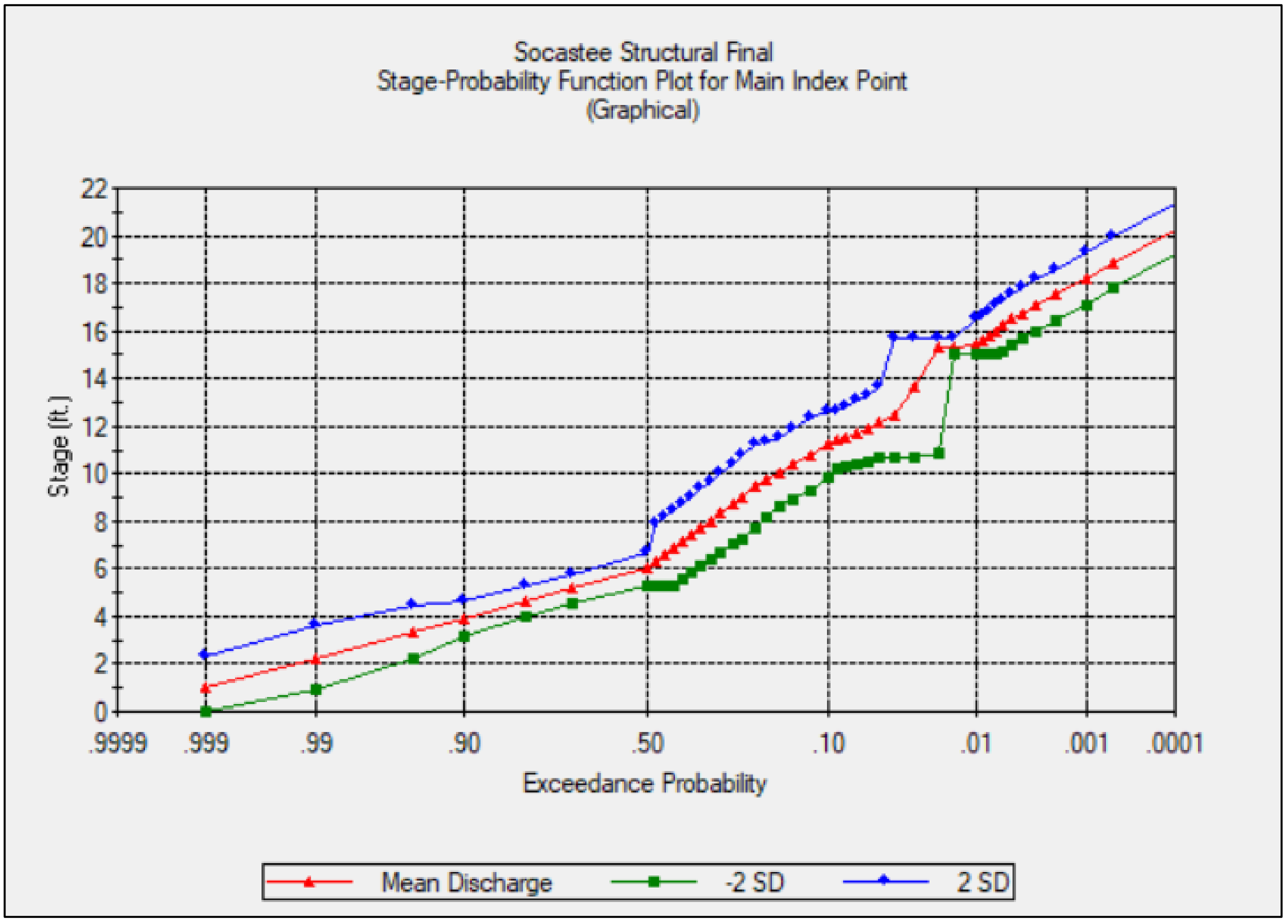


Figure I-4. Stage Probability function for Socastee

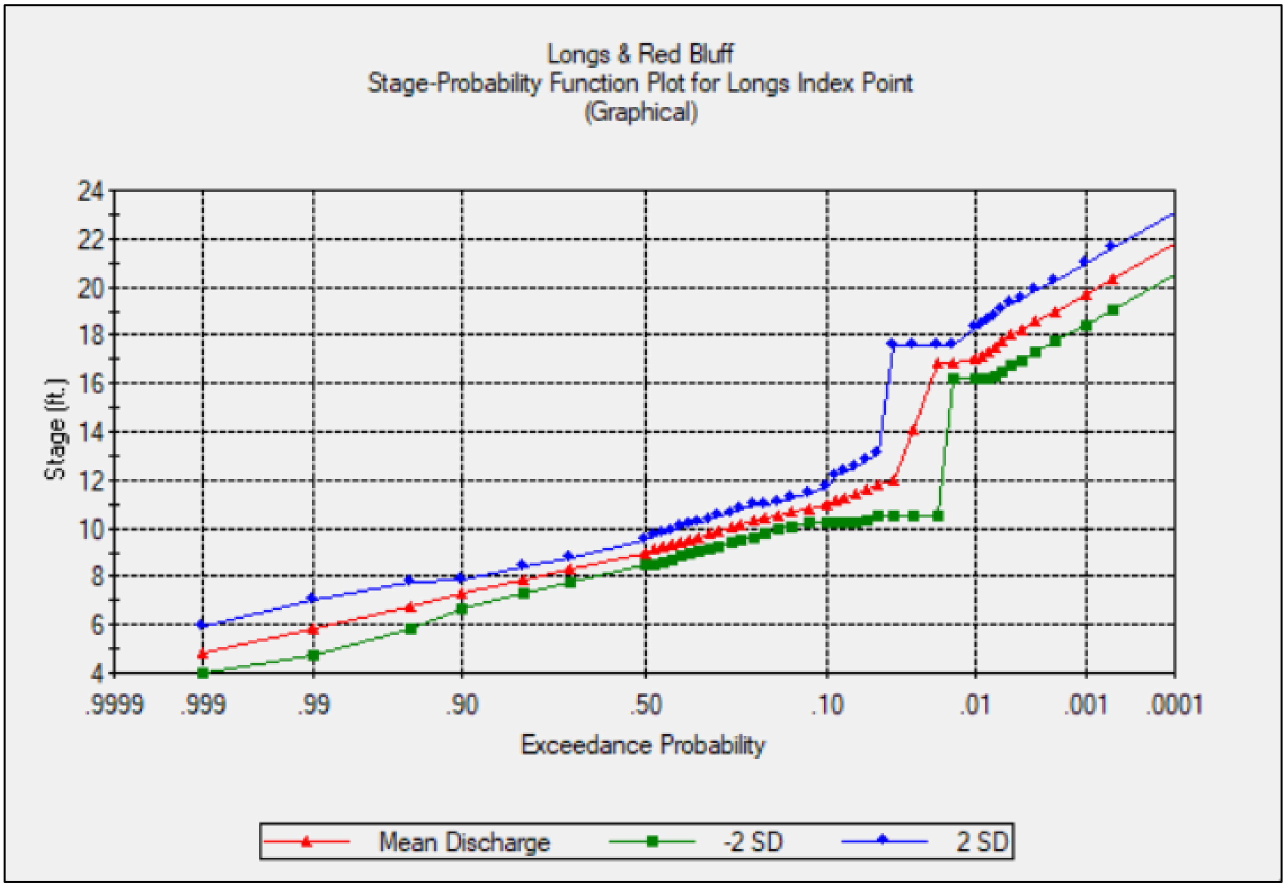


Figure I-5. Stage Probability function for Longs/Red Bluff

Table I-2. Total Uncertainty for Baseline 0.01-AEP Event.

Total Uncertainty for Baseline 0.01-AEP Event				
HEC-FDA Reach	Longs/ Red Bluff	Conway	Socastee	Bucksport
1	0.8	0.8	0.8	1
2	0.6	1	1	0.8
3	0.6	0.9	1	1
4	0.6	0.8	0.8	0.8
5	0.6	0.8	0.8	0.8

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

J DESIGN MATURITY

The hydrology and hydraulics (H&H) engineering development documented in this appendix satisfies the preliminary design (35%) maturity benchmarks described in ECB 2023-9, Civil Works Design Milestone Checklists. The H&H analyses include completion of initial hydraulic and hydrologic modeling necessary to support feasibility-level evaluations, consistent with the checklist requirement that “Initial Hydraulic and Hydrology (H&H) analysis [be] complete and shared with the PDT such that geotechnical and structural analysis can be performed” and that hydraulic loading conditions be defined for associated disciplines.

Specifically, the study includes fully developed HEC-HMS hydrologic modeling and HEC-RAS 2D hydraulic modeling, supported by assembled terrain, hydrologic, boundary condition, and land cover datasets. Model development, calibration to historic flood events, stability verification, and documentation of assumptions and uncertainties have been completed and coordinated with the PDT. Hydraulic outputs required for plan formulation, economic evaluation, engineering assessment, and real estate considerations have been generated and applied. These elements meet the intent of preliminary design maturity by establishing design-driving hydraulic conditions and providing a technical basis sufficient to support the Class 3 cost estimate.

The Engineering Design Maturity Assessment Form has been completed to formally document compliance with the 35% design maturity requirements of ECB 2023-9.

K 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 and modeled independently. Overall, the implementations of the evaluated structural measures lowered the water surface elevations in some locations near and around the weir locations and up and downstream of the relief bridges. There was a reduction of water in homes, but most were not fully removed from the inundation area.

Analysis of the hydraulic modeling was conducted to analyze the difference in water surface elevation. The results depict the differences in water depths at each structure for both the Relief Bridges (cross drains) in Conway and the Weir removal in Socastee. After analysis and refining the FWOP model gridding to reflect the FWP gridding there were no increases in water surface elevation at the structure level.

The literature review indicates that air temperatures are projected to rise by 5-10 degrees Fahrenheit by the second half of the 21st century, but future changes in average annual precipitation and streamflow remain uncertain. There is evidence that flooding risk in the Waccamaw River Watershed will increase due to more frequent and intense rainfall events. Because of increases in extreme rainfall, there is evidence pointing to the potential for reduced project performance towards the later half of the 21st century. Sea level projections show a broad range of increases along the coastline, but impacts are not operationally significant within the vicinity of upstream, proposed project features (See Appendix A-2 for more details).

L REFERENCES

Allen, R. J., Gaetano, A. T. Aeral Reduction Factors for Two Eastern United States Regions with High Rain-Gauge Density. Journal of Hydrologic Engineering. 0.1061/(ASCE)1084-0699(2005)10:4(327). July 2005.

Brommer, D.M., Cerveny, R.S., Balling Jr, R.C. (2007) Characteristics of long-duration precipitation events across the United States. Geophysical Research Letters 34.

Chen G., Tian H., Zhang C., Liu M., Ren W., Zhu W., Chappelka A.H., Prior S.A., Lockaby G.B. (2012) Drought in the Southern United States over the 20th century: Variability and its impacts on terrestrial ecosystem productivity and carbon storage. Climatic Change 114:379-397.

Chow, V.T. 1959. Open Channel Hydraulics, McGraw-Hill, New York, NY

Cook B.I., Smerdon J.E., Seager R., Cook E.R. (2014) Pan-Continental Droughts in North America over the Last Millennium. Journal of Climate 27:383-397.Dai, Z., Amatya, D.M., Sun, G., Trettin, C.C., Li, C., Li, H. (2011) Climate variability and its impact on forest hydrology on South Carolina coastal plain, USA. Atmosphere 2, 330-357.

Duan Q, Schaake J, Andreassian V, Franks S, Goteti G, Gupta HV, Gusev YM, Habets F, Hall A, Hay L, Hogue T, Huang M, Leavesley G, Liang X, Nasonova ON, Noilhan J, Oudin L, Sorooshian S,Wagener T, Wood EF (2006) Model parameter estimation experiment (MOPEX): an overview of science strategy and major results from the second and third workshops. J Hydrol 320:3–17.

England et al., 2017. Guidelines for Determining Flood Flow Frequency Bulletin 17C.

EPA, 2017. Updates to the Demographic and Spatial Allocation Models to Produce Integrated Climate and Land-Use Scenarios (ICLUS). Version 2. Environmental Protection Agency.

Feaster, T.D., Weaver, J.C., Gotvald, A.J., and Kolb, K.R. 2018. Preliminary peak stage and streamflow data at selected U.S. Geological Survey streamgaging stations in North and South Carolina for flooding following Hurricane Florence. September 2018. U.S. Geological Survey Open-File Report 2018–11

Gao, Y., J. S. Fu, J. B. Drake, Y. Liu and J. F. Lamarque (2012). "Projected changes of extreme weather events in the eastern United States based on a high resolution climate modeling system." Environmental Research Letters 7(4).

Grundstein A, Dowd J (2011) Trends in extreme apparent temperatures over the United States, 1949-2010. Journal of Applied Meteorology and Climatology 50:1650- 1653.

Horry County (2021), Horry County Flood Resilience Master Plan Summary Report, Sherwood Design Engineers, September 2021.

lark, D., Finch, J., Ihnatolya, B. Crabtree Creek Wake County, North Carolina. Revisions to Hydrologic and Hydraulic Analysis. Project Number: ARC-0900. EcoEngineering. June 2011.

ICLUS, 2017b. Integrated Climate and Land-Use Scenarios. ICLUS v2.1 Percent Impervious Surface Projections (A1, A2, B1, and B2). Environmental Protection Agency

Irizarry-Ortiz, M.M., Obeysekera, J., Park, J., Trimble, P., Barnes, J., Park-Said, W., Gadzinski, E. (2013) Historical trends in Florida temperature and precipitation. Hydrological Processes 27, 2225-2246.

Jayakody P, Parajuli PB, Cathcart TP (2013) Impacts of climate variability on water quality with best management practices in sub-tropical climate of USA. Hydrological Processes.

Kalra, A., T. C. Piechota, R. Davies and G. A. Tootle (2008). "Changes in U.S. streamflow and Western U.S. snowpack." Journal of Hydrologic Engineering 13(3): 156-163.

Kunkel KE, Liang X-Z, Zhu J (2010) Regional climate model projections and uncertainties of U.S. summer heat waves. Journal of Climate 23:4447-4458.

Li W, Li L, Fu R, Deng Y, Wang H (2011) Changes to the North Atlantic subtropical high and its role in the intensification of summer rainfall variability in the southeastern United States. Journal of Climate 24:1499-1506.

Melillo, M., Terese (T.C.) Richmond, and G. W. Yohe, Eds., U.S. Global Change Research Program, 396-417. doi:10.7930/J0NP22CB.

Misra, V., Michael, J.P., Boyles, R., Chassignet, E.P., Griffin, M., O'Brien, J.J. (2012) Reconciling the spatial distribution of the surface temperature trends in the Southeastern United States. Journal of Climate 25, 3610-3618.

Multi-Resolution Land Characteristics Consortium (MRLC). National Landcover Database (2016). Retrieved from https://www.mrlc.gov/.

NACSE, PRISM Climate data. 30-Year Normal Precipitation: Annual dataset. http://prism.oregonstate.edu/normals/. Accessed: 2021.

NOAA, 2013. Atlas 14: Precipitation-Frequency Atlas of the United States. Vol. 9 Version 2.0: Southeastern States. National Oceanic and Atmospheric Administration.

NOAA, 2017. NOAA Office for Coastal Management. Method Description Detailed Method for Modeling Sea Level Rise Inundation. January 2017. https://coast.noaa.gov/data/digitalcoast/pdf/slr-inundation-methods.pdf

NRCS, 1986. Urban Hydrology for Small Watersheds. Technical Report 55. Natural Resources Conservation Service. June 1986.

Palecki MA, Angel JR, Hollinger SE (2005) Storm precipitation in the United States. Part I: Meteorological characteristics. Journal of Applied Meteorology 44:933-946.

Patterson, L.A., Lutz, B., Doyle, M.W. (2012) Streamflow Changes in the South Atlantic, United States During the Mid- and Late 20th Century. Journal of the American Water Resources Association 48, 1126-1138.

Pryor SC, Howe JA, Kunkel KE (2009) How spatially coherent and statistically robust are temporal changes in extreme precipitation in the contiguous USA? International Journal of Climatology 29:31-45.

Qi, S., Sun, G., Wang, Y., McNulty, S.G., Myers, J.A.M. (2009) Streamflow response to climate and landuse changes in a coastal watershed in North Carolina. Transactions of the ASABE 52, 739-749.

Russell S, Kenning B, Sunnell G (1979) Journal of Hydraulics Division Estimating Design Flows for Urban Drainage. ASCE, Vol. 106, No. HY1, January, 1979.

SCACIS, Applied Climate Information System. U.S. Climate Resilience Toolkit. http://scacis.rcc-acis.org/. Accessed: 2021.

SCDNR (2015), South Carolina Flooding 2015. https://safe.menlosecurity.com/http://www.dnr.sc.gov/flood2015

SCDNR (2016), Hurricane Matthew 2016. https://safe.menlosecurity.com/http://www.dnr.sc.gov/matthew2016

SCDNR (2018), Hurricane Florence 2018. https://safe.menlosecurity.com/http://www.dnr.sc.gov/florence2018

SCDOT Design Manual, January 2019, Geology of the Coastal Plain of South Carolina, Geological Survey Bulletin 867. 1938, and USACE Waccamaw River Basin North Carolina and South Carolina, September 1981.

SCDOT. Best Available Elevation Data Summary. https://connect.ncdot.gov/resources/Photogrammetry/Photogrammetry%20Documents/Best%20Available%20Elevation%20Data%20Summary_210129.pdf.

South Carolina Emergency Management. QL2/QL1 LiDAR Collection. 2018. https://sdd.nc.gov/SDD/docs/LidarSummary.pdf.

SC State Climatology Office, Overview of South Carolina's Climate and Hazards, https://safe.menlosecurity.com/http://dnr.sc.gov/water/climate/sco

Scherer M, Diffenbaugh N (2014) Transient twenty-first century changes in daily-scale temperature extremes in the United States. Climate Dynamics 42:1383-1404.

Schwartz MD, Ault TR, Betancourt JL (2013) Spring onset variations and trends in the continental United States: Past and regional assessment using temperature- based indices. International Journal of Climatology 33:2917-2922.

Small D, Islam S, Vogel RM (2006) Trends in precipitation and streamflow in the eastern U.S.: Paradox or perception? Geophysical Research Letters 33. Tebaldi C (2006) Going To The Extremes: An Intercomparison of Model-Simulated Historical and Future Changes in Extreme Events. 79:185-211.

Thomson AM, Brown RA, Rosenberg NJ, Srinivasan R, Izaurralde RC (2005) Climote change impacts for the conterminous USA: An integrated assessment: Part 4: Water resources. Climatic Change 69:67-88.

U.S. Army Corps of Engineers (2015) Corps Water Management System (CWMS) Final Report for the Yadkin River Watershed.

U.S. Army Corps of Engineers, Deadmans Run Section 205 Lincoln, Nebraska. Appendix E: Hydrologic Analysis & Climate Assessment. USACE Omaha District. March 2018.

U.S. Army Corps of Engineers, Engineering and Construction Bulletin 2018-14 (rev1) (Guidance for Incorporating Climote Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects). Issued 10 September 2020.

U.S. Army Corps of Engineers, Engineering Manual (EM) 1110-2-1619 (Risk-based analysis for flood damage reduction studies). Issued 1996

U.S. Army Corps of Engineers, Engineering Pamphlet (EP) 1110-2-1 (Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation). Issued 2019.

U.S. Army Corps of Engineers, Engineering Regulation (ER) 1105-2-101 (Risk Assessment for Flood Risk Management Studies). Issued 2019.

U.S. Army Corps of Engineers, Engineering Regulation (ER) 1110-2-1150 (Engineering and Design for Civil Works Projects). Issued 1999.

U.S. Army Corps of Engineers, Engineering Regulation (ER) 1165-2-21. (Flood Damage Reduction Measures in Urban Areas). Issued 1980.

U.S. Army Corps of Engineers, Engineering Regulation (ER) 1100-2-8162. (Incorporating Sea Level Change in Civil Works Programs). Issued 2013.

U.S. Army Corps of Engineers, Recent US Climote Change and Hydrology Literature Applicable to US Army Corps of Engineers Missions, South Atlantic-Gulf Region 03. January 2015.

U.S. Army Corps of Engineers, Time Series Toolbox, Trend Analysis and Nonstationarity Detection. 2018.

nU.S. Army Corps of Engineers, Sea Level Analysis Tool (October 2024).

U.S. Army Corps of Engineers, Valley Creek Flood Risk Management Study. Appendix A: Engineering. USACE NWK, SAM. 2021.

USGS, Preliminary peak stage and streamflow data at selected stream gaging stations in North Carolina and South Carolina for flooding following Hurricane Matthew, October 2016, https://safe.menlosecurity.com/https://pubs.usgs.gov/publication/ofr20161205

USGS Scientific Investigations Report (SIR) 2009-5158. Magnitude and Frequency of Rural Floods in the Southeastern United States, Through 2006: Volume 2, North Carolina. Reston, VA 2008. (http://pubs.usgs.gov/sir/2009/5158/pdf/sir2009- 5158.pdf)

USGS Advisory Committee on Water Information, “Guidelines for Determining Flood Flow Frequency – Bulletin 17C”, 2018.

USGS, Flood Event Viewer. Hurricane Florence High-Water Marks. https://stn.wim.usgs.gov/FEV/#FlorenceSep2018. Accessed: 2021

USGS, Flood Event Viewer. Hurricane Matthew High-Water Marks. https://stn.wim.usgs.gov/FEV/#MatthewOctober2016. Accessed: 2021

USGS. StreamStats Program. 2019. http://streamstats.usgs.gov (accessed on 10 May 2021).

Villarini G, Smith HA, Vecchi GA (2013). Changing Frequency of Heavy Rainfall over the Central United States. Journal of Climate 26:351-357.

Walsh J, Wuebbles D, Hayhoe K, Kossin J, Kunkel KE, Stephens G, Thorne P, Vose RS, Wehner MF, Willis J, Anderson D, Kharin V, Knutson T, Landerer F, Lenton T, Kennedy J, Somerville R (2014) Appendix 3: Climate Science Supplement. in Melillo JM, Richmond TC, Yohe GW (eds.) Climate Change Impacts in the United States: The Third National Climate Assessment. U.S. Global Change Research Program, pp. 735-789.

Wang, R., Kalin, L., Kuang, W., Tian, H. (2013b) Individual and combined effects of land use/cover and climate change on Wolf Bay watershed streamflow in southern Alabama. Hydrological Processes, in press.

Wang H, Killick R, Fu X (2013) Distributional change of monthly precipitation due to climate change: Comprehensive examination of dataset in southeastern United States. Hydrological Processes, in press.

Wang H, Schubert S, Suarez M, Chen J, Hoerling M, Kumar A, Pegion P (2009) Attribution of the seasonality and regionality in climate trends over the United States during 1950-2000. Journal of Climate 22:2571-2590.

Wang J, Zhang X (2008) Downscaling and projection of winter extreme daily precipitation over North America. Journal of Climate 21:923-937.

Weaver, J.C., Feaster, T.D., and Robbins, J.C., 2016, Preliminary peak stage and streamflow data at selected stream-gaging stations in North Carolina and South Carolina for flooding following Hurricane Matthew, October 2016: U.S. Geological Survey Open-File Report 2016 –1205, 38 p., https://doi.org/10.3133/ofr20161205.

Westby, R.M., Lee, Y.-Y., Black, R.X. (2013) Anomalous temperature regimes during the cool season: Long-term trends, low-frequency mode modulation, and representation in CMIP5 simulations. Journal of Climate 26, 9061-9076.

Xu, X., Liu, W., Rafique, R., Wang, K. (2013) Revisiting Continental U.S. Hydrologic Change in the Latter Half of the 20th Century. Water resources management 27, 4337-4348.