Neuse River Basin Flood Risk Management Feasibility Study

Technical Report

Appendix A. Hydrology and Hydraulics

US Army Corps
of Engineers

January 2023

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1 Introduction

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

1.1 Vertical Datum

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

2 Basin Overview

2.1 Location

The Neuse River is formed by the confluence of the Eno and Flat Rivers about 8 miles north of the City of Durham, NC (USACE, 1960). The basin has a total drainage area of approximately 6,200 square miles and is considered in this study to extend from Orange and Person Counties at its headwater to Pamlico Sound and Carteret County at its outlet. The Neuse River reaches tidal waters near State Highway 43, upstream of the City of New Bern, NC. It lies entirely within the boundaries of North Carolina. The Neuse River basin is roughly 180 miles long and ranges in width from 35 to 45 miles through most of its length. The basin is fully or partial contained within 18 counties. The total Neuse River basin makes up about 11-percent of the area of North Carolina (USGS, 1957). A map of the Neuse River basin is shown in Figure 1.

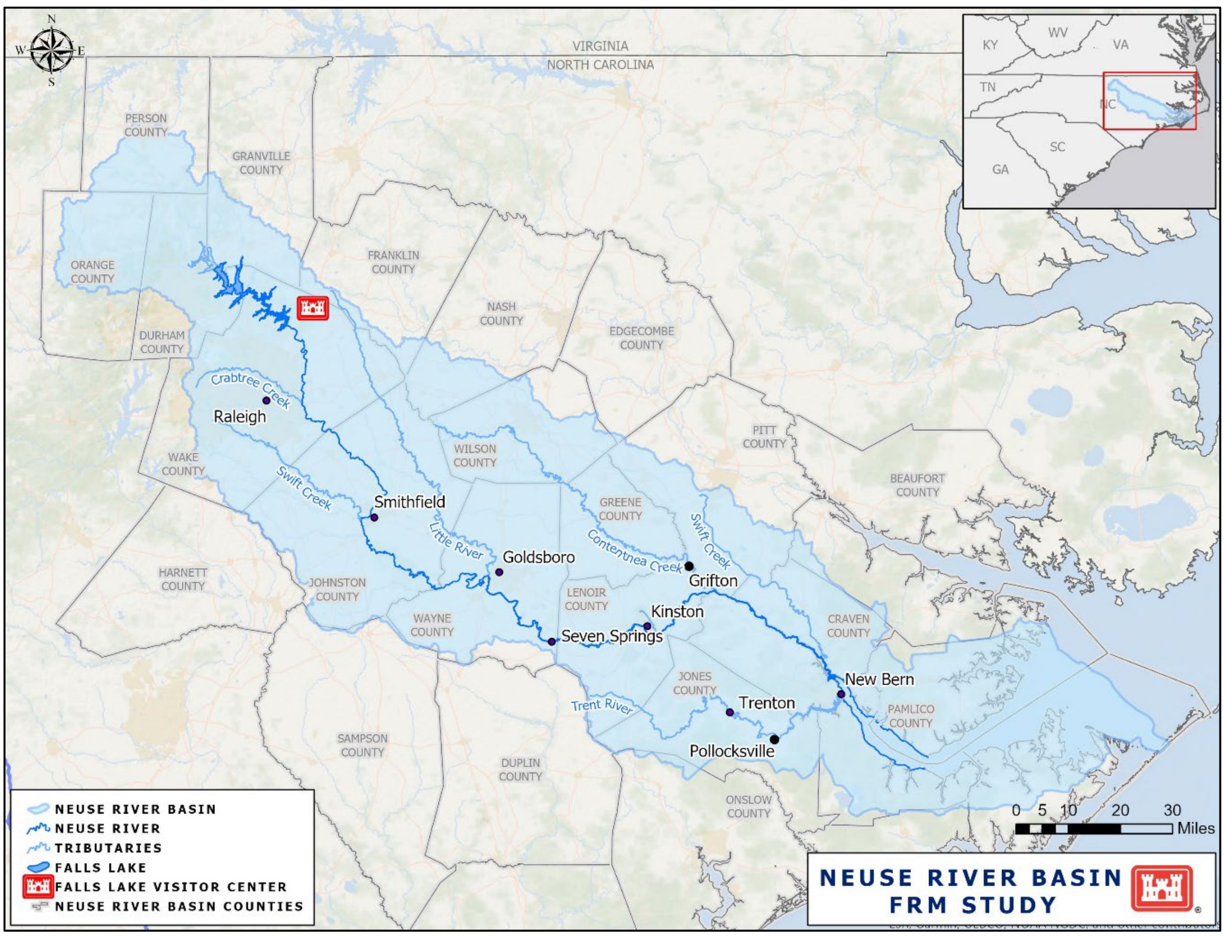

Figure 1. Neuse River Basin Study Area

2.2 Flood Risk Management Infrastructure

An upper portion of roughly 1/6th of the Neuse River basin's total drainage area is captured by the Falls Lake Dam federal project. Constructed in the early 1980s, this federal flood risk management infrastructure site consists of an earth embankment dam and ~19 square mile reservoir that receives inflow from roughly 770 square miles of contributing drainage area. The project serves the primary mission of flood risk management. It also supports water supply, water quality, and recreation. The dam is located north of the City of Raleigh and is considered the beginning of the Neuse River mainstem. Flow is regulated as it is released from the dam. Below Falls Lake, the river flows southeast for about 180 miles, past the Cities of Smithfield, Goldsboro, Kinston, and New Bern. Pertinent reservoir data for Falls Lake is shown in Figure 2. Falls Lake Dam releases relative to major population centers downstream, including percentage of uncontrolled subbasin area to total basin area, and associated water travel times is listed in Table 1.

Table 1. Falls Releases Relative to Downstream Uncontrolled Drainage Area and Population Centers

Location	Total Drainage Area (sq. mi.)	Uncontrolled Drainage Area downstream of Falls (sq. mi.) (% of Total Area)	Distance Below Falls (river miles)	Water Travel Time from Dam (days)
Falls Dam	770	--	--	--
Clayton	1150	380 (33)	32	0.5 to 0.75
Smithfield	1206	426 (36)	56	0.75 to 1
Goldsboro	2399	1629 (68)	99	3 to 5
Kinston	2692	1922 (71)	144	5 to 10

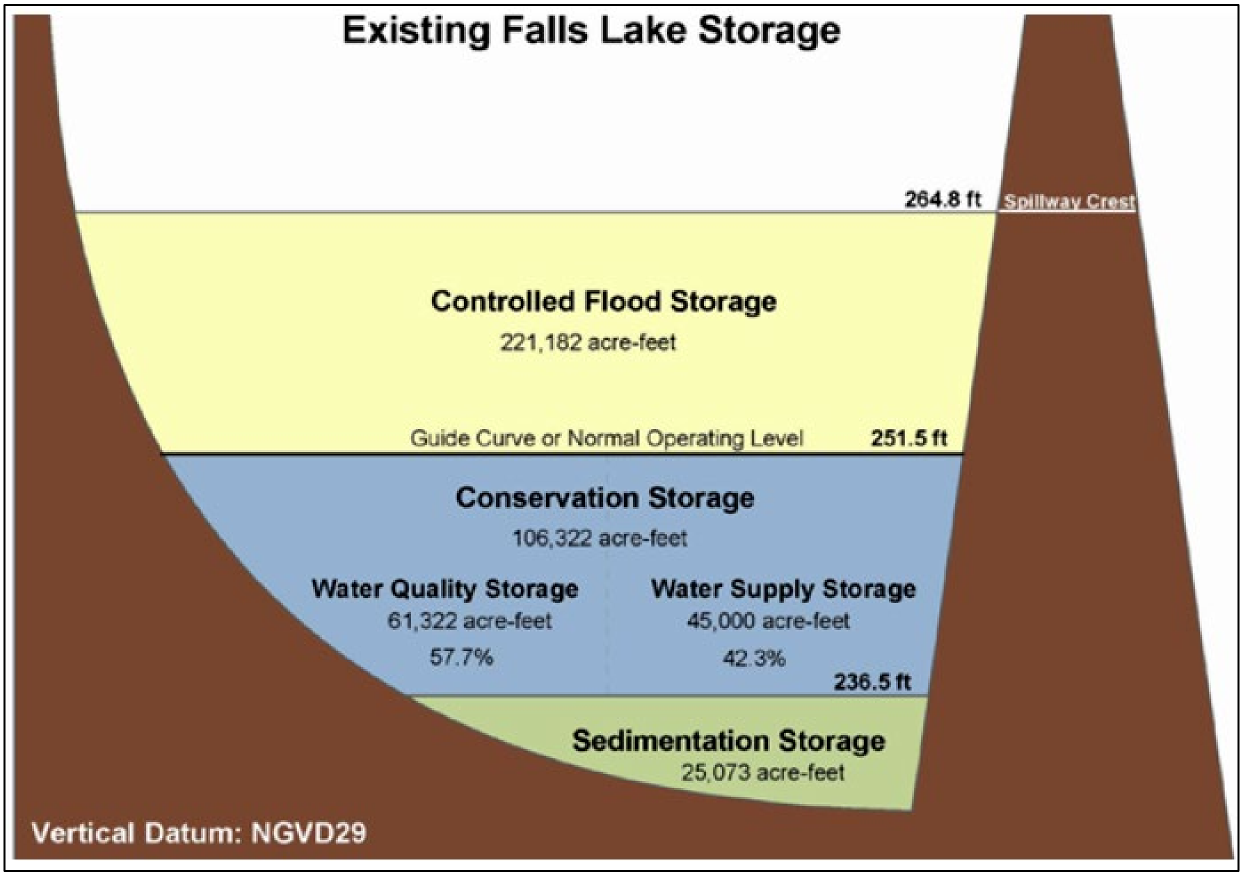

Figure 2. Falls Lake Reservoir Pertinent Data

There are at least 19 additional reservoirs throughout the basin, most in its upper portion, and hundreds of smaller impoundment facilities upstream of Kinston (NCDOT, 2020). At least eighteen of these reservoirs are owned and operated by non-federal entities (Falls Lake is the largest and only federal reservoir) (USACE, 2013). Those reservoirs consist of a wide variety of structures including millponds, beaver impoundments, water supply reservoirs and flood storage structures, all of which are typical of the Piedmont region of the upper Neuse River basin. Fewer reservoirs are in the lower basin because the Coastal Plain physiographic province generally consists of relatively flat topography underlain by highly pervious sands. Dams with an assigned Hazard Potential Classification (Low, Significant, or High) from the National Inventory of Dams (https://nid.sec.usace.army.mil/) is shown in Figure 3.

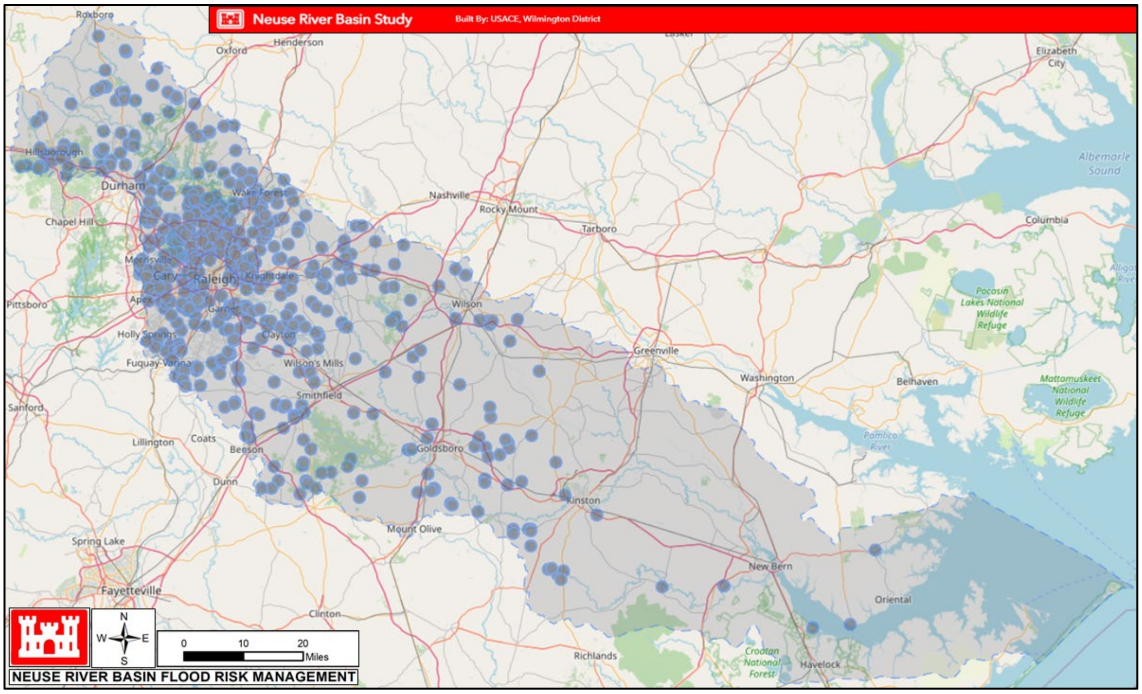

Figure 3. National Inventory of Dams

2.3 Stream Characteristics

The Neuse River basin includes numerous small to moderately sized tributaries that join the Neuse River mainstem throughout its delineation. Major confluences with Neuse are located near Raleigh, Smithfield, Goldsboro, Grifton, and New Bern. Its headwater tributaries rise in the hilly Piedmont section of North Carolina, then flow through a belt, or zone, known as the “Fall Line”, where the streams flatten in slope as they reach the Coastal Plain. Streams in the lower reaches of the Coastal Plain tend to be sluggish in flow, and swamp and marshes are predominant (USACE, 1960). There are almost 3,500 freshwater stream miles in the Neuse River basin (NCDEQ, 2009). A selection of streams contributing to the Neuse River mainstem, and the upper basin are listed in Table 2. The natural dendritic characteristics of the basin are shown in Figure 4.

Table 2. Select Tributaries within the Neuse River Basin (Source: USACE, NCEM, USGS)

Stream	Drainage Area (sq. mi.)

Flat River	184
Eno River	260
Ellerbe Creek	37
Crabtree Creek	145
Walnut Creek	46
Swift Creek	155
Middle Creek	130
Black Creek	95
Mill Creek	170
Falling Creek (Wayne Co)	118
Little River	320
Bear Creek	64
Falling Creek (Lenoir Co)	52
Southwest Creek	68
Mosley Creek	50
Contentnea Creek	1,009
Core Creek	74
Swift Creek (Craven Co)	240
Bachelor Creek	62
Trent River	241
Neuse River	3,200

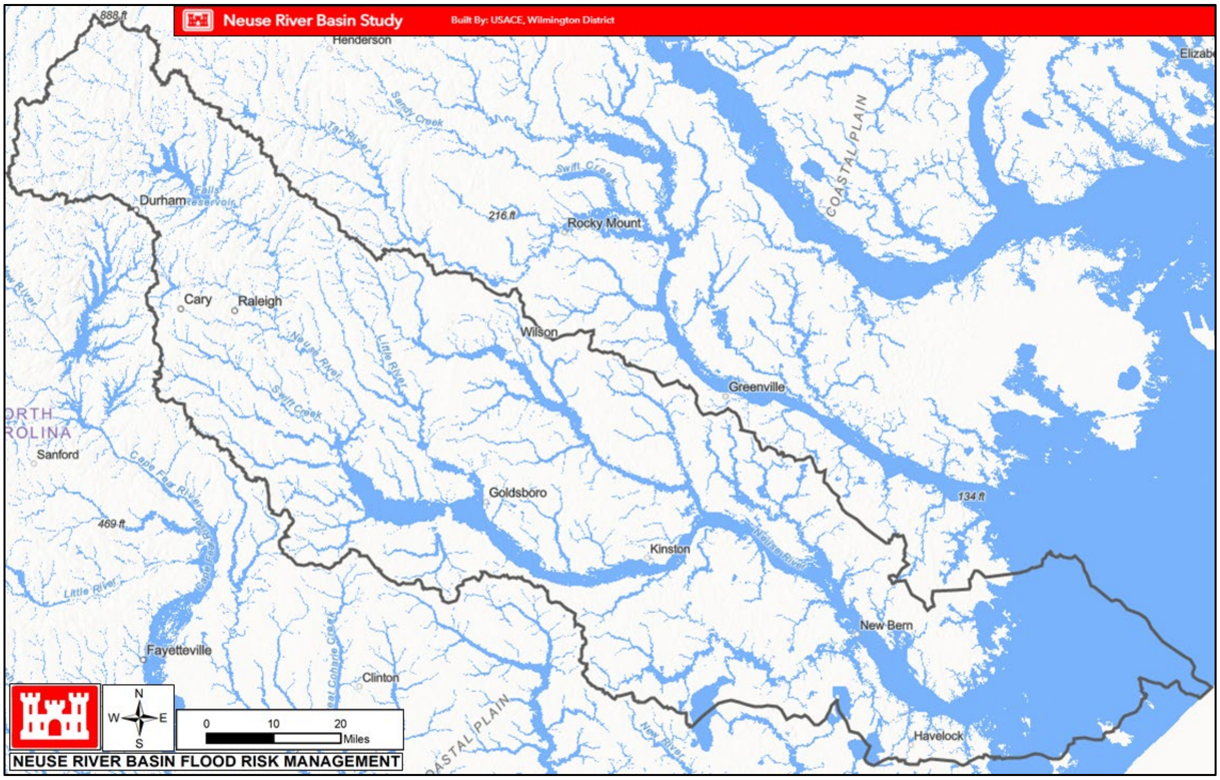

Figure 4. Dendritic Flow paths in the Neuse River Basin

2.4 Land Cover

The most current (2019) National Land Cover Database (NLCD) for the Neuse River basin is shown in Figure 5. 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. From the northwest-most region of the basin and extending southeast to Clayton, land cover can be characterized as highly developed within city limits of Raleigh and surrounding suburban areas, forested land and pasture north of the city, and woody wetlands along the perimeter of Falls Lake. Within the middle reach of the basin, from Smithfield to near Kinston, land is characterized with extensive cultivated crops, scattered evergreen forest and woody wetlands, and developed areas within city limits. South of Kinston, within the lower reach of the basin, cultivated crops are relatively decreased in volume, woody wetlands are greatly increased in volume, development surrounds the New Bern area, and open water is associated with Pamlico Sound and the mouth of the Neuse River. Percentages of land cover type over the entire Neuse River basin are listed in Table 3.

Table 3. NLCD 2019 Land Cover Type Breakdown within the Neuse River Basin

Land Cover Type	Percent of Total Neuse River Basin Area
Open Water	2.8
Developed, Open Space	7.0
Developed, Low Intensity	2.9
Developed, Medium Intensity	1.0
Developed, High Intensity	0.4
Barren Land	0.2
Deciduous Forest	19.4
Evergreen Forest	13.9
Mixed Forest	10.7
Shrub/Scrub	2.6
Grassland/Herbaceuous	2.3
Pasture/Hay	7.8
Cultivated Crops	13.8
Woody Wetlands	14.0
Emergent Herbaceuous Wetlands	1.2

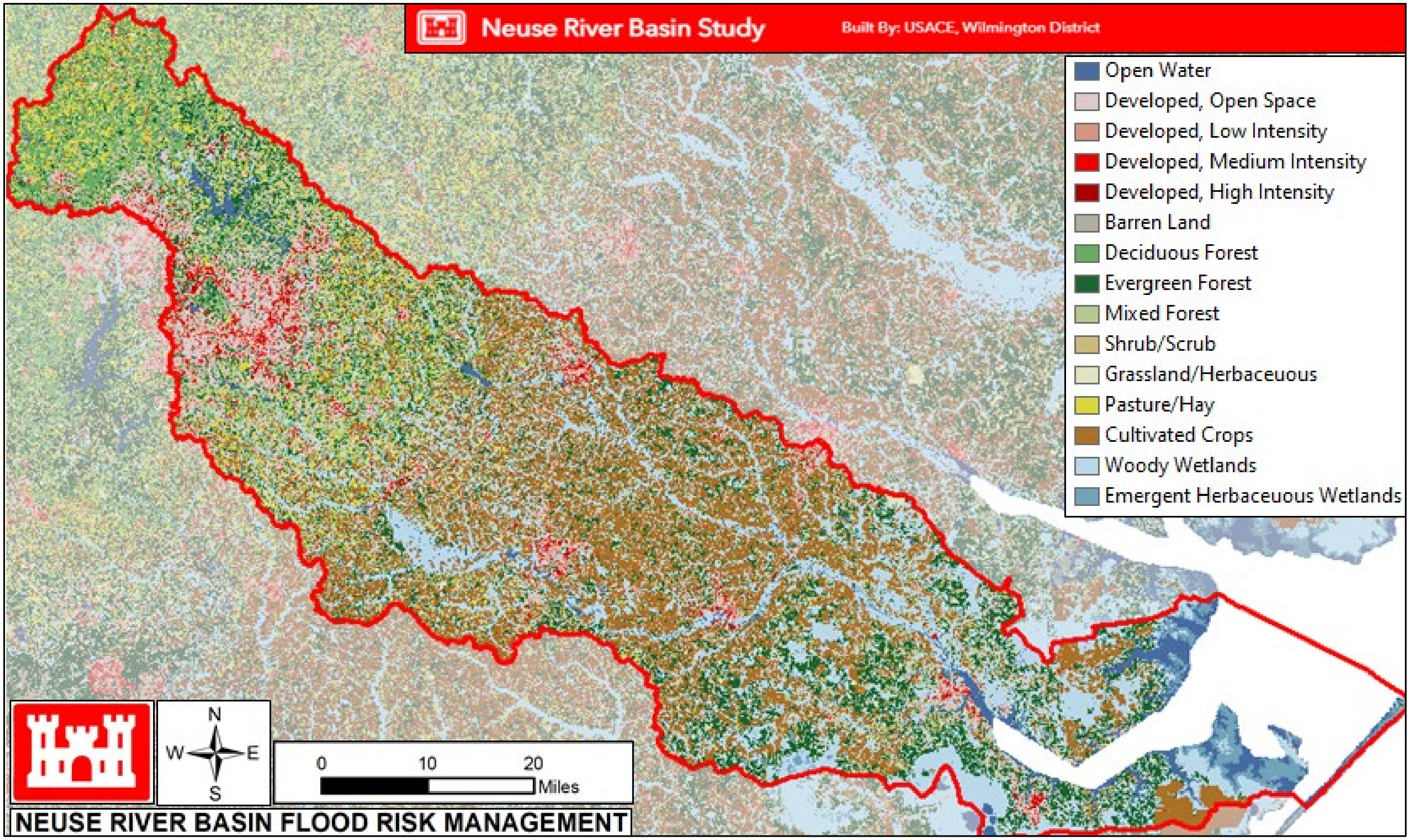

Figure 5. NLCD (2019) for Neuse River Basin

2.5 Climate

The Neuse 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 Neuse River basin ranges from about 46 inches near Raleigh, NC up to 54 inches near New Bern, NC. 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 2.9 inches (NACSE, 2021). A study of the rainfall records shows the wettest year of record to be 2018 when the rainfall near New Bern was approximately 76 inches. The driest year of record was 1941 when the rainfall above Falls Lake Dam was 27.6 inches (USACE, 1984). Droughts occasionally damage crops throughout the basin and cause water storages. Snow constitutes only a small portion of the precipitation and does not affect runoff appreciably.

Storm occurrences in the Neuse 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. North 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 prolong precipitation.

2.6 Topography

The Neuse River basin lies within the Piedmont Plateau and Coastal plain physiographic provinces. These regions run southwest to northeast, in contrast to the northwest to southeast orientation of the study area. The boundary between these two regions is a belt, or zone, about 40 miles in width, known as the “Fall Line”. The northwestern boundary of this zone crosses the basin near Raleigh, NC and the southeastern edge passes near Wilson, NC. The Piedmont Plateau consists largely of rolling hills and deeply eroded valleys. The top of the hills are remnants of former peneplain which has greatly weathered. The elevation of the Piedmont Plateau varies in the Neuse River basin from 800 feet at the headwaters of the Eno and Flat Rivers to about 200 feet where it merges into the Coastal Plain. The remainder of the drainage area of the Neuse River is in the Coastal Plain. The topography in this region varies from rolling sandhills at its western boundary to almost level land as it approaches the Atlantic Ocean, its larger portion being gently rolling in character. The stream valleys are relatively wide, with large areas subject to overflow.

2.7 Geology

The surface mantle of the Piedmont Plateau consists largely of soils of slate or granite origin, the principal types being composed of sand and clay in varying mixtures. The topsoils are usually shallow and are underlain by slate, sandstone, quartz, and granite, or other igneous material. The large streams have, in general, cut their beds down to basement rocks which are igneous in origin. Faults and fractures are unusual in this region, and there are generally good foundations from dams. It is in this region that the Falls Lake Dam federal project is located. The Coastal Plain is composed largely of sand, gravel, and marine deposits of comparatively recent origin. The whole is underlain by the basement rocks (USACE, 1984).

2.8 Previous Studies

2.8.1 FEMA Flood Insurance Studies

Original Federal Emergency Management Agency (FEMA) Flood Insurance Studies (FIS) for counties within the Neuse River basin study area date back to the early 1990s. These studies included hydrologic and hydraulic analyses for the majority of watercourses in the basin. Many of the initial FIS for these counties were prepared by USACE for FEMA under an inter-agency agreement. Streams were studied in varying degrees of detail due to the study’s mixed rural and urban footprint and availability of engineering data.

2.8.2 USACE Studies

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

Neuse River Basin, N.C., 1963. This report investigated the need for flood protection (flood risk management), water supply, water-quality control, and reaction in the Neuse River basin. Prior study efforts related to this report dated back to the early 1930s. This report investigated multiple large-scale reservoirs throughout the basin. Outcome of this report was the confirmation for federal interest in the construction of Falls Lake Dam in Raleigh, NC.

Neuse River, North Carolina Reconnaissance Report, 1984. This report was requested by the State of North Carolina after a period of study inactivity, dating back to the late 1970s. Specific emphasis was placed on municipal and industrial water supply, water quality, and flood control (flood risk management).

Neuse River, NC Final Survey Report, 1991. This report was authorized to review water resources need of the Neuse River basin, with particular reference to the feasibility of constructing the Wilson Mills, Buckhorn, and Beulahtown Dams and Reservoirs. The report outcome was no federal interest in reservoir development in the basin at that time.

Neuse River Basin Integrated Feasibility Report and Environmental Assessment, 2013. This report was originally scope for interests of flood risk management, environmental protection and restoration, and related purposes. Outcomes of the study were multiple areas of environmental restoration; however, no federal interest was identified for flood risk management improvements in the basin.

2.8.3 State Studies

Neuse River Basin Flood Analysis and Mitigation Strategies Study, 2018. This report was conducted by North Carolina Emergency Management and North Carolina Department of Transportation following the Hurricane Matthew event in 2016. The report investigated primary sources of flooding within the Neuse River basin and identified and assessed possible mitigation strategies to prevent future flood damage. A quantitative hydrologic engineering model of the Neuse River basin was created for this effort by contractors of the State of North Carolina (AECOM, 2018). Outcomes of this report were assessments of flooding sources, structural flood impact, and planning-level mitigation strategies for the Neuse River basin.

Flood Abatement Assessment for Neuse River basin, 2020. This report was conducted by North Carolina Department of Transportation with partnership with NC Sea Grant and North Carolina State University. It documented hydrologic and hydraulic modeling, engineering analyses, coordinated technical meetings, and organized community outreach efforts that focused on flood mitigation for the Neuse River basin. Outcome of this report was a better understanding of riverine flooding in the basin, development of potential mitigation measures, improvements to early warning systems for transportation-related infrastructure, assessment of future flooding, and improvements to local floodplain ordinances.

Identification and Prioritization of Tributary Crossing Improvements, 2020. This report was conducted by North Carolina Department of Transportation with partnership with North Carolina State University, NC Cooperative Extension, and NC Sea Grant. The report investigated flash flooding along tributary streams to the Neuse River to identify key crossings and develop a prioritization process for upgrading the crossings to improve municipalities' resilience to flooding. Outcome of this effort was a prioritization of key crossings for improvement for tributaries in the Smithfield, Goldsboro, and Kinston areas.

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

2.9 Existing Flood Risk

2.9.1 Raleigh, NC

There has historically been concern about the flooding on Crabtree Creek and Walnut Creek, both tributaries of the Neuse River. Crabtree Creek has a history of recurring flood damages to floodplain development. In response to these concerns, the Natural Resources Conversation Service (NRCS) constructed multiple flood retarding structures within the Crabtree Creek watershed to reduce the magnitude and frequency of the flood problem. While some segments of Crabtree Creek have undergone extensive retrofitting with flood-proofing measure (Crabtree Valley Mall), significant overbank flooding still exists. Wide floodplains near the Wake Forest Rd crossing are exacerbated by the Big Branch and Pigeon House Branch tributaries that drain into Crabtree Creek over a relatively short distance. There is very little natural floodplain left that has not been influenced in some way by the intense urbanization that has occurred in the Crabtree Creek watershed.

While this upper portion of the Neuse River basin has fared well in response to the recent significant tropical events (Hurricane Matthew, 2016, and Hurricane Florence, 2018), Tropical Storm Alberto, in 2006, significantly impacted this region. Furthermore, the hilly terrain and steep stream gradients expose this area to the risk of flash flooding following short duration but intense local rainfall. The necessity for heavily used transportation routes that cross these creeks also create flooding risks due to debris blockages at bridge/culvert structures, especially in areas downstream of Umstead State Park, a heavily forested, undeveloped subbasin. The FEMA effective flood zones along Crabtree Creek in Raleigh provided by North Carolina Flood Risk Information System (NCFRIS) are shown in Figure 6. Floods of record of the Crabtree Creek near US-1 in Raleigh are shown in Figure 8 and listed in Table 4.

The Walnut Creek basin, with a drainage area of about 50 square miles, is located south of downtown Raleigh and flows roughly parallel to Crabtree Creek. Walnut Creek possesses similar flood risk to Crabtree Creek as they're both typically within the same precipitation footprint for significant storm events. There are two moderately sized reservoirs and dams along the Walnut Creek watercourse, Lake Johnson and Lake Raleigh. Following Hurricane Matthew in 2016, the City of Raleigh implemented structural modifications to the Lake Johnson Dam to allow for additional control over when flow is release from its reservoir. The city now has the capability to preemptively lower water levels within Lake Johnson ahead of a forecasted storm event. Downstream conditions and the ability to handle additional flow from dam releases are heavily weighted in their decision-making process. FEMA effective flood zones along Walnut Creek are shown in Figure 7.

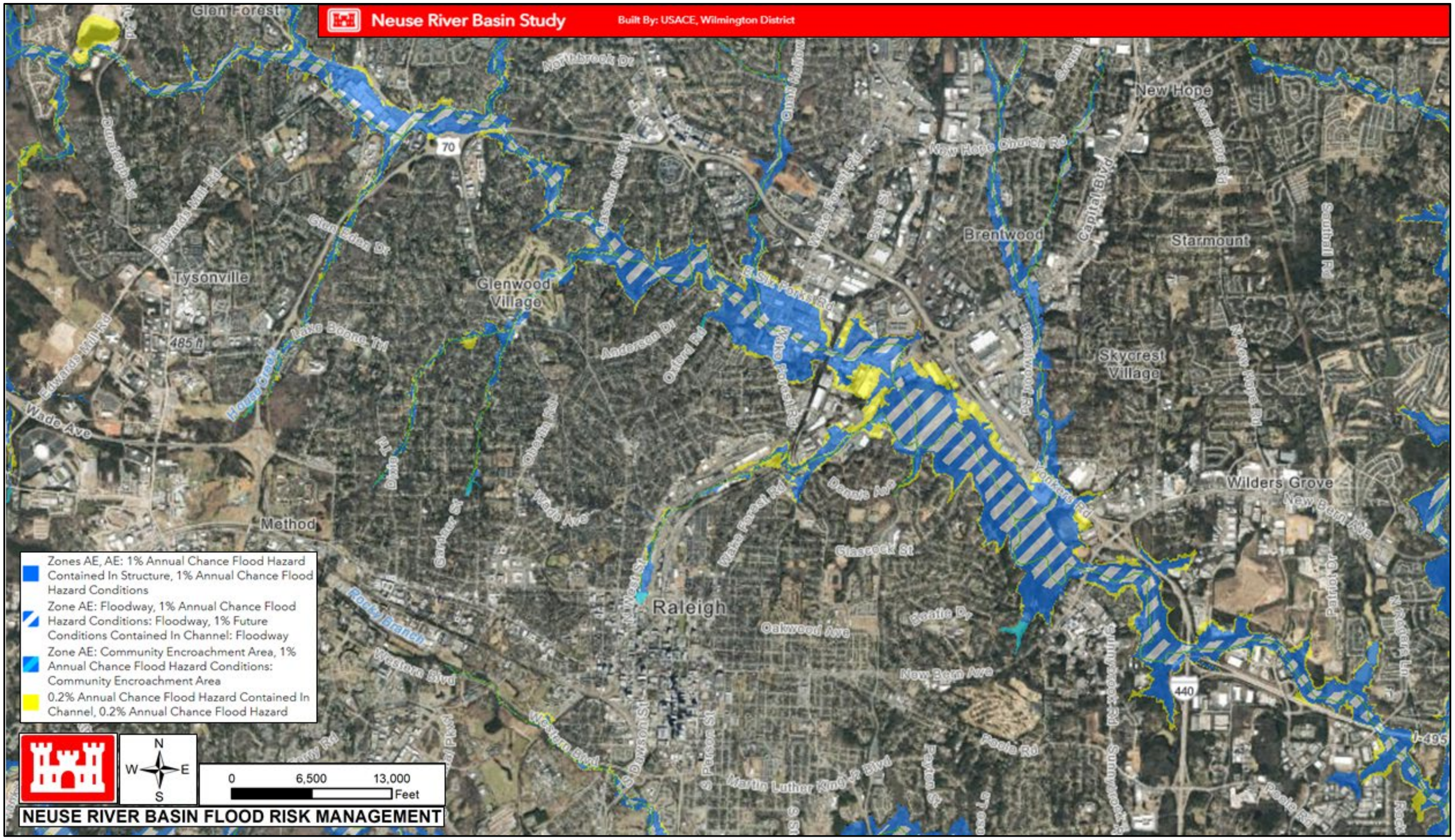

Figure 6. FEMA Effective Flood Zone – Crabtree Creek, Raleigh, NC

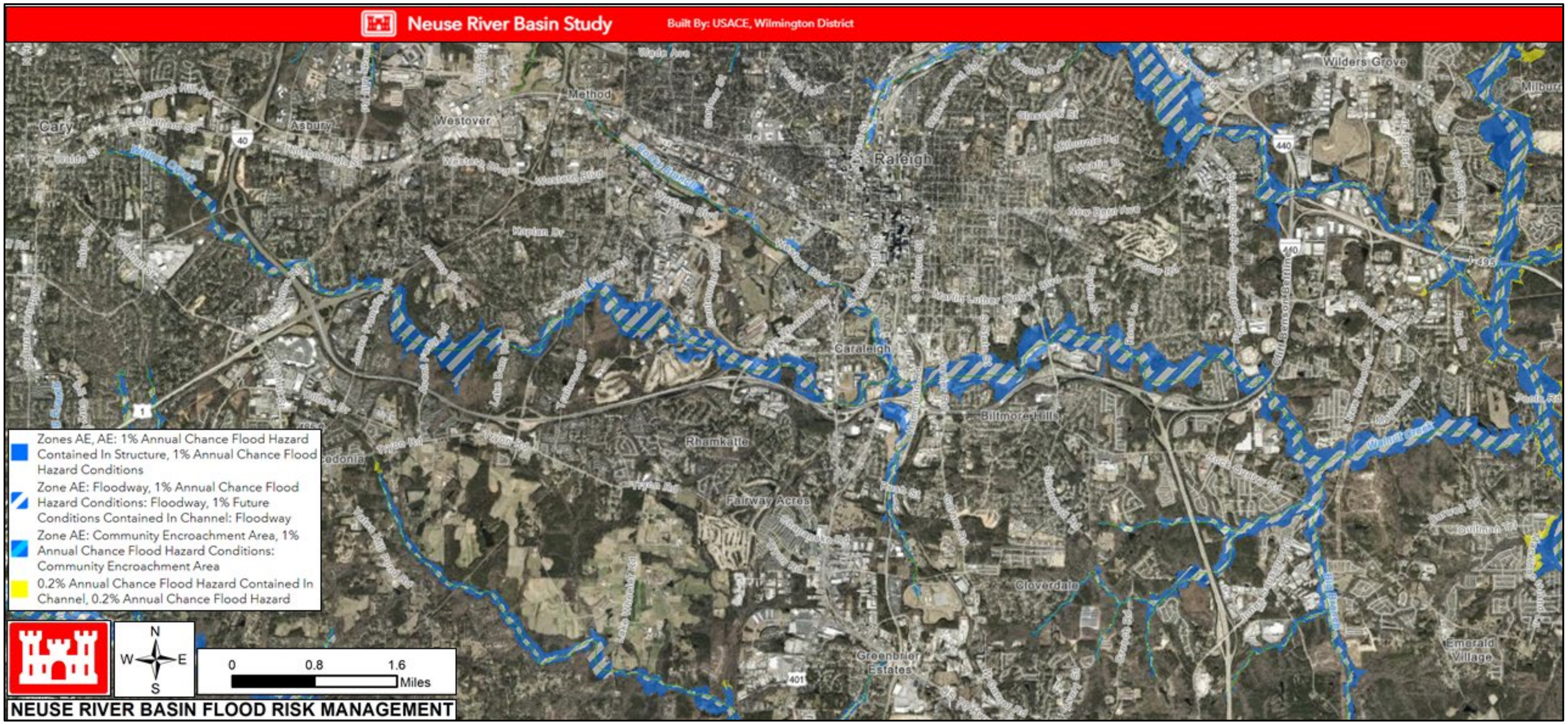

Figure 7. FEMA Effective Flood Zone – Walnut Creek, Raleigh, NC

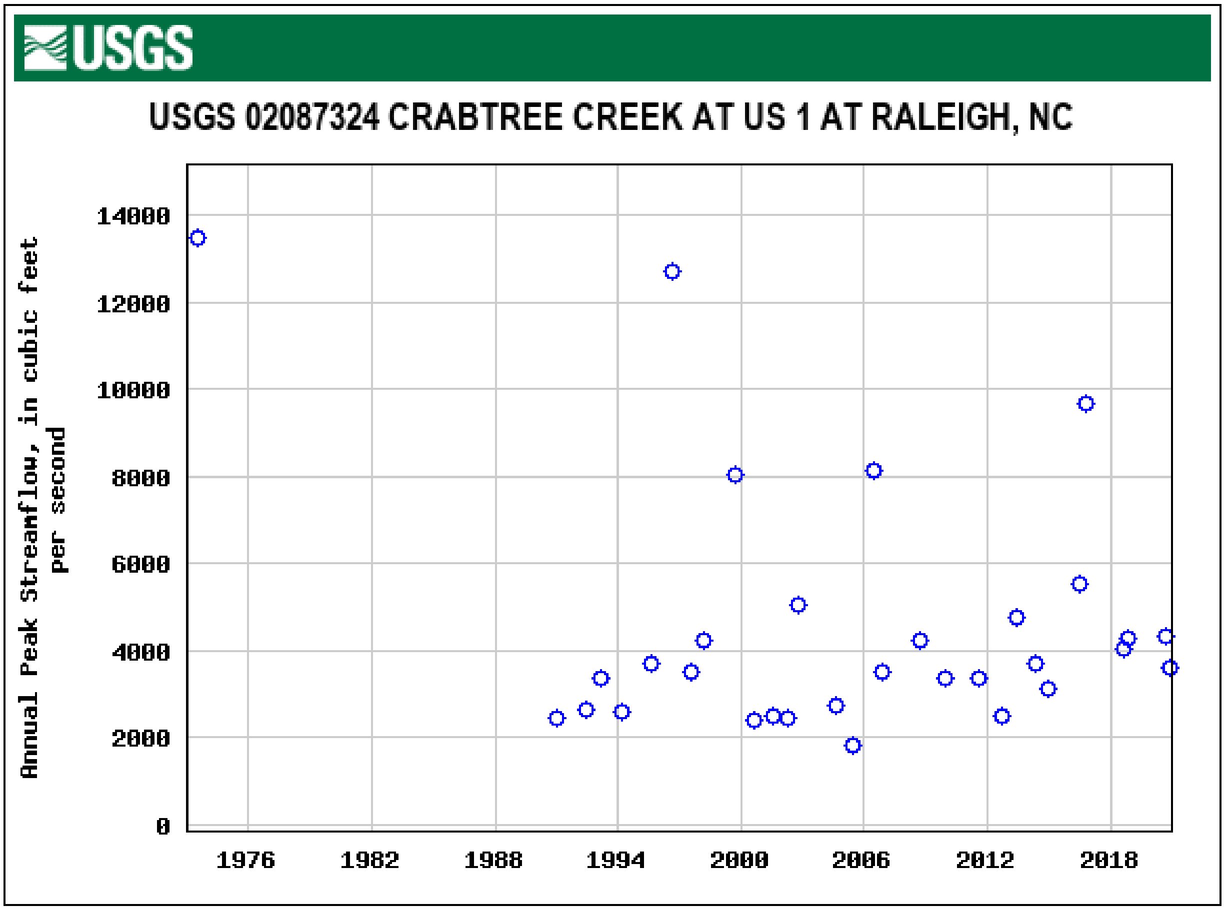

Figure 8. Floods of record of the Crabtree Creek near US-1 in Raleigh, NC

Table 4. Select Floods of Record of the Crabtree Creek near US-1 in Raleigh, NC

Date	Streamflow (cfs)	Gage Height (ft)
6/29/1973	13,500	17.98
1/12/1991	2,450	10.66
6/26/1992	2,610	11.02
3/4/1993	3,330	12.4
3/2/1994	2,600	11.01
8/28/1995	3,670	12.99
9/6/1996	12,700	18.23
7/24/1997	3,500	13.14
3/19/1998	4,230	14.08
9/16/1999	8,050	16.88
9/4/2000	2,390	11.15
7/27/2001	2,480	10.37
4/1/2002	2,460	10.47
10/11/2002	5,040	14.59
8/13/2004	2,730	10.9
6/7/2005	1,830	9.18
6/14/2006	8,150	16.93
11/22/2006	3,490	12.18
9/6/2008	4,240	13.54
12/3/2009	3,370	12.23
8/6/2011	3,350	12.17
9/6/2012	2,470	10.09
6/8/2013	4,770	14.4
5/16/2014	3,710	12.89
12/24/2014	3,130	11.7
7/17/2016	5,510	15.24
10/8/2016	9,650	17.49
8/20/2018	4,030	13.42
11/13/2018	4,280	13.77
9/1/2020	4,340	13.85
11/12/2020	3,590	12.67

2.9.2 Smithfield, NC

Flooding along the Neuse River inundate portions of Smithfield over a short distance from the river's left bank. Areas west of the river also experience flooding from Swift Creek and Middle Creek, major tributaries which drain into the Neuse River near the city. Meanders in the Neuse River's flow path cause it to overflow its banks near the junction point with the tributaries. The Johnston County Public Utilities Wastewater Treatment Plant is located within the FEMA 0.01-AEP flood zone and is partially within the regulatory floodway. The plant has historically been impacted by major tropical storm events (Hurricane Floyd, 1999, and Hurricane Matthew, 2016). Several miles downstream of the city the natural floodplain narrows to about 1,400 feet wide, and this physical constriction can influence flooding upstream within the city limits.

Small tributaries that flow from east to west, Spring Branch and Buffalo Creek, respond quickly to local rainfall caused by events such as summer thunderstorm, which creates a concern for flash flooding. These two creeks flow through much of the city and drain directly into the Neuse River. They are characterized by multiple stream crossings within in a short distance. Routine flood risk to structures adjacent to Spring Branch has resulted in a potential comprehensive nonstructural solution that is currently being pursued by the State of North Carolina. The FEMA effective flood zones near Smithfield provided by NCFRIS are shown in Figure 9. FEMA Effective Flood Zone – Smithfield, NC. Floods of record of the Neuse River near Smithfield are shown in Figure 10 and select events are listed in Table 5.

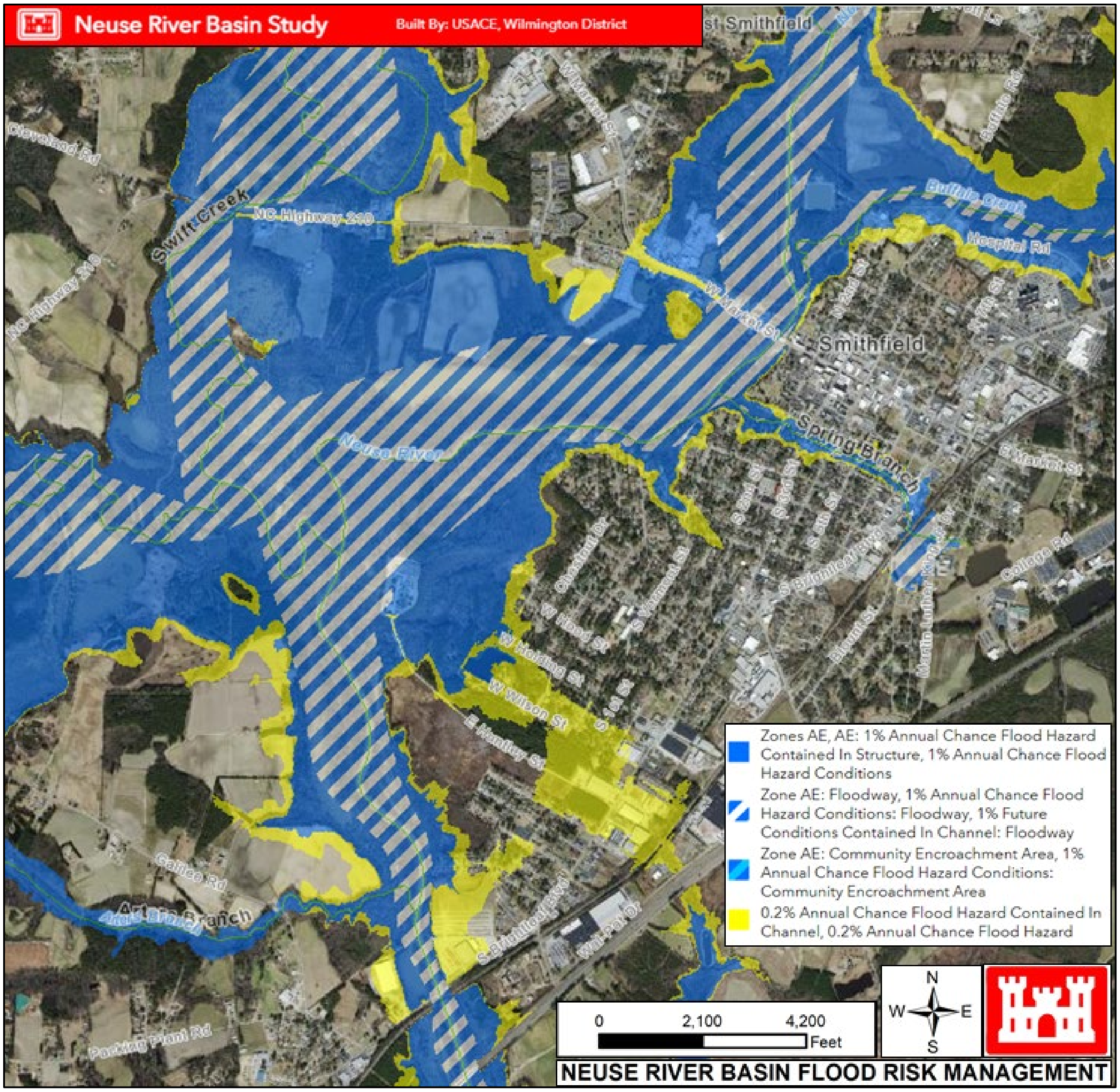

Figure 9. FEMA Effective Flood Zone – Smithfield, NC

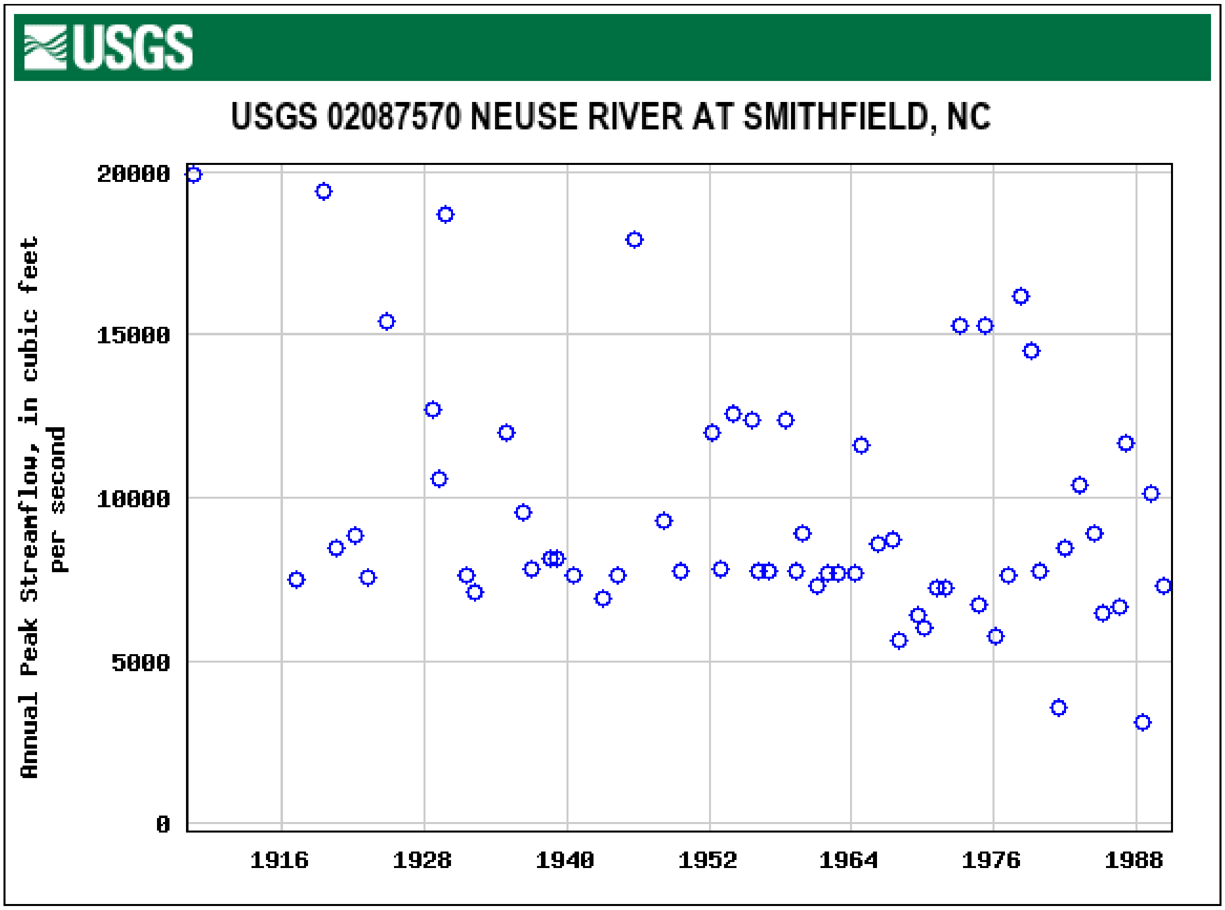

Figure 10. Floods of Record of the Neuse River near Smithfield, NC

Table 5. Select Floods of Record of the Neuse River near Smithfield, NC

Date	Streamflow (cfs)	Gage Height (ft)
3/20/1912	--	18.60
9/5/1913	--	19.40
2/22/1914	--	17.50
12/28/1914	--	18.20
2/8/1916	--	17.50
4/23/1918	--	17.80
7/24/1919	19,400	26.80
7/22/1920	8,480	19.70
2/13/1921	--	18.20
3/6/1922	8,810	20.00
9/30/1924	--	22.40
10/1/1924	15,400	24.40
2/5/1926	--	16.80
3/9/1927	--	15.80
9/21/1928	12,700	22.90
3/7/1929	10,600	21.40
10/3/1929	18,700	26.40
4/19/1933	--	16.40
4/15/1934	--	17.90
12/2/1934	12,000	22.40
4/9/1936	9,520	20.60
7/31/1938	8,160	19.40
2/13/1939	8,160	19.40
7/16/1941	--	16.40
9/9/1942	--	14.70
9/20/1945	17,900	25.90
12/31/1945	--	18.40
1/22/1947	--	16.60
2/16/1948	9,280	20.40
11/5/1949	--	15.70
4/12/1951	--	14.30
3/7/1952	12,000	22.40
1/24/1954	12,600	22.80
9/5/1955	12,400	22.70
5/9/1958	12,400	22.70

Date	Streamflow (cfs)	Gage Height (ft)
4/22/1959	7,740	19.00
10/25/1959	8,920	20.12
10/7/1964	11,600	22.10
3/7/1966	8,580	19.75
6/20/1967	8,690	19.90
2/5/1973	15,300	23.65
3/21/1975	15,300	23.65
4/29/1978	16,200	23.11
3/1/1979	14,500	22.16
1/5/1982	8,460	18.39
3/20/1983	10,400	19.77
5/31/1984	8,900	18.72
3/2/1987	11,700	20.58
3/25/1989	10,100	19.54
9/18/1999	--	26.72
10/1/1999	--	20.17
4/2/2001	--	16.37
4/2/2002	--	15.41
3/21/2003	--	19.10
8/17/2004	--	17.38
1/15/2005	--	14.07
6/16/2006	--	22.95
11/23/2006	--	18.80
9/7/2008	--	16.56
6/17/2009	--	16.78
2/7/2010	--	18.55
10/1/2010	--	17.91
9/4/2012	--	10.77
6/9/2013	--	19.56
5/17/2014	--	19.28
12/25/2014	--	18.98
12/24/2015	--	18.33
10/10/2016	--	29.09
9/17/2018	--	18.90
11/14/2018	--	19.91

2.9.3 Goldsboro, NC

The flood problem at Goldsboro, NC, is extensive with the 0.002-AEP event floodplain extending over a large portion of the city and surrounding development. In addition to the main stem of the Neuse River, significant flooding occurs from the Little River on the west side of town, Big Ditch through the city center, and Stoney Creek on the east side of town. The FEMA effective flood zones near Goldsboro provided by NCFRIS are shown in Figure 11. Flooding for the 0.002-AEP event along the US-117 corridor reach a depth of 6-7 feet. Floods of record of the Neuse River near Goldsboro are shown Figure 12 and listed in Table 6.

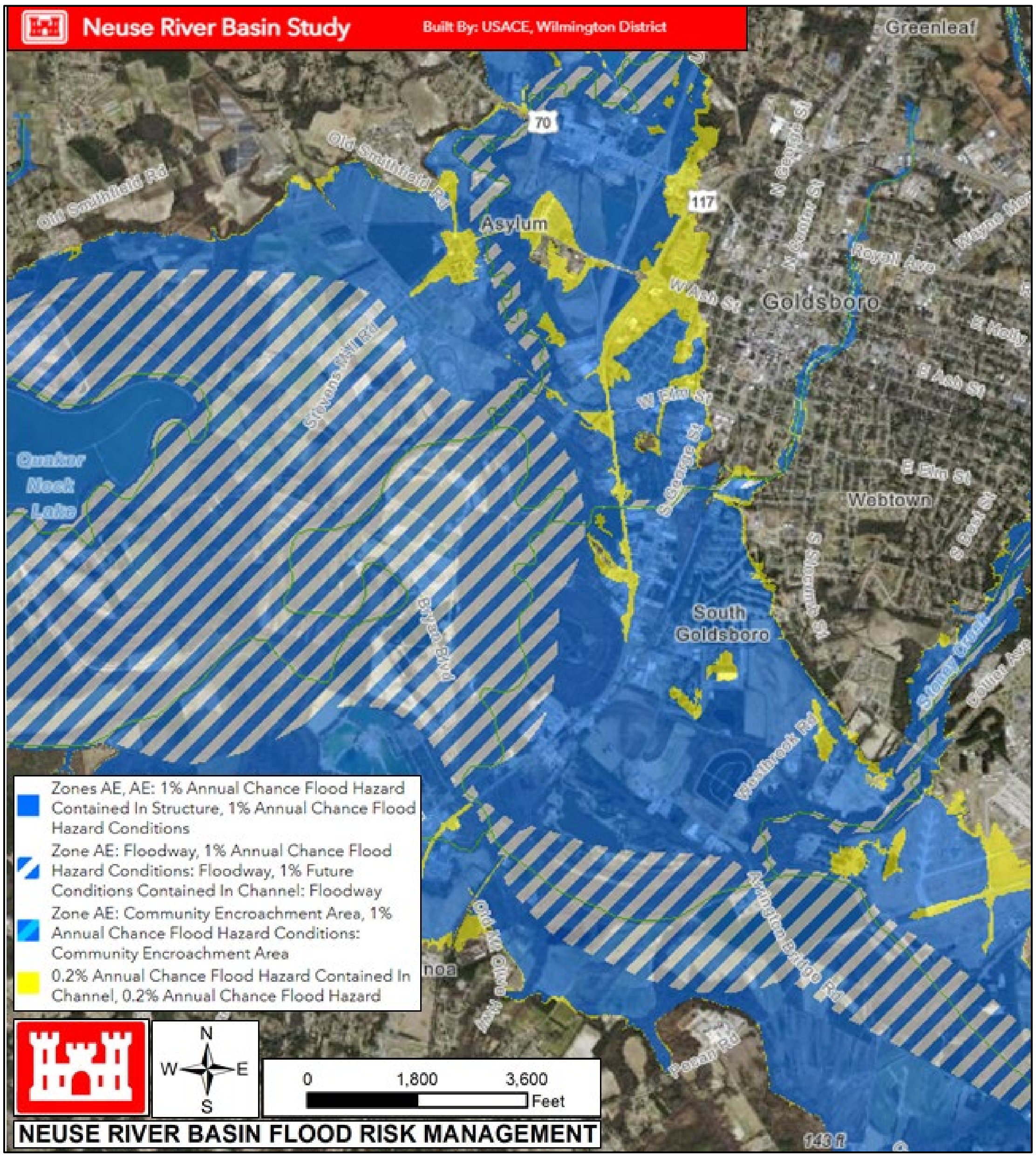

Figure 11. FEMA Effective Flood Zones – Goldsboro, NC

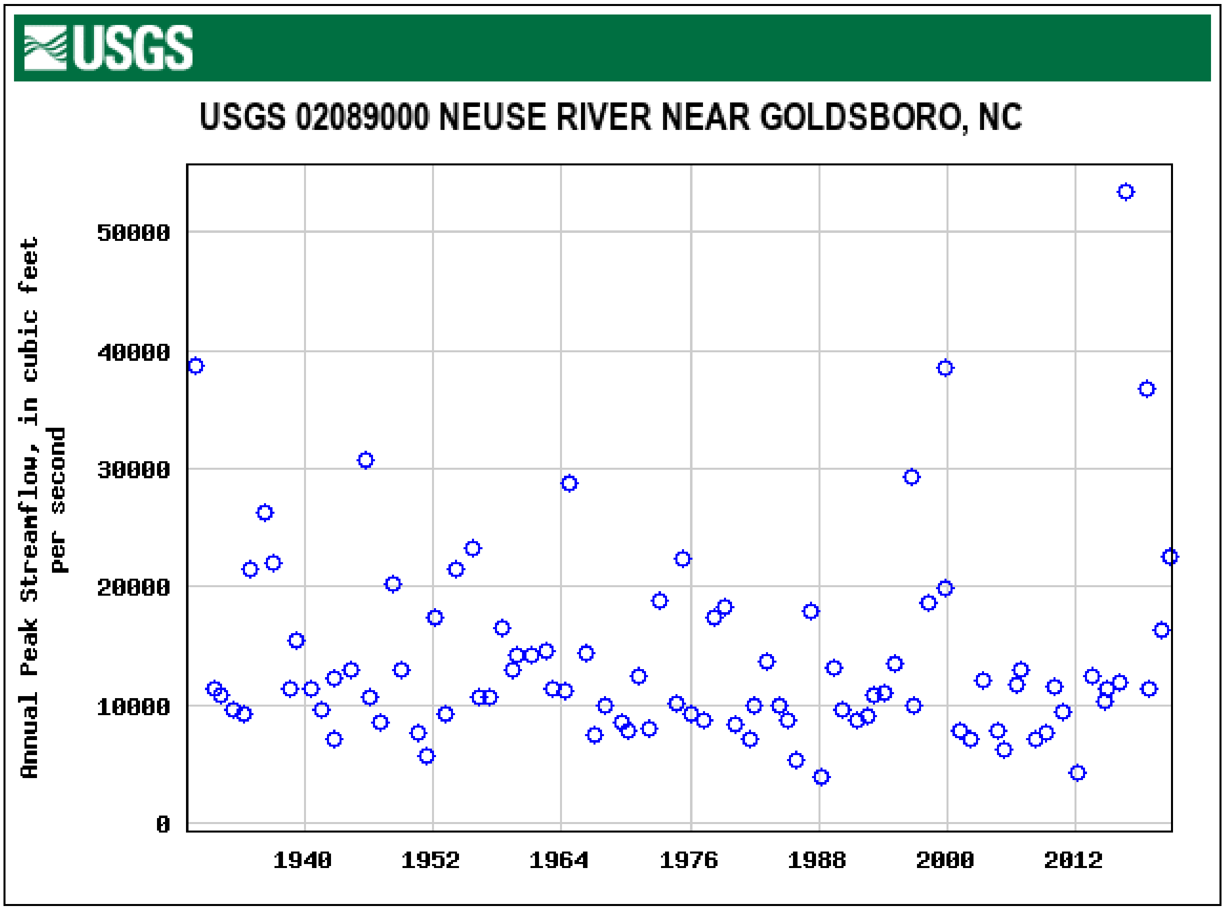

Figure 12. Floods of Record of the Neuse River near Goldsboro, NC

Table 6. Select Floods of Record of the Neuse River near Goldsboro, NC

Date	Streamflow (cfs)	Gage Height (ft)
10/5/1929	38,600	27.3
12/6/1934	21,400	--
4/11/1936	26,300	--
2/3/1937	22,000	--
3/7/1939	15,500	--
9/23/1945	30,700	--
2/19/1948	20,300	--
3/11/1952	17,300	22.29
1/28/1954	21,400	23.77
9/8/1955	23,200	24.36
5/13/1958	16,500	22.35
10/9/1964	28,800	26.07
2/9/1973	18,800	23.2
3/24/1975	22,300	24.39
5/3/1978	17,400	22.74
3/6/1979	18,200	23.02
3/6/1987	18,000	22.93
9/12/1996	29,300	26.21
3/14/1998	18,700	23.02
9/20/1999	38,500	28.85
10/4/1999	19,900	23.47
10/12/2016	53,400	29.74
9/18/2018	36,700	27.6
2/11/2020	16,400	22.31
11/16/2020	22,500	--

The October 2016 flood event (Hurricane Matthew) caused at least 1 life to be lost and extensive economic damages including the inundation of hundreds of structures in Goldsboro (Overton, 2016). Residential subdivisions south of the Neuse River cutoff channel and clusters of residential homes east of US-117 experienced inundation of several feet above first floor elevations. Parcels surrounding Cherry Research Farm and Neuse Correctional Institution were similarly left inundated for a prolonged period of time. The Seymour Johnson Air Force Base east of Goldsboro was also impacted by flooding.

2.9.4 Kinston, NC

Details of flooding documented near Kinston, NC date back to the 1940s. Consistent flood risks are associated with difficulties of citizens evacuating the floodplain, becoming stranded, and/or requiring rescue. As recent as 2016, during and following Hurricane Matthew, major transportation routes were significantly impacted. Routes HWY-258, Queens St, and NC-11, all major thoroughfares that connect the north and south sides of the floodplain were impassable for several days. The HWY-258 and Queens St intersection was underwater by several feet and south of the HWY-258 and NC-11 intersection, the road was flooded to a depth of 5-7 feet. Approximately 40-percent of the total land area of the city of Kinston lies in the northern floodplain of the Neuse, including most of the downtown district. The historical Lincoln City area, south of the downtown district, has remained exposed to historic flooding. The city has undergone partnership with Federal and State agencies to implement nonstructural programs in response to being repeatedly flooded. As a result, the majority of structures in this area have been removed from the floodplain.

Floods on Adkins Branch, a small tributary that traverses through the City of Kinston and drains directly into the Neuse River floodplain, have been characterized by flash flooding. Due to the stream’s relatively small size and high degree of development, flood stages along most of Adkins Branch are reached only a few hours after intense rainfall begins; and the stream remains out of its bank generally for less than 18 hours.

The FEMA effective flood zones near Kinston provided by NCFRIS are shown in Figure 13. Floods of record of the Neuse River near Kinston are shown in Figure 14 and select events are listed in Table 7.

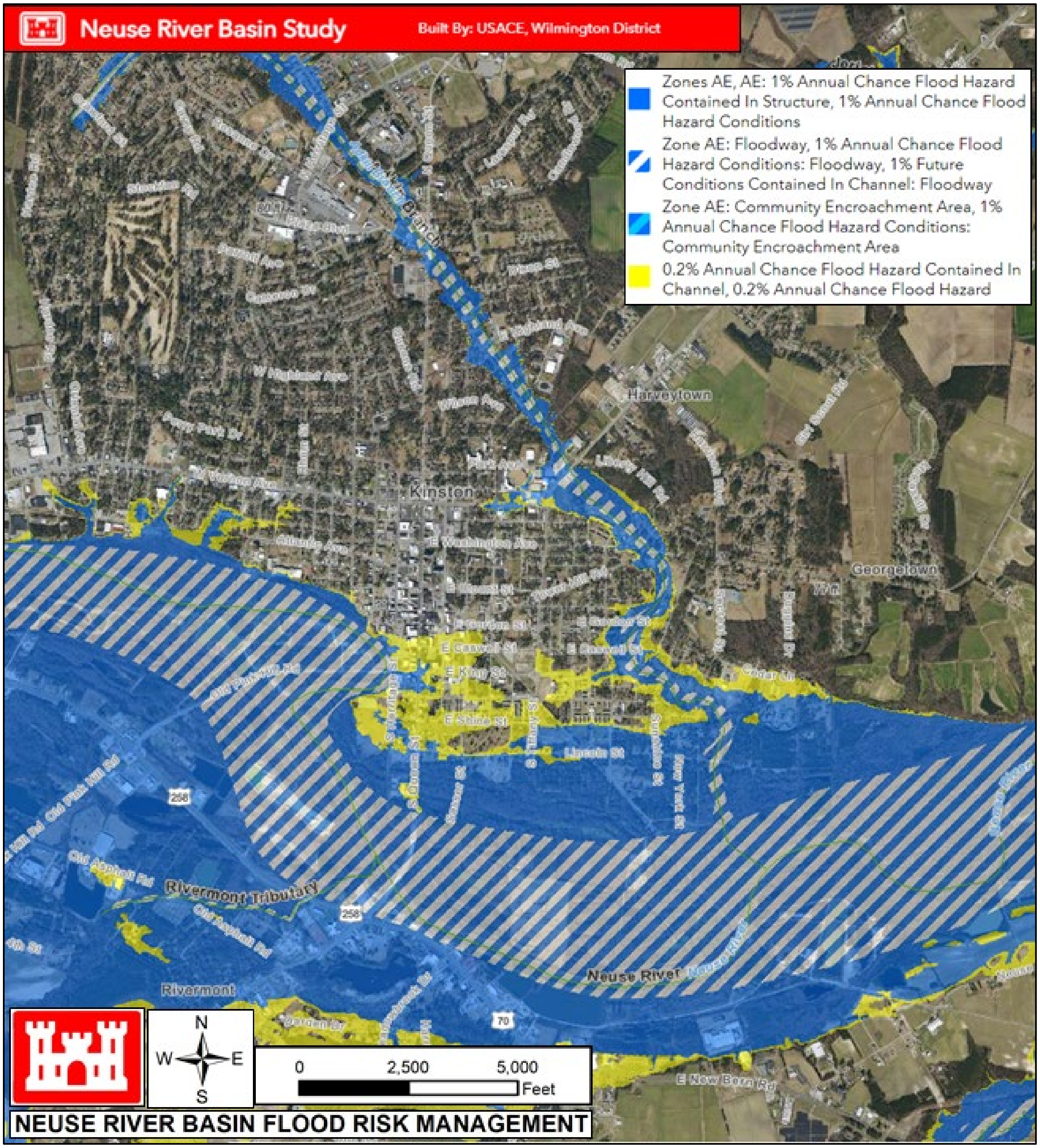

Figure 13. FEMA Effective Flood Zones – Kinston, NC

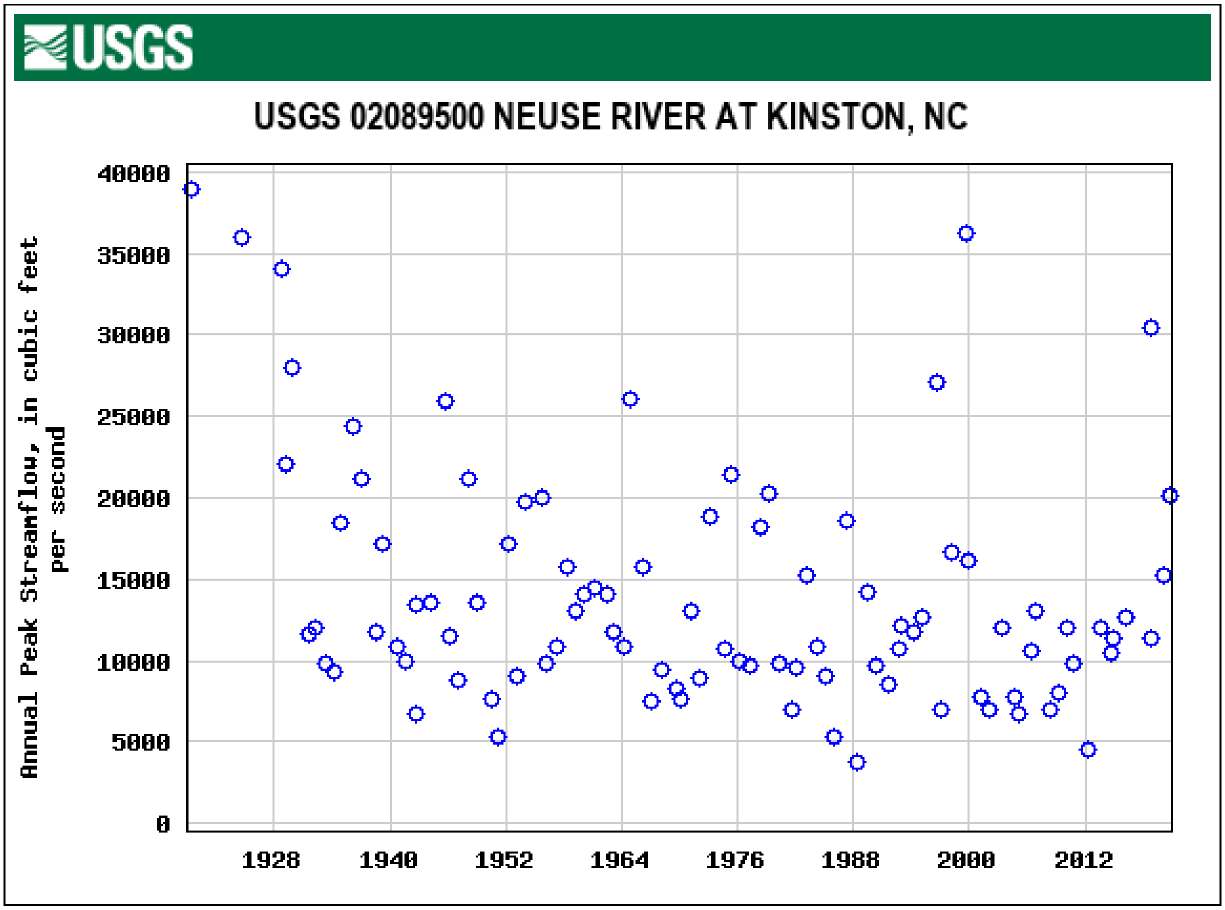

Figure 14. Floods of Record of the Neuse River near Kinston, NC

Table 7. Select Floods of Record of the Neuse River near Kinston, NC

Date	Streamflow (cfs)	Gage Height (ft)
7/1919	39,000	25
10/1924	36,000	24.7
9/25/1928	34,000	24.2
3/12/1929	22,000	20.8
10/9/1929	28,000	22.48
8/23/1931	11,600	16
3/16/1932	12,000	16.24
12/9/1934	18,500	19.16
4/14/1936	24,400	20.9
2/6/1937	21,200	20.04
8/7/1938	11,800	16.65
3/9/1939	17,200	18.88
8/25/1940	10,900	16.14
10/23/1942	13,400	17.7
3/30/1944	13,600	17.82
9/27/1945	25,900	22.41
2/21/1946	11,500	16.8
2/22/1948	21,100	20.75
12/11/1948	13,600	17.83
3/14/1952	17,100	19.18
2/1/1954	19,800	20.28
9/12/1955	20,000	20.81
3/27/1956	9,820	16.26
3/11/1957	10,800	16.4
5/17/1958	15,800	18.7
4/29/1959	13,100	17.64
2/15/1960	14,100	18
3/5/1961	14,400	18.08
7/11/1962	14,100	18.02
1/29/1963	11,700	16.92
3/23/1964	10,900	16.63
10/13/1964	26,000	22.86
3/13/1966	15,800	18.69

Date	Streamflow (cfs)	Gage Height (ft)
3/13/1971	13,000	17.63
2/13/1973	18,900	20.16
8/13/1974	10,700	16.47
3/27/1975	21,400	21.18
2/7/1976	10,000	16.06
5/7/1978	18,200	20.15
3/9/1979	20,200	20.72
3/28/1983	15,200	18.67
3/31/1984	10,900	16.58
3/9/1987	18,600	20.03
5/11/1989	14,200	18.22
8/20/1992	10,700	16.46
1/16/1993	12,100	17.19
3/11/1994	11,800	17.04
2/26/1995	12,600	18.04
9/17/1996	27,100	23.26
3/17/1998	16,700	20.08
9/22/1999	36,300	27.71
10/25/1999	16,100	19.83
4/17/2003	12,000	--
6/24/2006	10,600	16.7
11/30/2006	13,100	18.18
2/14/2010	12,000	17.52
10/7/2010	9,780	16.14
7/20/2013	12,000	17.54
9/17/2014	10,400	16.53
1/1/2015	11,300	17.12
2/12/2016	12,600	17.88
9/21/2018	30,500	25.78
11/22/2018	11,300	17.12
2/15/2020	15,200	19.02
11/19/2020	20,100	21.51

2.9.5 Rural Areas

Throughout much of the 19th century, flooding to rural floodplains consisting of woodlands and cultivated crops land cover has resulted in significant agricultural and economic losses. These floodplains included Johnston and Wayne Counties, between the Cities of Smithfield and Goldsboro, Lenoir County near Kinston, and Wilson and Greene Counties adjacent to Contentnea Creek. The floodplains in these areas have a large footprint at 1.5 to 3 miles in width.

Lands extensively used for agricultural purposes have had natural drainage paths altered to drain more efficiently following localized, high flow conditions. Auxiliary culverts and elevated roadway berms are commonly utilized; however, during significant flood events, these modifications can cause adverse impacts. When drainage outlets lack capacity due to backwaters from river mainstems, they cause prolonged stagnated floodwaters.

2.9.6 New Bern, NC

Flood risk to the City of New Bern is predominately caused by tropical storms. Wind-driven tides have historically caused significant storm surge along the lands adjacent to the Pamlico Sound and up the mouths of tributaries. The confluence of the Neuse River and Trent River into the Pamlico Sound exacerbates nearby flooding to the downtown New Bern area. Similar issues are seen as the Neuse River and Swift Creek confluence upstream of the city. Prolonged high water near the confluences can also create drainage issues further upstream. The length of flooding is highly variable due to conditions upstream that may cause secondary, smaller flow peaks to crest after the main event passed. While this second peak may lead to nuisance flooding, it can also expose transportation routes to inundation. The FEMA effective flood zones provided by NCFRIS are shown in Figure 15.

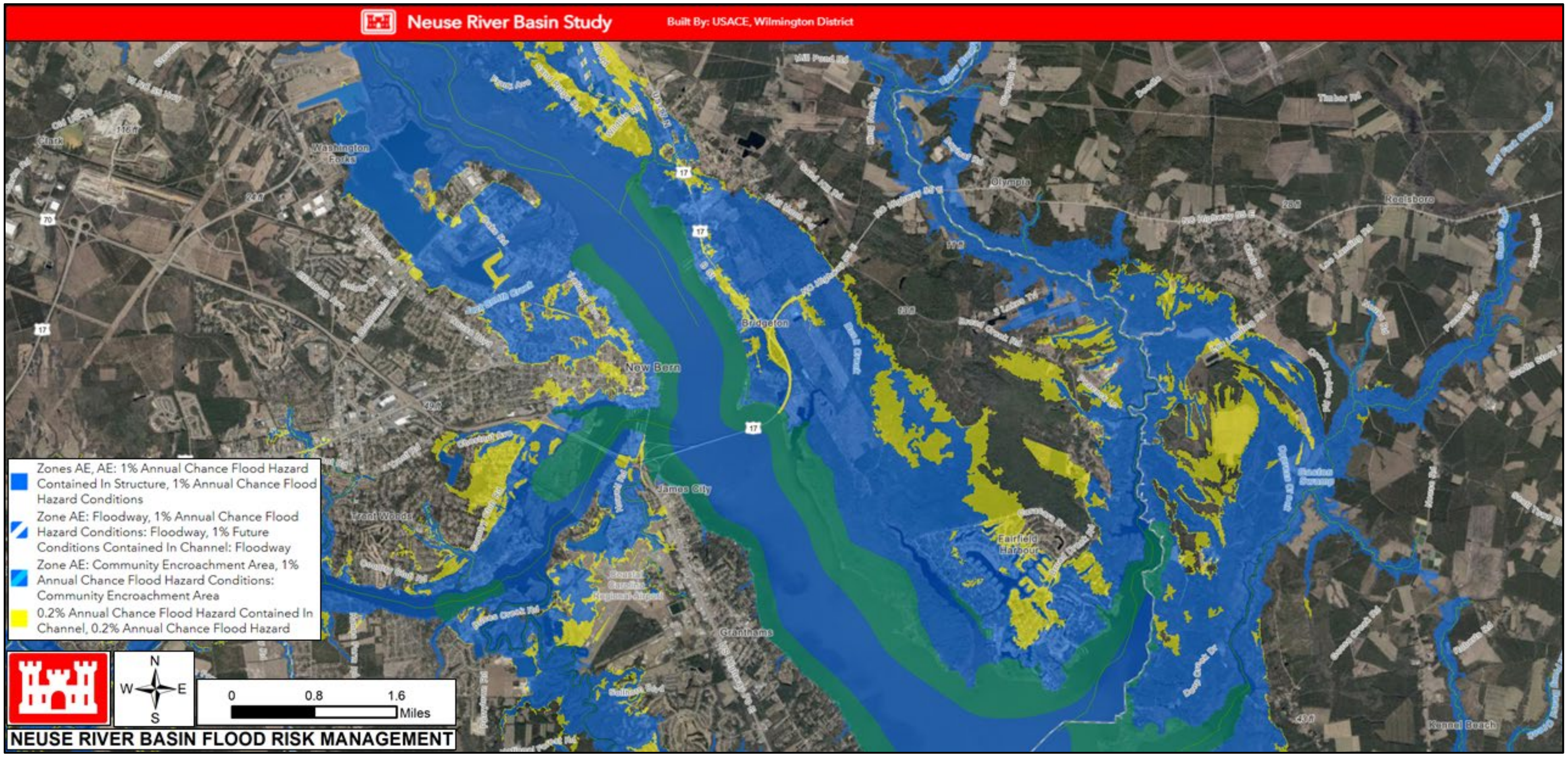

Figure 15. FEMA Effective Flood Zone – New Bern, NC

2.9.7 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. Furthermore, multiple studies have shown that a significant percentage of flood-related fatalities are related to transportation. According to NCDOT, at least 1,700 roads were closed during Hurricane Matthew (2016) and 2,500 roads were closed during Hurricane Florence (2018) (NCDOT, 2020). NCDOT has compiled a summary of major routes, considered strategic transportation corridors, and other primary roads that are historically vulnerable to inundation (NCDOT, 2021). 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. Select route locations throughout the basin and their range of inundation are listed in Table 8.

Table 8. Select Routes in Neuse River Basin Counties Vulnerable to Flood-based Inundation

County	Route	River Crossing/Flood Source	Road Inundation Depth (ft) 0.1-AEP	0.04-AEP	0.02-AEP	0.01-AEP	0.002-AEP
Johnston	US-70	Neuse River	--	--	--	--	2-5
Johnston	US-70 Bus	Neuse River	--	--	--	--	>5*
Johnston	NC-210	Swift Creek	--	--	--	--	2-5
Johnston	NC-210	Middle Creek	--	--	--	--	2-5
Johnston	Brightleaf Blvd	Spring Branch	0.5-2	0.5-2	0.5-2	0.5-2	0.5-2
Johnston	Brightleaf Blvd	Buffalo Creek	--	--	0.1-0.5	0.1-0.5	0.5-2
Wayne	NC-581	Little River/Neuse River	--	--	--	--	0.5-2
Wayne	US-70	Little River	--	--	--	--	0.5-2*
Wayne	US-70 Bus	Big Ditch	0.5-2	0.5-2	2-5	2-5	2-5
Wayne	US-117	Neuse River	--	--	0.5-2*	2-5*	>5*
Wayne	US-117 Bus	Big Ditch	--	0.5-2	0.5-2	0.5-2	2-5
Wayne	Arrington Bridge Rd	Neuse River	2-5*	>5*	>5*	>5*	>5*
Wayne	US-13	Stoney Creek	--	--	--	0.1-0.5	0.5-2
Wayne	NC-581/Bill Lane Blvd	Neuse River	--	0.1-0.5*	2-5*	2-5*	2-5*
Wayne	NC-111	Neuse River	--	0.5-2*	0.5-2*	2-5*	>5*
Lenoir	NC-903	Neuse River	--	--	--	0.5-2	2-5
Lenoir	NC-55/W King St	Neuse River	--	--	--	--	0.5-2
Lenoir	US-258	Neuse River	--	--	0.1-0.5*	0.5-2	2-5
Lenoir	US-70	Neuse River Tributary	--	--	0.1-0.5*	0.5-2	2-5

Lenoir	NC-11	Neuse River	--	--	0.5-2*	2-5*	>5*
Lenoir	US-258/S Queen St	Neuse River	--	--	--	0.5-2*	2-5*
Lenoir	NC-55 E	Neuse River	--	0.5-2*	2-5*	2-5*	>5*
Craven	NC-118	Neuse River	--	--	0.5-2	2-5	>5
Craven	NC-118	Swift Creek	--	--	--	0.5-2	2-5
Craven	NC-43/Main St	Swift Creek/Mauls Swamp	--	--	0.1-0.5*	0.5-2	>5
Craven	NC-42/Weyerhauser Rd	Swift Creek	--	--	--	0.5-2*	2-5*
Craven	NC-42/Weyerhauser Rd	Neuse River	--	--	--	0.5-2*	2-5*
Craven	US-17	Little Swift Creek	--	--	--	--	0.1-0.5*
Craven	NC-43/Washington Post Rd	Bachelor Creek	--	--	--	--	2-5*
Craven	US-17	Mills Branch	--	--	--	0.1-0.5*	0.5-2
Craven	US-17	Neuse River	--	--	0.1-0.5*	2-5*	2-5*
Craven	NC-55	Neuse River	--	--	--	--	0.5-2*
Jones	US-17/Main St	Trent River	--	--	0.1-0.5	0.5-2	2-5
Jones	NC-58	Mill Run	--	--	--	--	2-5
Jones	NC-58	Little Hell Creek	--	--	--	--	0.5-2
Jones	NC-41	Trent River	--	0.1-0.5*	0.5-2*	2-5*	>5*
Jones	NC-58/Market St	Crooked Run	--	--	--	0.5-2	2-5
Jones	NC-41	Musselshell Creek	--	2-5	2-5	2-5	>5
Jones	NC-58	Trent River	--	--	--	0.5-2*	2-5*
Pitt	NC-118/Queen St	Contentnea Creek South Tributary	--	--	--	--	0.5-2

Pitt	NC-118/S Highland Ave	Contentnea Creek	--	0.5-2*	0.5-2*	2-5*	>5*
Pitt	NC-11	Contentnea Creek	--	--	--	0.1-0.5*	0.5-2*
Pitt	NC-121	Little Contentnea Creek	--	0.1-0.5*	0.5-2*	0.5-2*	2-5*
Greene	NC-123	Contentnea Creek	--	--	0.5-2*	2-5*	2-5*
Wilson	NC-58	Contentnea Creek	--	--	0.1-0.5*	0.5-2*	2-5*
Wilson	NC-222	Toisnot Swamp	0.1-0.5*	0.5-2*	2-5*	2-5*	>5*
Wilson	NC-42/Herring Ave E	Toisnot Swamp	--	0.5-2	0.5-2	0.5-2	2-5
Wilson	NC-42	Toisnot Swamp	--	2-5*	2-5*	>5*	>5*
Wilson	US-264	Hominy Swamp Creek	0.5-2	0.5-2	2-5	2-5	2-5
Wilson	US-264	Contentnea Creek	--	--	0.5-2	0.5-2	2-5
Wilson	US-301	Contentnea Creek	--	--	--	--	0.5-2
Wilson	NC-42	Bloomery Swamp	--	--	--	--	2-5
Wilson	I-795	Contentnea Creek	--	0.5-2*	0.5-2*	2-5*	>5*
Wilson	NC-42	Shepard Branch	--	--	--	--	2-5
Wilson	NC-42 W	Contentnea Creek	--	--	--	--	2-5
Wilson	NC-581	Contentnea Creek	--	0.1-0.5*	0.5-2*	2-5*	>5*
Nash	US-264	Turkey Creek	--	0.1-0.5	0.5-2	2-5	2-5
Nash	US-264 Alt	Moccasin Creek	--	--	0.1-0.5*	0.5-2*	2-5*
Nash	US-264	Little Creek	--	--	--	0.1-0.5	0.5-2
Nash	US-264	Moccasin Creek	--	--	--	--	0.5-2
Pamlico	NC-33	Jones Bay	--	0.1-0.5	0.5-2	2-5	2-5
Pamlico	NC-304	Jones Bay	--	--	0.1-0.5	0.5-2	2-5

Pamlico	NC-304	Gale Creek	--	--	0.1-0.5	0.5-2	2-5
Pamlico	NC-304	Bear Creek	--	--	0.1-0.5	0.5-2	2-5
Pamlico	NC-304	Vandemere Creek	--	--	0.1-0.5	0.5-2	2-5
Pamlico	NC-304	Smith Creek	--	--	--	--	0.5-2
Pamlico	NC-304	Chapel Creek	--	0.1-0.5*	0.5-2*	0.5-2	2-5
Pamlico	NC-304	Bay River	--	--	0.1-0.5	0.5-2	2-5
Pamlico	NC-304	North Prong Bay River	--	--	0.5-2	0.5-2	2-5
Pamlico	NC-55	South Prong Bay River	--	--	0.1-0.5*	0.5-2*	2-5*
Pamlico	NC-55	Alligator Creek	--	--	0.1-0.5	0.5-2	2-5
Pamlico	NC-55	Trent Creek	--	--	0.5-2*	2-5*	2-5*
Pamlico	NC-55	Greens Creek	--	--	0.5-2*	2-5*	2-5*
Pamlico	NC-55	Morris Creek	--	--	--	--	2-5*

-- AEP event not assessed

* Inundation depth taken adjacent to flooding source and/or at bridge approaches making river crossing/route impassable

As listed in the preceding table, depths of up to 5 feet of water are to be expected during the significant frequency storm events. These depths along the major streams such as the Neuse River, Little River, Middle Creek, and others, are likely to persist for multiple hours or days. The inundated length of roadway ranged up to several hundred feet beyond the actual stream crossing for these major rivers. Additionally, critical timing of historic flooding resulted in fluctuations of roadways that appear momentarily passable but can swiftly become dangerous. This timing aspect can be contributed to complex interactions of flow barriers within wide floodplains and uncertain flow hydrograph attenuation. These flooding characteristics compound risk when considered along wide simple inundation depths.

3 Data Collection

3.1 Hydrologic Data

3.1.1 Streamflow and Stage Data

The United States Geological Survey (USGS) provides extensive coverage of streamflow and stage records throughout the study area. There are multiple sites that have an established record dating back to the early 20th century. Therefore, a number of sites downstream of Falls Lake have captured both unregulated (pre-1983) and regulated periods (post-1983) of operation. Table 9 provides a summary of available data for select USGS sites that were utilized for the purposes of this study.

Table 9. Select USGS streamflow sites pertinent to the Neuse River basin study

Site ID	Description	Drainage Area (sq mi)	Peak Streamflow Period of Record (CY)	Datum (ft, NAVD88)
02085070	Eno River at Hillsborough, NC	66	1928-2020	486.7
02085070	Eno River near Durham, NC	141	1964-2020	269.92
02085500	Flat River at Bahama, NC	149	1926-2020	346.85
02086500	Flat River at Dam near Bahama, NC	168	1928-2020	255.7
0208521324	Little River at SR1461 near Orange Factory, NC	78.2	1988-2020	382.69
208524975	Little R bl Little R Trib at Fairntosh, NC	98.9	1996-2020	263.6
02087183	Neuse River near Falls, NC	771	1945-2020	198.4
0208726005	Crabtree Cr at Ebenezer Church Rd nr Raleigh, NC	76	1989-2020	223.9
02087275	Crabtree Creek at HWY 70 at Raleigh, NC	97.6	1973-2020	202.9

02087324	Crabtree Creek at US1 at Raleigh, NC	121	1973-2020	182.36
02087359	Walnut Creek at Sunnybrook Drive nr Raleigh, NC	29.8	1996-2020	182.24
02087500	Neuse River near Clayton, NC	1150	1919-2020	127.5
02087570	Neuse River at Smithfield, NC	1206	1908-1990	98.3
0208758850	Swift Creek near McCullars Crossroads, NC	250.4	1989-2020	35.8
02088000	Middle Creek near Clayton, NC	83.5	1940-2020	83.5
02088383	Little River near Zebulon	55	2009-2020	230.7
02089000	Neuse River near Goldsboro, NC	2399	1866-2020	41.9
02089500	Neuse River at Kinston, NC	2692	1919-2020	9.7
02091000	Nahunta Swamp near Shine, NC	80.4	1955-2020	49.7
02091500	Contentnea Creek at Hookerton, NC	773	1928-2020	14.85
02091814	Neuse River near Fort Barnwell, NC	3900	1996-2020	0.0
02092500	Trent River near Trenton, NC	168	1928-2020	18.0

From Table 9 it can be seen that all but one site has a period of peak flow record extending through calendar year 2020. The Neuse River at Smithfield, NC site (02087570) halted streamflow and stage records in 1990. Its calibrated rating curve has been used to approximate recent historic flooding events, though there is a high degree of uncertainty due to the potential change in the Neuse River's cross-sectional area that has occurred since 1990. No streamflow or gage height data from site 02087570 was used in analyses conducted as part of the Neuse River Basin Flood Risk Management (FRM) study. 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.

A spreadsheet-based assessment was carried out to help identify potential issues with gage site stability in regard to stage-discharge uncertainty. Sites that appeared to record large differences in stages for similar discharge scenarios may be inappropriate sources to establish Hydrology and Hydraulics (H&H) uncertainty such as standard deviation or natural uncertainty. The assessment was primarily focused on gage sites along the Neuse River mainstem, specifically, sites 02087500 (Clayton), 02089000 (Goldsboro), and 02089500 (Kinston). All three major sites recorded flow that was regulated by Falls Lake Dam, designating a roughly 40-year period from early 1980s to current year, a flow regime that in addition to dam releases also include natural runoff volumes that would differ from a pre-dam H&H condition. Gage site field measurements were provided by USGS and separated into two general flow conditions, in-channel flow, and out-of-bank flow. For the purposes of this study, out-of-bank flow was most critical.

For site 02087500, roughly 60 field measurements were looked at, of which 9 were flagged as discrepancies. Only one of these field measurement discrepancies was based on the post-Falls Lake Dam flow regime, all others were based on a timespan of up to 67 years between observation dates. Furthermore, this one measurement (relative stage difference of 0.8-ft) had been taken at a relatively frequent discharge of nearly 6,000 cfs, barely out of channel conditions.

For site 02089000, roughly 60 field measurements were looked at, of which 11 were flagged as discrepancies. Again, only one of these field measurement discrepancies was based on the post-Falls Lake Dam flow regime, all others were based on a timespan of up to 84 years between observation dates. Similar to the Clayton site, this single observation (relative stage difference of 0.5-ft) was made during relatively frequent out-of-bank flows, at roughly 10,000 cfs.

For site 02089500, roughly 100 field measurements were looked at, of which 23 were flagged as discrepancies. Four of these field measurement discrepancies were based on the post-Falls Lake Dam flow regime, all others were based on a timespan of up to 69 years between observation dates. The largest of these post-Falls Lake Dam flows was roughly 9,400 cfs, representing a relatively frequent out-of-bank flow.

As a result of this assessment, the relatively low number of stage-discharge discrepancies, captured within the post-Falls Lake Dam flow regulated flow regime, signified that this type of gage and natural uncertainty was unlikely to be a significant engineering constraint in plan formulation.

3.1.2 Rainfall Data

Rainfall data for the events utilized in calibration and validation of the H&H models were obtained from the Wilmington District Water Management Server and also provided by the National Weather Service (NWS) Southeast River Forecast Center (SRFC). Data were obtained as National Oceanic and Atmospheric Administration (NOAA) Stage IV gridded precipitation in XMRG format. Stage IV is an hourly quality-controlled rainfall product available on a 4.0-kilometer (2.6-mile) grid across the United States. The hourly rainfall data in the XMRG file format was unpacked into the Standard Hydrologic Grid (SHG) format and spatially interpolated to a 500-meter grid using the gridloadXMRG program. The gridded data was then imported into the Meteorologic Visualization Utility Engine (HEC-MetVue) program and basin average hyetographs were created from the grid for each subbasin in the hydrology model (SAM, 2021).

In addition to streamflow sites, USGS provides a number of precipitation-recording stations in the upper basin, within Wake County (Crabtree Creek and Walnut Creek watersheds). Due to their limited applicability for basin-wide analysis, records were used as a comparison to the gridded rainfall data described in the preceding paragraph. Likewise, rain gage sites within the Community Collaborative Rain, Hail & Snow (CoCoRaHS) network were used to generally describe the precipitation impacts during historic flood events.

3.2 Topographic Data

Through a collaboration of various State agencies, namely North Carolina Emergency Management and North Carolina Department of Transportation, 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. All data included intensity values and was collected to support a 19.6 cm or 0.64-foot Non-Vegetated Vertical Accuracy (NVA) at a 95% confidence level (NCDOT, connect.ncdot.gov). Upon the conclusion of post-processing of LAS data, a digital elevation model (DEM) of last-return points was produced (bare-earth model). The data are referenced vertically to the North American Vertical Datum of 1988 (NAVD88) and horizontally to the North American Datum of 1983 (NAD83). The DEMs were provided as tiles in .tif format by USACE South Atlantic Wilmington (SAW) and mosaicked to form a continuous DEM for use in modeling and mapping. A similar topographic product was developed using previous State-collected LiDAR data circa 2005 to supplement the more computationally intensive QL2 set due to the large study area.

Channel surveys from multiple sources were used to enhance study area DEMs. Cross sectional geometry within stream banks were obtained from FEMA hydraulic modeling and were merged with LiDAR-derived overbank floodplain. 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 georeference older FEMA hydraulic models, with emphasis placed on assuring accuracy at structural stream crossings. Bathymetry was also utilized from previous USACE Continuing Authorities Program (CAP) efforts, such as the CAP1135 study near Goldsboro, NC (USACE, 2015). In the lower reaches of the Neuse River and within Pamlico Sound, bathymetry was supplemented with National Oceanic and Atmospheric Administration (NOAA) nautical charts. There were no new bathymetric surveys taken as part of this feasibility-level study.

3.3 Structural Data

The majority of hydraulic structures within the study extents were based on FEMA hydraulic modeling provided by the North 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 North Carolina Department of Transportation and USACE Wilmington District.

4 Historic Events

4.1 Overview

The following Table 10 provides a list historic flooding events prior to 2016 in the Neuse River basin, as compiled by USGS, and presented in a recent Hurricane Florence-related publication: *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, Open-File Report 2018-1172*:

Table 10. List of Historic Flood Events, Provided by USGS

Event Date	Quantified Impacts (state-wide)	Description
August, 1908	--	Set flood of record for upper portion of Neuse River basin.
September 15-17, 1933	Lives lost, 21; damages, $3 million	Storm tides set new peak stage, based on high-water marks in New Bern, NC.
September 17, 1945		Floods on upper Neuse.
October 15, 1954	Lives lost, 19; damage, $125 million	Hurricane Hazel, the costliest storm in the State's history to date.
August 12 and 17, 1955	Damage, $58 million	Hurricanes Connie and Diane. Estuaries of Neuse and Pamlico Rivers hardest hit.
September 5-6, 1996	Lives lost, 25; damages, $2.4 billion	Widespread rainfall totals of 5 to 10+ inches across central and eastern North Carolina. Substantial hurricane strength winds felt far inland.

4.2 Hurricane Matthew

In the fall of 2016, Hurricane Matthew caused significant damage to the State of North Carolina, both in economic and life-safety terms. The event resulted in damage estimates in North Carolina that exceeded $1.5 billion and nearly 30 deaths were attributed to the hurricane (NCSU, 2017). A roughly 15-year period of quiet tropical storm activity in much of the Neuse River basin, following the devastating 1999 Hurricane Floyd event, was abruptly ended in October of 2016.

Hurricane Matthew originated along the African coast in late September 2016. As a tropical wave, it quickly moved westward where near Barbados it became Tropical Storm Matthew. It eventually became a hurricane off the coast of South America and underwent rapid intensification by early October 2016. After impacting Haiti, Cuba, and the Bahamas, the storm was able to maintain Category 3 and 4 winds. There was a period of weakening as the hurricane made its way northwest along the eastern coast of Florida and had been downgraded to a Category 1 storm as it paralleled southern portions of the South Carolina coast. It made landfall just south of McClellanville, South Carolina on 8-October. Its path shifted back east where its center remained just offshore of North Carolina on 9-October.

Widespread showers and thunderstorms impacted the Neuse River basin over a nearly 48-hour period as the storm's western side circulated through the middle of the basin. Areas near Smithfield, Goldsboro, and Kinston experienced significant rainfall. CoCoRaHS rain gage stations near Goldsboro, Kinston, and New Bern, reported 13.3-, 16.5-, and 8.5-inches, respectively (SC ACIS, 2022). State-wide precipitation totals for Hurricane Matthew, as reported by NWS, is shown in Figure 16.

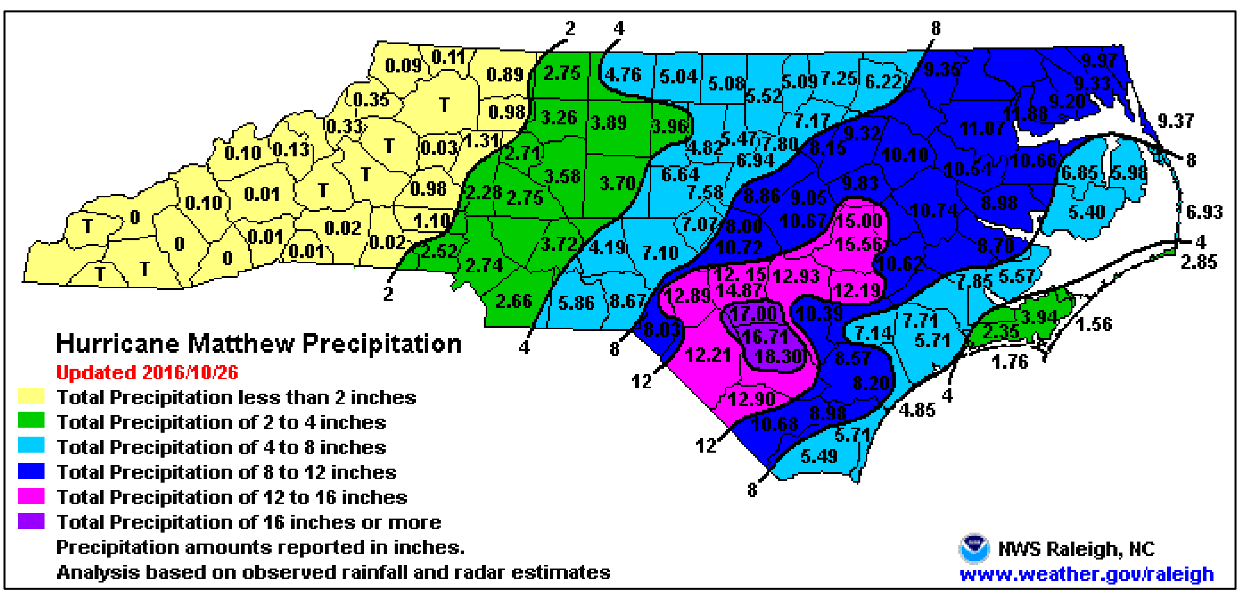

Figure 16. National Weather Service – Hurricane Matthew Precipitation

USGS reported new streamflow peaks of record for stream gages located at Neuse River near Goldsboro, NC (02089000) and Neuse River at Kinston, NC (02089500) (USGS, 2016). The stage-only stream gage at Smithfield, NC (02087570), set a new peak record from Hurricane Matthew, which exceeded the previous record set from Hurricane Floyd in 1999. The peak stage and discharge recorded at the Clayton stream gage (02087500) during the event were the second highest in the period of regulated record since the 1984 water year. Its highest observations were set during Hurricane Floyd in 1999.

The Falls Lake Dam reservoir elevation prior to the event was near elevation 251.7 ft, NAVD88. Releases from the project were reduced to near 100 cfs roughly 2.5 days prior to the storm's arrival in the lower basin. Discharge recorded immediately below the project was maintained at that minimum flow for approximately 15 days while the uncontrolled downstream portion of the basin responded to the hurricane event. Peak discharges were observed at Goldsboro and Kinston on 12-October and 14-October, respectively. The uncontrolled peak flows at Goldsboro and Kinston were 53,400 cfs and 38,200 cfs, respectively. The discrepancy between peak flows at these two locations suggested that significant overbank floodplain attenuation was characteristic of this segment of the Neuse River. by On 21-October, flood releases began from Falls Lake Dam. The releases would result in a secondary peak flow progressing downstream; however, it was purposefully delayed to not contribute to the much higher uncontrolled hydrograph peaks seen near Goldsboro and Kinston. Furthermore, the federal project flood releases were only a fraction of the uncontrolled peak flow, at 8% and 11% of the Goldsboro and Kinston peaks, respectively.

4.3 Hurricane Florence

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

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

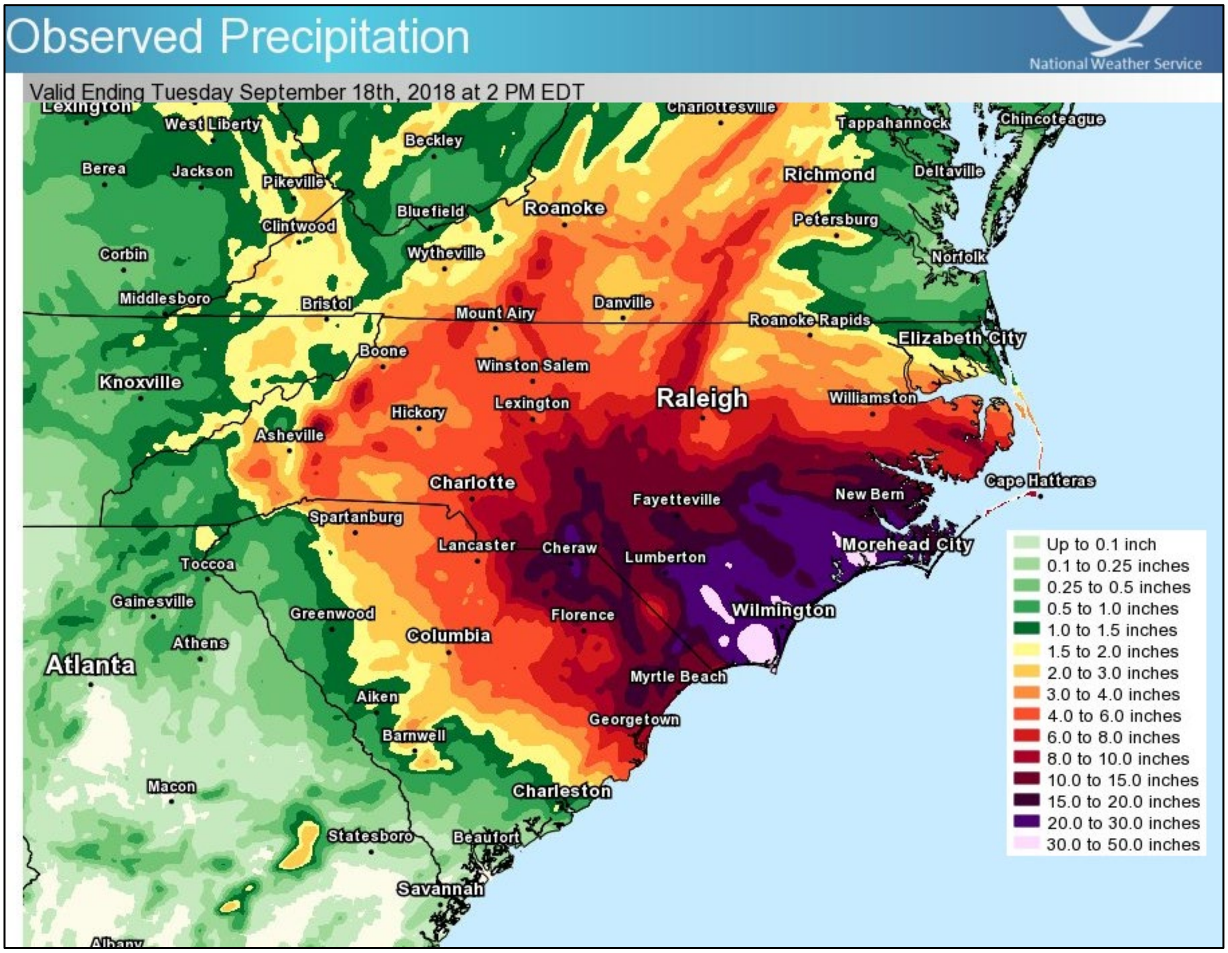

Figure 17. National Weather Service - Hurricane Florence Observed Precipitation

USGS reported that 28 stream gage sites in North Carolina and South Carolina show a new peak record following Hurricane Florence. Within the Neuse River basin, USGS site 02092500, Trent River near Trenton, NC (67 period-of-record) had a new peak of record discharge of 67,700 cfs and a peak gage height of 24.23 feet. USGS estimated this to be less frequent than a 0.002-AEP event. Other gage sites within the basin that had a new peak of record included Mountain Creek at SR1617 near Bahama and Ellerbe Creek near Gorman (USGS, 2018).

The Falls Lake Dam reservoir elevation prior to the event was near elevation 251.6 ft, NAVD88. Releases from the project were near 100 cfs. Discharge recorded immediately below the project were maintained at that minimum flow for approximately 12 days while the uncontrolled downstream portion of the basin responded to the hurricane event. Peak discharges were observed at Goldsboro and Kinston on 18-September and 21-

September, respectively. The uncontrolled peak flow at Goldsboro and Kinston were 36,700 cfs and 30,500 cfs, respectively.

Effects of reservoir performance for Hurricane Florence were analyzed through a NCDOT and NCSU joint effort, performed independent and impartial to USACE. The following Figure 18 was provided in the 2020 Flood Abatement Assessment for Neuse River Basin; it documented the recorded discharge for the Hurricane Florence hydrograph at multiple stream gage locations in the basin and were compared to Fall Lake Dam releases.

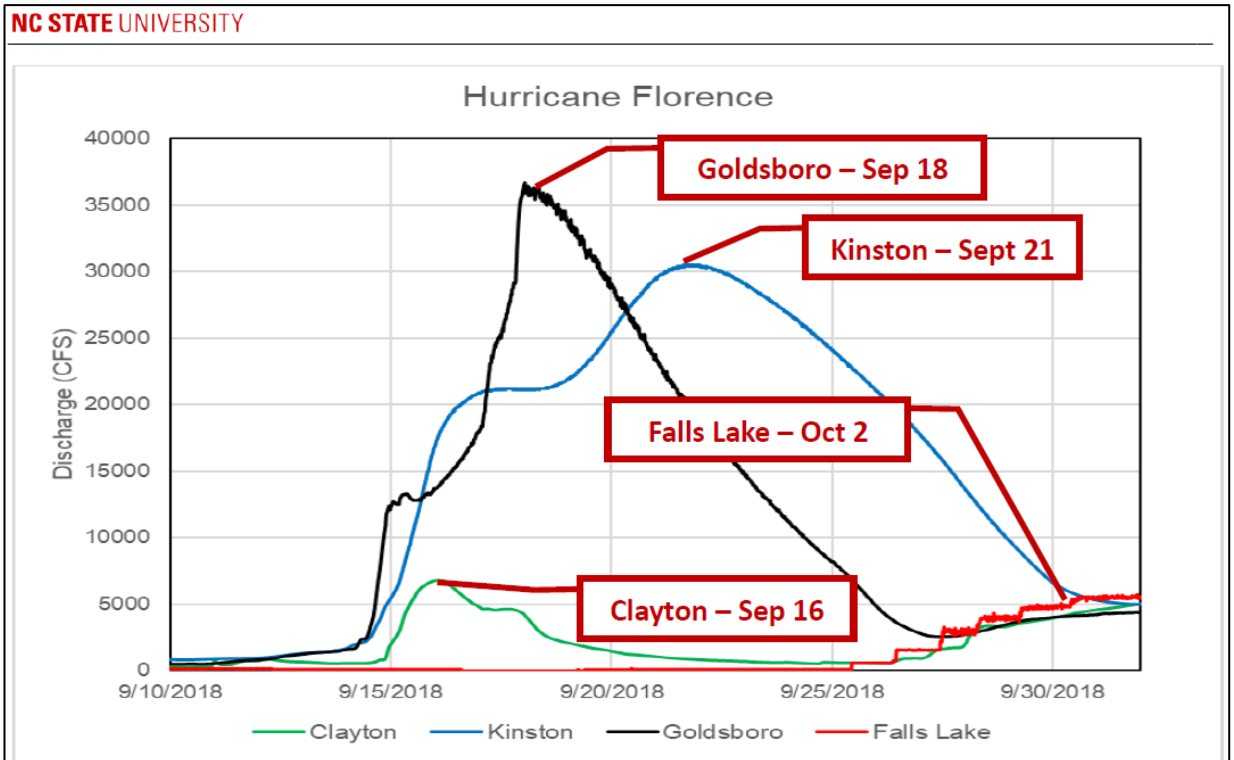

Figure 2-8. Discharges at Falls Lake in relation to discharge observed at downstream gages in the Neuse River basin during Hurricane Florence.

Figure 18. Hydrograph response to Hurricane Florence, presented in NCDOT, 2020 Report

As seen in the provided figure above, releases from Fall Lake Dam were timed such that the uncontrolled hydrographs downstream had peaked and began receding by the time flood releases from the project reached Goldsboro and Kinston. As such, the federal project played virtually no role in the peak flows and associated flood depths caused by the hurricane. Over the total hydrograph duration, eventual peak discharge released from the dam accounted for a fraction of the uncontrolled flow, at 15% of the Goldsboro peak and 18% of the Kinston peak (NCDOT, 2020).

For additional non-biased assessment of Falls Lake Dam operations and its effects during Hurricane Florence, please refer to the referenced NCDOT/NCSU document in the preceding paragraphs.

5 Existing Conditions

5.1 Hydrology

The total Neuse River basin is approximately 6,200 square miles which includes 770 square miles above the Falls Lake Dam federal project as well as over 400 square miles of drainage area within the Pamlico Sound estuary. For the Neuse River basin FRM study, the upper limits of the hydrologic model extended to the headwaters of the Eno and Flat River. The upstream limit of the Neuse River mainstem is the downstream face of the Falls Lake Dam. Major tributary subbasins in the hydrologic model study area include: Crabtree Creek, Walnut Creek, Swift Creek, Middle Creek, Black Creek, Mill Creek, Falling Creek, Little River, Big Ditch, Bear Creek, Southwest Creek, Contentnea Creek, Little Contentnea Creek, Nahunta Swamp, Hominy Swamp Creek, Swift Creek (Craven Co.), and Trent River. Select major tributaries in the basin are shown in Figure 19.

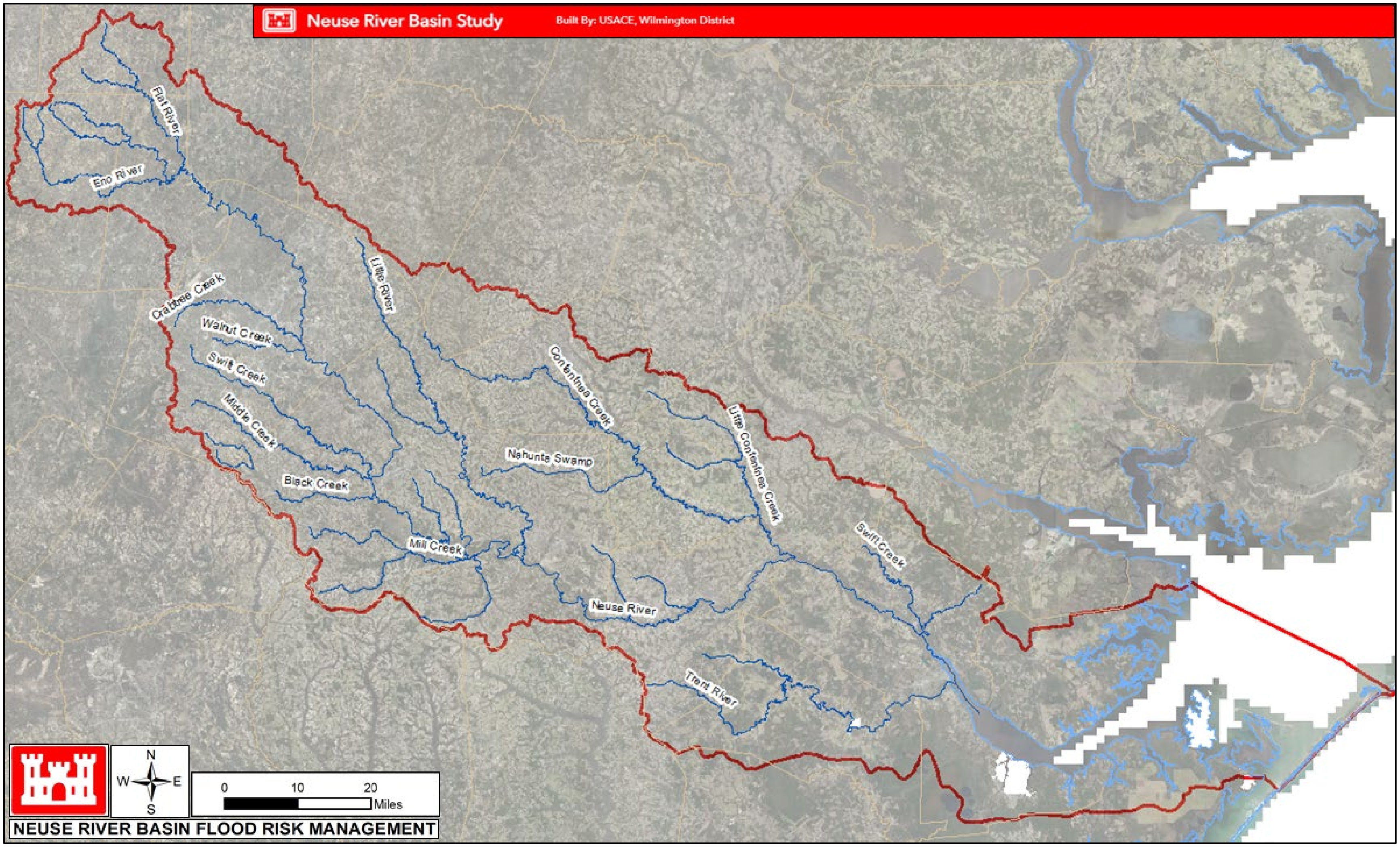

Figure 19. Select Neuse River Basin Tributaries

5.1.1 Hydrology Model Background

A total of five separate planning-level hydrologic models were developed to assess existing conditions in the Neuse River basin, using the USACE Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) software, version 4.8. Given the Neuse River basin's large size and number of tributaries, as well as variety in urban landscape, it was decided that multiple separate models would best serve the intent in formulating local flood risk management measures. One comprehensive basin model was developed for hydrologic assessment along the mainstem of the Neuse River as well as the following headwaters and major tributaries: Eno River, Little River (Durham Co.), Flat River, Walnut Creek, Swift Creek (Johnston Co.), Middle Creek, Little River (Wayne Co.), Swift Creek (Craven Co.), Contentnea Creek, and Trent River. 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.

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. Upon review of these areas, it was determined that subbasin-specific HEC-HMS modeling would be required. 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 Creek required much smaller subbasin delineations in area to account for the high density of streams, impoundments, and confluences.

Four subbasin-specific HEC-HMS models were developed in parallel with the basin-wide model. These smaller scale modeling footprints included the following Neuse River tributaries: Crabtree Creek in Raleigh, Hominy Swamp Creek in Wilson, Big Ditch in Goldsboro, and Adkins Branch in Kinston. Notable, these subbasin-specific areas were also included in the Neuse River mainstem basin model, albeit in lesser detail, especially for the Crabtree Creek watershed.

5.1.2 Model Overview

5.1.2.1 Basin Delineation

The USACE Corps Water Management System (CWMS) HEC-HMS Neuse River model was primarily developed to allow for efficient water management within the basin; therefore, basin delineation was mostly limited to known USGS gage locations. The CWMS model was determined to have too few subbasin elements for this feasibility-level evaluation and was not utilized for basin delineation. Furthermore, development the CWMS Neuse River model was part of a pilot study for CWMS implementation, and as such, had not undergone modernization or appreciable improvements for multiple years. Subbasins for the Neuse River Mainstem HEC-HMS model were verified and manually re-delineated from the existing AECOM model, developed by the State of North Carolina in 2018, using HEC-HMS 4.8 Geographic Information System (GIS) features and Hydrologic Unit Code 10 (HUC-10) subbasins. QL2 LiDAR was determined to be too computationally intensive for processing within HEC-HMS due to the large basin size, and the older LiDAR dataset was utilized. A number of subbasins in the AECOM model were merged together due to their relatively small size and to reduce the amount of uncertainty during the calibration process. For comparison, AECOM model subbasin areas ranged from 0.4 to 316 square miles, with an average of 50 square miles. The Neuse River mainstem model subbasins ranged 0.2 to 365 square miles, with an average of 90 square miles. In addition, subbasins were delineated below the outlet point within the AECOM model, to include the lower Neuse River and major tributaries, Swift Creek and Trent River. While the AECOM model did not include basin elements for the drainage area above Falls Lake, subbasins were delineated at USGS gage locations in the Neuse River mainstem model. A total of 56 subbasins were delineated for the Neuse River basin mainstem model. The total basin area was roughly 5,050 square miles. The final subbasin delineation for the Neuse River mainstem HEC-HMS model is shown in Figure 20.

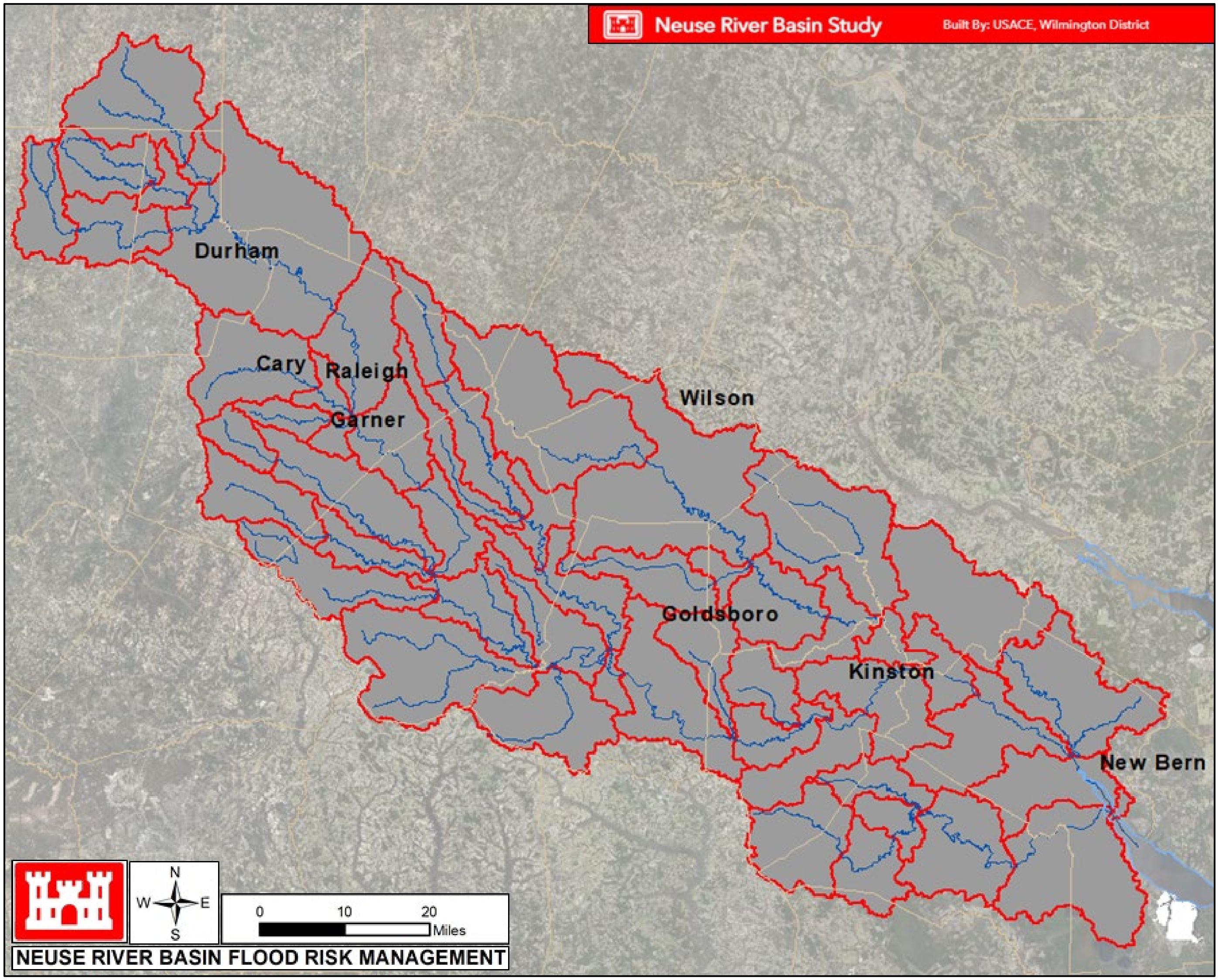

Figure 20. Neuse River Mainstem HEC-HMS Subbasins

The existing Crabtree Creek HEC-HMS model, developed by AECOM, included a detailed delineation of subbasins. No changes were made to this delineation during utilization for the Neuse River basin feasibility study. The following description of their delineation process is being provided as follows:

> Basin delineations and drainage areas were determined using a 50’ x 50’ grid size digital elevation model (DEM) generated from 3D points and breaklines provided by the City of Raleigh. Drainage areas computed using the 50’x 50’ DEM often differ from published values at USGS gage locations. Such differences are usually the result of the difference in resolution of the base terrain data used to delineate drainage boundaries. In North Carolina, published USGS drainage areas are usually determined by manual delineation using 1:24,000 or 1:62,500 scale topographic maps. In order to maintain consistency, drainage areas computed from the 50’x 50’ DEM were used in all analyses in this study (AECOM, 2010).

A total of 252 subbasins were delineated for the Crabtree Creek basin HEC-HMS model. The total basin area was roughly 145 square miles. Crabtree Creek subbasin delineation is shown in Figure 21.

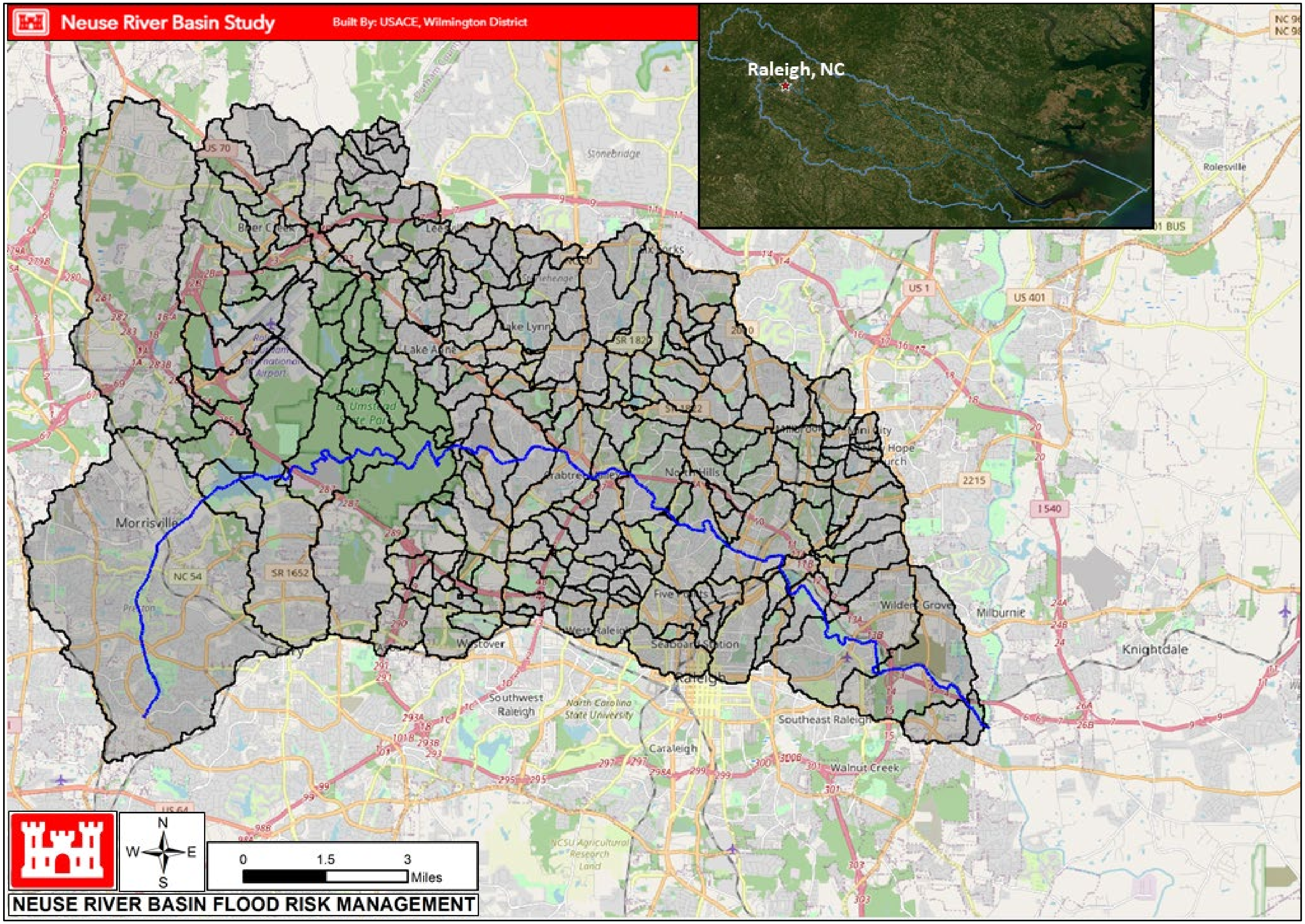

Figure 21. Crabtree Creek Subbasin Delineation (AECOM)

Three separate HEC-HMS models were developed for Hominy Swamp Creek, Big Ditch, and Adkins Branch. These models were much smaller and were able to better utilize QL2 LiDAR in their delineation process. The built-in GeoHMS equivalent tools of HEC-HMS 4.8 were utilized to process the terrain data. The delineation process underwent multiple iterations before being finalized due to the highly urbanized watersheds in Goldsboro and Kinston. A total of 17 subbasins were delineated for Hominy Swamp Creek with a total basin area of about 11.5 square miles. The outlet point of the Hominy Swamp Creek model was approximately 2 miles upstream of the confluence with Contentnea Creek. The final Hominy Swamp Creek subbasin delineation is shown in Figure 22. A total of 12 subbasins was delineated for Big Ditch with a total basin area of 3.0 square miles. The final Big Ditch subbasin delineation is shown in Figure 23. A total of 14 subbasin was delineated for Adkins Branch with a total basin area of 6.0 square miles. The final Adkins Branch subbasin delineation is shown in Figure 24.

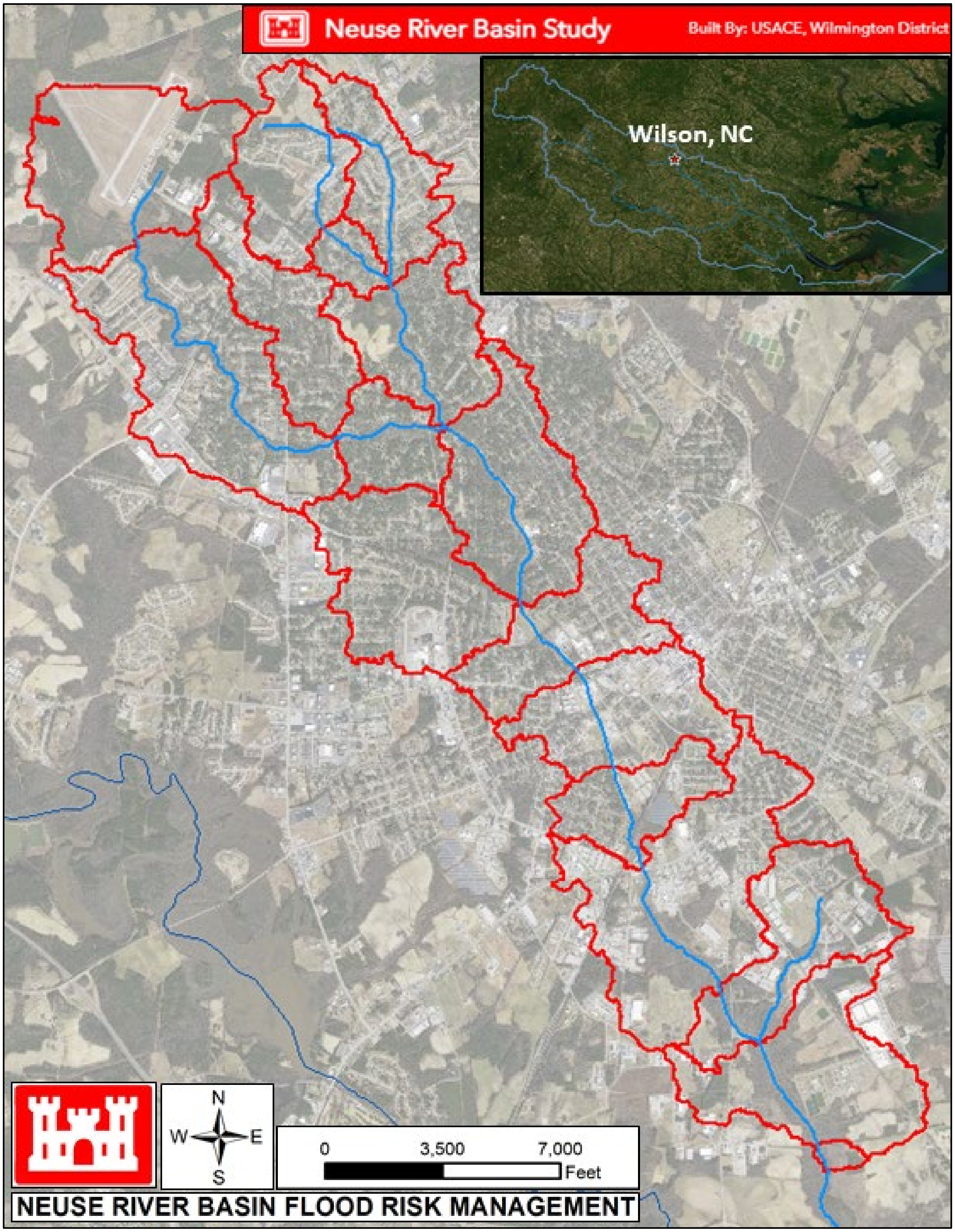

Figure 22. Hominy Swamp Creek Subbasin Delineation

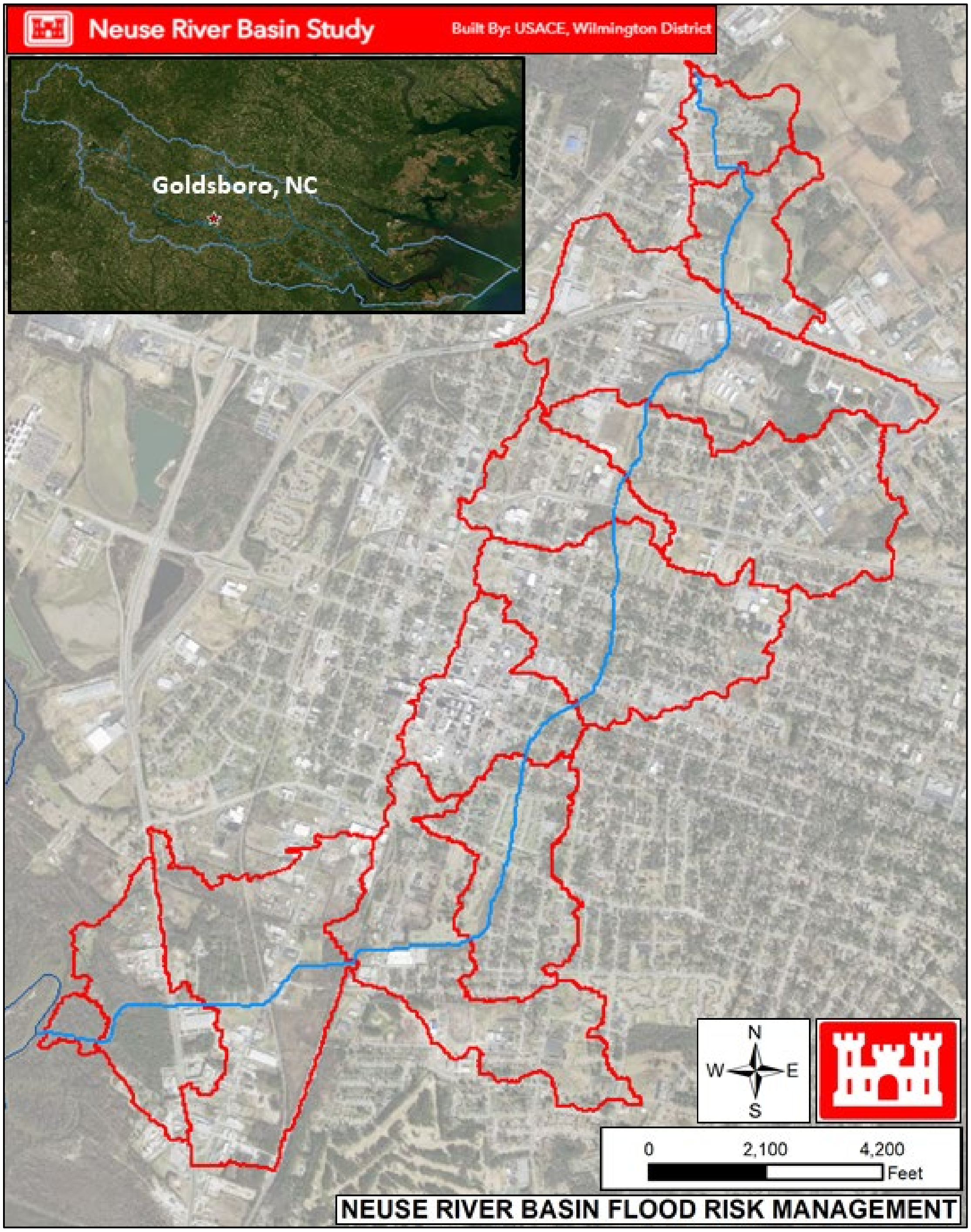

Figure 23. Big Ditch Subbasin Delineation

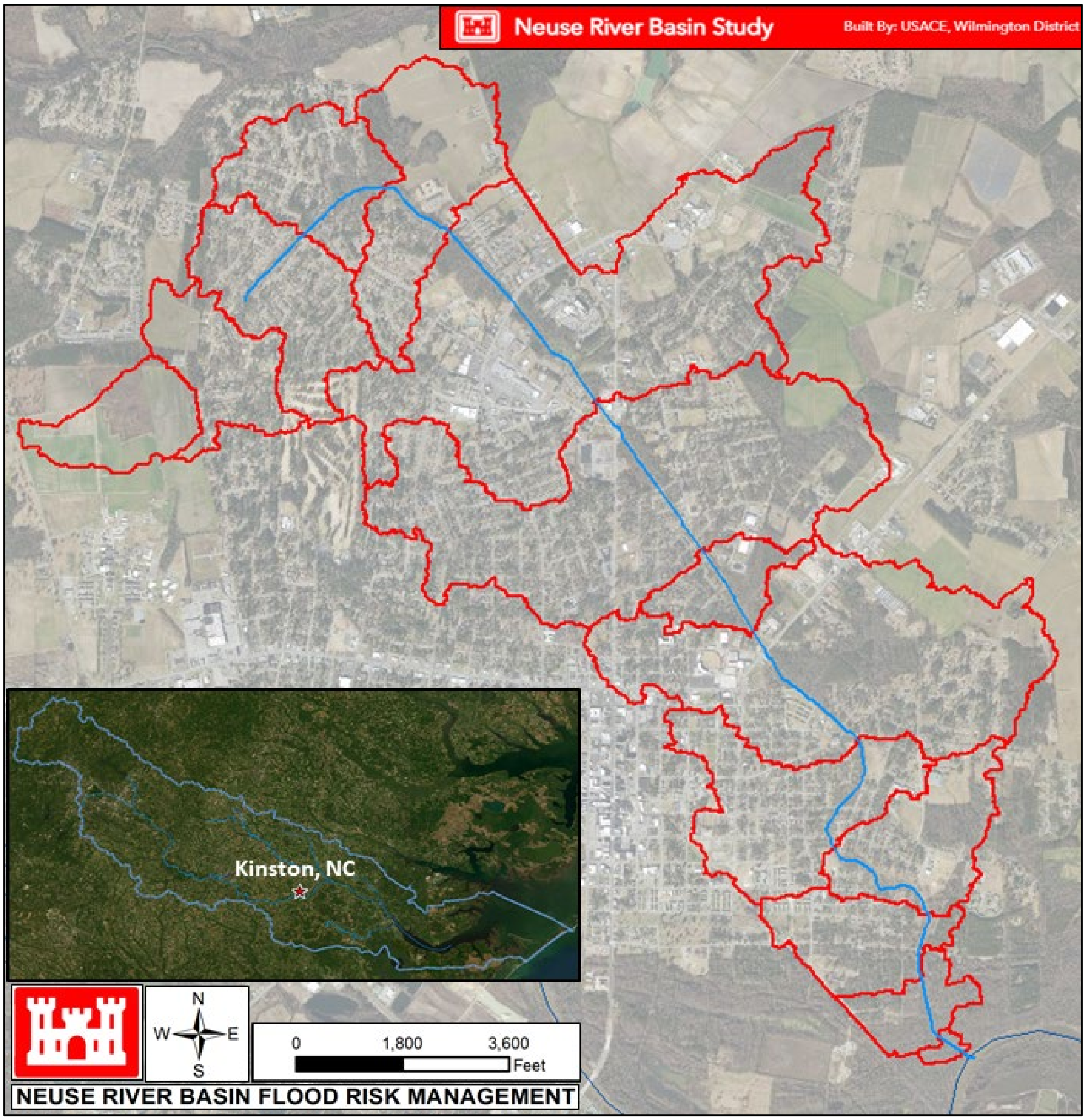

Figure 24. Adkins Branch Subbasin Delineation

5.1.2.2 Rainfall Losses

For all five 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. For the Crabtree Creek model, land use data was developed from data contained in Wake County, North Carolina tax parcel data shapefiles (AECOM, 2011). 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. The composite curve number matrix assumed for this assessment is listed in Table 11. The curve number matrix utilized for the Crabtree Creek model, consistent with the land use classifications specific to Wake County is listed in Table 12.

Table 11. SCS Composite Curve Number Matrix

	Hydrologic Soil Group			
Type	A	B	C	D
Open Water	99	99	99	99
Developed, Open Space	39	61	74	80
Developed, Low Intensity	51	68	79	84
Developed, Medium Intensity	61	75	83	87
Developed, High Intensity	89	92	94	95
Barren Land	63	77	85	88
Deciduous Forest	36	60	73	79
Evergreen Forest	30	55	70	77
Mixed Forest	36	60	73	79
Shrub/Scrub	35	56	70	77
Herbaceuous	49	69	79	84
Hay/Pasture	39	61	74	80
Cultivated Crops	64	75	82	85
Woody Wetlands	36	56	70	77
Emergent Herbaceuous Wetlands	72	80	87	93

Table 12. Curve Number Matrix used in Crabtree Creek HEC-HMS Model (AECOM)

Table 2-3: Curve Number Matrix

Land Use	SCS Hydrologic Soil Classification					
	A	A/D	B	B/D	C	D
Woods	30	40	55	60	70	77
Open Space	39	43	61	68	74	80
Water	99	99	99	99	99	99
Wetland/Brush	35	45	56	60	70	77
Streets	98	98	98	98	98	98
Cultivated Agriculture Straight Row - Good	67	72	78	82	85	89
Institutional	54	62	70	75	80	85
Urban districts: Industrial	81	83	88	90	91	93
Light Residential (1 acre)	51	60	68	72	79	84
Medium Residential (1/3rd acre)	57	65	72	77	81	86
Heavy Residential (1/8th acre or less)	77	81	85	87	90	92
Urban districts: Commercial	89	91	92	93	94	95

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.

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 calibration events. Final subbasin curve number values across all HEC-HMS models are listed in Table 13 through Table 17.

Table 13. Neuse River Mainstem Basin Final Subbasin Curve Number

Subbasin	Initial Abstraction (in)	Curve Number
B10	1.0	66.9
B11	0.7	74.0
B15	1.0	67.5
B16	0.8	71.2
B19	0.9	69.2
B21	1.0	67.6
B23	1.1	65.5
B24	0.9	70.0
B25	1.5	57.2
B26	0.8	72.2
B28a	0.8	70.7
B28b	0.8	71.4
B29a	1.0	66.3
B29b	0.8	71.5
B30	0.9	67.9
B31	1.0	66.7
B32	0.8	71.8
B35	1.1	63.5
B37	1.0	66.0
B39a	0.7	75.2
B39b	0.8	72.3
B40	0.8	71.0
B41	0.8	71.1
B43	0.9	68.0
B44	1.1	64.9
B46	0.8	70.5
B47	0.9	67.9
B49	1.1	65.1

Subbasin	Initial Abstraction (in)	Curve Number
B5	0.6	77.4
B50	1.3	60.3
B52	1.1	64.8
B53	0.9	68.1
B54	1.4	59.2
B55	1.0	66.4
B56	1.1	64.3
B59	0.8	71.7
B6	0.9	69.5
B60	1.0	66.0
B60b	1.2	62.9
B61	1.3	60.8
B62	1.0	65.9
B62d	1.1	63.5
B62f	0.9	68.1
B62h	1.2	62.4
B63a	0.9	68.7
B63d	1.5	57.9
B64	1.0	67.3
B66a	1.0	66.0
B67	0.5	79.5
B68a	0.8	71.4
B68b	0.9	68.3
B68c	1.0	66.5
B68d	0.8	70.7
B68e	0.5	78.8
B68f	0.6	76.2
B69	1.1	63.7

Table 14. Crabtree Creek Basin Final Subbasin Curve Number

Subbasin	Initial Abstraction (in)	Curve Number	Subbasin	Initial Abstraction (in)	Curve Number	Subbasin	Initial Abstraction (in)	Curve Number
BASIN16	0.45	81.7	HSC44	0.68	74.6	MSH41	0.74	73.0
BASIN18	0.95	67.7	HSC45	0.70	74.2	MSH42	0.75	72.7
BASIN19	0.64	75.8	HSC47	0.65	75.6	MSH43	0.86	70.0
BASIN2	0.89	69.2	HSC48	0.71	73.9	PH10	0.87	69.7
BASIN20	1.02	66.3	HSC52	0.68	74.5	PH11	0.75	72.7
BASI17	0.72	73.5	HSC54	0.72	73.4	PH3	0.80	71.5
BASI24	0.73	73.2	HSC58	0.44	81.8	PH4	0.85	70.1
BB1	0.91	68.7	HSC59	0.96	67.6	PH5	0.74	73.0
BB10	0.58	77.5	HSC60	0.28	87.8	PH6	0.65	75.5
BB11	0.78	72.0	HSC61	0.38	84.0	PH7	0.74	73.0
BB12	0.78	72.0	HSC62	0.70	74.0	PH9	0.64	75.9
BB13	0.93	68.3	HSC63	0.46	81.3	RC1	1.49	57.4
BB2	0.77	72.3	HSC64	0.62	76.3	RC10	1.02	66.2
BB3	0.99	66.8	HSC65	0.54	78.8	RC11	1.51	56.9
BB5	0.90	69.0	HSC66	0.58	77.4	RC12	1.03	66.1
BB6	0.87	69.6	HSC67	0.56	78.1	RC13	1.26	61.4
BB7	0.89	69.1	HSC68	0.52	79.4	RC15	1.33	60.1
BB8	0.74	72.9	LBC1	0.92	68.4	RC16	1.26	61.4
BB9	0.83	70.6	LBC10	0.87	69.6	RC17	1.65	54.8
BrB-1	0.95	67.8	LBC11	1.21	62.3	RC18	1.21	62.3
BrB-2	0.77	72.2	LBC12	0.87	69.6	RC19	1.02	66.3
BVR1	0.93	68.3	LBC13	0.77	72.1	RC2	1.46	57.8
BVR10	0.90	68.9	LBC2	0.44	82.1	RC20	1.12	64.1
BVR11	1.03	66.1	LBC3	0.77	72.1	RC21	1.37	59.4
BVR12	1.04	65.9	LBC4	0.98	67.2	RC3	1.47	57.7
BVR13	1.01	66.4	LBC5	0.79	71.8	RC4	0.84	70.4
BVR14	0.97	67.2	LBC6	0.66	75.1	RC5	1.12	64.1
BVR15	1.07	65.1	LBC7	0.71	73.7	RC6	0.95	67.7
BVR16	1.14	63.7	LBC75	0.82	71.0	RC7	0.94	68.1
BVR2	1.01	66.5	LBC76	0.74	72.9	RC8	0.90	69.0
BVR3	0.96	67.6	LBC8	0.89	69.1	RC9	1.46	57.8
BVR4	0.90	69.1	LBC9	0.68	74.6	SCY1	1.13	64.0
BVR5	0.89	69.3	MC10	0.71	73.8	SYCT13	1.58	55.9
BVR6	0.94	68.1	MC100	0.93	68.2	SYC10	1.80	52.6
BVR7	1.08	64.9	MC101	1.04	65.7	SYC11	1.08	64.9

BVR8	0.89	69.1	MC102	0.75	72.7	SYC12	1.66	54.7
BVR9	0.98	67.2	MC103	1.06	65.4	SYC13	1.01	66.5
CTC125	0.64	75.9	MC11	1.44	58.1	SYC14	0.45	81.7
CTC126	0.58	77.5	MC110	0.79	71.7	SYC15	0.75	72.7
CTC22	0.79	71.8	MC124	0.99	66.9	SYC16	0.57	77.7
CTC23	0.60	77.0	MC13	0.69	74.3	SYC17	1.14	63.7
CTC25	1.11	64.4	MC15	0.86	69.9	SYC18	1.08	64.9
CTC26	1.35	59.7	MC16	1.14	63.7	SYC19	1.24	61.7
CTC27	1.22	62.2	MC18	0.95	67.9	SYC2	1.23	61.8
CTC28	1.83	52.2	MC19	1.00	66.7	SYC20	1.05	65.7
CTC29	1.32	60.3	MC2	0.97	67.4	SYC21	0.87	69.7
CTC30	1.61	55.4	MC20	0.81	71.1	SYC22	0.96	67.6
CTC31	0.76	72.5	MC200	1.15	63.5	SYC23	0.87	69.8
CTC32	0.78	71.9	MC201	0.94	68.0	SYC24	0.89	69.1
CTC33	0.86	70.0	MC21	1.02	66.3	SYC3	1.49	57.4
CTC34	0.76	72.5	MC22	0.95	67.8	SYC4	1.54	56.5
CTC35	0.94	68.0	MC23	0.86	69.9	SYC5	1.58	55.9
CTC35A	0.86	69.9	MC24	1.37	59.3	SYC6	1.87	51.6
CTC35B	0.99	67.0	MC25	1.01	66.5	SYC7	1.86	51.8
CTC36	0.87	69.8	MC26	1.06	65.5	SYC8	1.60	55.6
CTC38	0.93	68.3	MC27	0.99	66.9	SYC9	1.93	50.8
CTC39	0.93	68.3	MC3	0.91	68.7	SYT1-4	0.84	70.5
CTC40	0.92	68.5	MC5	0.97	67.3	SYT1_1	1.64	54.9
CTC41	0.96	67.6	MC7	0.99	66.9	SYT1_2	1.57	56.0
CTC42	0.91	68.7	MC8	0.81	71.1	SYT2-1	0.92	68.6
HC1	1.56	56.2	MC9	0.51	79.6	SYT2-2	0.58	77.4
HC10	1.06	65.3	MSH170	0.36	84.7	SYT2-3	0.65	75.5
HC11	0.95	67.7	MSH180	0.34	85.5	SYT2-4	1.03	65.9
HC12	0.95	67.7	MSH20	0.35	85.2	SYT2-5	1.07	65.1
HC13	1.04	65.7	MSH21	0.57	77.9	SYT2-6	0.93	68.3
HC14	0.78	71.9	MSH22	0.45	81.6	SYT2-7	0.89	69.3
HC2	1.47	57.7	MSH23	0.35	85.1	TC250	0.89	69.2
HC3	1.44	58.2	MSH24	0.63	76.1	TC251	0.73	73.2
HC4	0.71	73.9	MSH25	0.47	81.1	TC252	0.90	69.0
HC5	1.10	64.6	MSH26	0.59	77.2	TC253	1.32	60.3
HC6	0.57	77.9	MSH27	0.70	74.2	TC254	1.04	65.7
HC7	1.22	62.2	MSH28	0.62	76.4	TC255	0.96	67.6
HC8	1.04	65.8	MSH29	0.45	81.6	TC256	1.19	62.7
HC9	0.83	70.6	MSH30	0.64	75.9	TC257	0.76	72.6
HSC29	0.60	76.9	MSH31	0.46	81.2	TC258	0.83	70.8

HSC30	0.50	80.0	MSH32	0.37	84.3	TC259	0.73	73.3
HSC33	0.68	74.6	MSH33	0.48	80.7	TC260	0.85	70.2
HSC34	0.74	73.1	MSH34	0.47	81.1	TC261	0.66	75.1
HSC36	0.68	74.5	MSH35	0.46	81.2	TC262	0.88	69.4
HSC37	0.34	85.3	MSH36	0.48	80.5	TC263	0.93	68.3
HSC38	0.67	74.9	MSH37	0.58	77.6	TC264	0.87	69.6
HSC39	0.59	77.2	MSH38	0.29	87.5	TC265	1.04	65.7
HSC40	0.63	76.0	MSH39	0.68	74.5	TC266	0.90	69.0
HSC43	0.58	77.6	MSH40	0.77	72.2	TC267	0.89	69.2

Table 15. Hominy Swamp Creek Basin Final Subbasin Curve Number

Subbasin	Initial Abstraction (in)	Curve Number
s1	0.54	78.8
s10	0.49	80.3
s11	0.45	81.7
s12	0.60	76.8
s14	0.85	70.2
s2	0.40	83.4
s21	0.61	76.6
s3	0.49	80.3
s30	0.42	82.5
s32	0.74	72.9
s34	0.86	69.8
s4	0.72	73.6
s5	0.66	75.2
s51	0.85	70.1
s6	0.63	76.1
s7	0.65	75.4
s8	0.65	75.4

Table 16. Big Ditch Basin Final Subbasin Curve Number

Subbasin	Initial Abstraction (in)	Curve Number
s1	0.42	82.5
s10	0.53	79.1
s12	0.19	91.5
s13	0.38	84.0
s14	0.38	83.9
s16	0.50	80.0
s2	0.06	97.2
s3	0.22	90.1
s4	0.44	82.0
s45	0.55	78.4
s53	0.77	72.2
s54	0.36	84.9

Table 17. Adkins Branch Basin Final Subbasin Curve Number

Subbasin	Initial Abstraction (in)	Curve Number
s1	0.34	85.3
s10	0.31	86.6
s12	0.44	82.0
s13	0.60	76.9
s14	0.06	96.9
s16	0.49	80.2
s2	0.37	84.5
s3	0.50	80.0
s4	0.45	81.5
s45	0.39	83.8
s53	0.00	99.0
s54	0.25	88.8
s8	0.38	84.1
s9	0.24	89.4

5.1.2.3 Subbasin Response

Transform methods used within the separate models were chosen based on the availability of calibration data and overall basin size and complexity. For the three smaller basin models, Hominy Swamp Creek, Big Ditch, and Adkins Branch, the SCS Unit Hydrograph method was used. Lag time values were derived from the following time of concentration equation (1):

$$T_c = 2.2 * \left(\frac{L * L_C}{\sqrt{Slope_{10-85}}}\right)^{0.3}$$

Longest flow paths (L), centroidal flow paths (Lc), and slope parameters were estimated using the GIS features within HEC-HMS 4.8. Values from this equation were multiplied by 0.6 to equate an approximate lag time.

The SCS Unit Hydrograph method was also used in the Crabtree Creek HEC-HMS model. Lag time values were estimated using the SCS TR-55 method. Method requirements of overland flow, shallow concentrated flow, open channel flow, and lake flow were developed through the use of geospatial analysis and a collection of survey cross sections to calculate the channel component.

The Neuse River mainstem HEC-HMS model used the Clark Unit Hydrograph transform method. It was considered the most compatible method for use with the gridded precipitation format of calibration and validation events. Clark unit hydrograph values were estimated using equation (1) above, and the following storage coefficient relationship:

$$\frac{R}{T_c + R} = 0.65$$

The equation above represented a proportional relationship between Time of Concentration and the Clark Method’s storage coefficient, R, as described in the HEC-HMS user’s manual. Several references suggested that initial R value can reasonably range between 0.5 and 0.7 (Russel et al., 1979, Storey et al., 2009, and USACE, 2015). Therefore, an initial value of 0.65 was chosen.

Initial parameter values for the transform methods using the equations above for all HEC-HMS models were adjusted during calibration to best fit observed data. Final subbasin transform method values for each HEC-HMS model are listed in Table 18 through Table 22.

Table 18. Crabtree Creek Basin Final Subbasin Transform Parameters

Subbasin	Lag Time (min)	Subbasin	Lag Time (min)	Subbasin	Lag Time (min)
BASIN16	4	HSC44	17	MSH41	19
BASIN18	50	HSC45	18	MSH42	26
BASIN19	24	HSC47	27	MSH43	18
BASIN2	21	HSC48	7	PH10	41
BASIN20	21	HSC52	17	PH11	22
BASI17	140	HSC54	14	PH3	22
BASI24	26	HSC58	13	PH4	29
BB1	12	HSC59	8	PH5	17
BB10	14	HSC60	7	PH6	13
BB11	10	HSC61	8	PH7	13
BB12	14	HSC62	10	PH9	18
BB13	13	HSC63	4	RC1	26
BB2	8	HSC64	10	RC10	16
BB3	9	HSC65	11	RC11	23
BB5	15	HSC66	14	RC12	30
BB6	12	HSC67	9	RC13	33
BB7	19	HSC68	13	RC15	18
BB8	19	LBC1	36	RC16	4
BB9	23	LBC10	46	RC17	18
BrB-1	25	LBC11	20	RC18	34
BrB-2	10	LBC12	43	RC19	15
BVR1	16	LBC13	75	RC2	38
BVR10	7	LBC2	18	RC20	17
BVR11	12	LBC3	31	RC21	10
BVR12	10	LBC4	14	RC3	21
BVR13	17	LBC5	31	RC4	33
BVR14	8	LBC6	36	RC5	23
BVR15	13	LBC7	15	RC6	18
BVR16	15	LBC75	39	RC7	19
BVR2	14	LBC76	35	RC8	4
BVR3	5	LBC8	52	RC9	29
BVR4	12	LBC9	24	SCY1	4
BVR5	9	MC10	14	SYCT13	30
BVR6	16	MC100	19	SYC10	10
BVR7	11	MC101	9	SYC11	48
BVR8	9	MC102	4	SYC12	22

BVR9	18	MC103	8	SYC13	31
CTC125	35	MC11	15	SYC14	19
CTC126	19	MC110	5	SYC15	22
CTC22	92	MC124	19	SYC16	19
CTC23	15	MC13	14	SYC17	17
CTC25	45	MC15	25	SYC18	22
CTC26	10	MC16	4	SYC19	31
CTC27	32	MC18	16	SYC2	15
CTC28	30	MC19	29	SYC20	18
CTC29	79	MC2	18	SYC21	17
CTC30	14	MC20	19	SYC22	13
CTC31	25	MC200	4	SYC23	13
CTC32	25	MC201	19	SYC24	10
CTC33	26	MC21	17	SYC3	25
CTC34	21	MC22	10	SYC4	21
CTC35	31	MC23	7	SYC5	18
CTC35A	4	MC24	4	SYC6	24
CTC35B	11	MC25	16	SYC7	17
CTC36	33	MC26	16	SYC8	22
CTC38	4	MC27	26	SYC9	24
CTC39	26	MC3	12	SYT1-4	18
CTC40	59	MC5	21	SYT1_1	22
CTC41	39	MC7	41	SYT1_2	24
CTC42	22	MC8	14	SYT2-1	16
HC1	29	MC9	2	SYT2-2	18
HC10	6	MSH170	22	SYT2-3	34
HC11	13	MSH180	15	SYT2-4	29
HC12	11	MSH20	11	SYT2-5	41
HC13	9	MSH21	9	SYT2-6	5
HC14	12	MSH22	38	SYT2-7	20
HC2	18	MSH23	10	TC250	10
HC3	9	MSH24	25	TC251	15
HC4	9	MSH25	17	TC252	31
HC5	14	MSH26	15	TC253	13
HC6	12	MSH27	11	TC254	16
HC7	33	MSH28	25	TC255	16
HC8	10	MSH29	11	TC256	19
HC9	4	MSH30	23	TC257	28
HSC29	9	MSH31	5	TC258	4
HSC30	14	MSH32	13	TC259	5
HSC33	15	MSH33	20	TC260	30

HSC34	15	MSH34	14	TC261	13
HSC36	15	MSH35	18	TC262	15
HSC37	16	MSH36	16	TC263	4
HSC38	15	MSH37	18	TC264	24
HSC39	16	MSH38	6	TC265	11
HSC40	17	MSH39	30	TC266	19
HSC43	19	MSH40	25	TC267	16

Table 19. Hominy Swamp Creek Basin Final Subbasin Transform Parameters

Subbasin	Lag Time (min)
s1	151
s10	93
s11	130
s12	87
s14	110
s2	107
s21	96
s3	78
s30	66
s32	92
s34	71
s4	82
s5	104
s51	49
s6	84
s7	127
s8	102

Table 20. Neuse River Mainstem Basin Final Subbasin Transform Parameters

Subbasin	Time of Concentration (hr)	Storage Coefficient (hr)	Subbasin	Time of Concentration (hr)	Storage Coefficient (hr)
B10	6.3	9.8	B5	10.9	10.3
B11	7.1	9.8	B50	9.7	38.8
B15	14.8	34.3	B52	7.5	13.7
B16	7.3	10.1	B53	8.0	30.9
B19	16.6	21.9	B54	4.6	7.2
B21	15.0	10.7	B55	6.6	23.3
B23	6.4	11.8	B56	17.4	27.2
B24	3.8	20.1	B59	18.0	28.3
B25	12.2	23.3	B6	12.7	24.3
B26	19.9	35.6	B60	17.5	46.5
B28a	11.1	11.4	B60b	16.6	27.3
B28b	5.8	9.6	B61	7.4	15.0
B29a	8.8	32.2	B62	10.6	8.9
B29b	9.7	28.9	B62d	14.0	15.1
B30	17.7	35.9	B62f	12.8	19.5
B31	5.9	10.2	B62h	11.9	27.9
B32	5.6	7.9	B63a	17.5	38.0
B35	18.0	24.8	B63d	12.3	24.9
B37	10.1	21.6	B64	11.5	26.8
B39a	27.0	47.0	B66a	8.7	15.7
B39b	17.0	45.8	B67	34.2	30.3
B40	12.6	29.4	B68a	31.1	31.4
B41	24.8	44.2	B68b	34.6	37.2
B43	7.9	19.7	B68c	36.8	23.3
B44	23.0	58.1	B68d	30.9	18.2
B46	13.7	15.8	B68e	54.2	22.8
B47	18.5	36.8	B68f	53.4	19.9
B49	18.3	62.4	B69	10.9	22.6

Table 21. Adkins Branch Basin Final Subbasin Transform Parameters

Subbasin	Lag Time (min)
s1	123
s10	88
s12	165
s13	95
s14	42
s16	105
s2	116
s3	188
s4	121
s45	66
s53	14
s54	133
s8	168
s9	93

Table 22. Big Ditch Basin Final Subbasin Transform Parameters

Subbasin	Lag Time (min)
s1	62
s10	93
s11	88
s2	88
s3	71
s4	80
s49	42
s5	56
s50	46
s7	64
s8	58
s9	50

5.1.2.4 Baseflow

For the Neuse River mainstem HEC-HMS model, the recession method was used to account for baseflow during historic and design storm events. Initial discharge was based on per area values. Subbasin recession constant and a ratio to peak threshold type, ratio was used. These values were based on knowledge of typical values for these parameters for relatively small urban and rural watersheds in the study area as well as adjacent major river basins (Tar River and Cape Fear River). The initial baseflow parameters were adjusted during model calibration to best fit observed data at select sites throughout the basin. Upon calibration and validation, the final parameter values listed in Table 23 were used in existing conditions and future without project conditions models.

For the Crabtree Creek, Hominy Swamp Creek, Big Ditch, and Adkins Branch HEC-HMS models, baseflow was not included due to the absence of calibration sources and their relatively small watershed area.

Table 23. Final Baseflow Parameters for Neuse River Mainstem HEC-HMS Model

Subbasin	Initial Discharge (cfs/sq mi)	Recession Constant	Ratio to Peak	Subbasin	Initial Discharge (cfs/sq mi)	Recession Constant	Ratio to Peak
B10	0.90	0.90	0.01	B5	0.50	0.50	0.01
B11	0.50	0.50	0.04	B50	0.50	0.90	0.08
B15	0.10	0.50	0.01	B52	3.00	0.50	0.01
B16	0.50	0.50	0.01	B53	1.00	0.80	0.01
B19	0.50	0.50	0.01	B54	3.00	0.50	0.01
B21	1.00	0.50	0.01	B55	0.90	0.50	0.01
B23	0.50	0.50	0.01	B56	1.00	0.80	0.01
B24	0.50	0.70	0.20	B59	1.00	0.95	0.01
B25	0.50	0.50	0.08	B6	0.50	0.50	0.01
B26	1.00	0.80	0.01	B60	1.00	0.95	0.01
B28a	0.90	0.50	0.01	B60b	1.00	0.95	0.01
B28b	0.90	0.50	0.01	B61	1.00	0.95	0.01
B29a	0.50	0.50	0.01	B62	10.00	0.70	0.01
B29b	0.50	0.50	0.01	B62d	10.00	0.70	0.01
B30	1.00	0.80	0.01	B62f	10.00	0.70	0.01
B31	0.50	0.50	0.01	B62h	10.00	0.70	0.01
B32	0.90	0.80	0.10	B63a	3.00	0.50	0.01
B35	1.00	0.70	0.01	B63d	3.00	0.50	0.01
B37	0.50	0.50	0.01	B64	3.00	0.50	0.01
B39a	0.50	0.50	0.01	B66a	3.00	0.50	0.01
B39b	0.50	0.50	0.01	B67	0.50	0.50	0.01
B40	3.00	0.50	0.01	B68a	0.50	0.50	0.01
B41	1.00	0.95	0.01	B68b	0.50	0.50	0.01
B43	0.50	0.50	0.01	B68c	0.50	0.50	0.01
B44	0.50	0.90	0.08	B68d	0.50	0.50	0.01
B46	0.90	0.80	0.01	B68e	1.50	0.80	0.10
B47	1.00	0.95	0.01	B68f	1.50	0.80	0.10
B49	0.50	0.90	0.08	B69	0.50	0.50	0.01

5.1.2.5 Reach Routing

Modified-Puls reach routing was used in both the Neuse River mainstem basin and the Crabtree Creek basin HEC-HMS models. For Crabtree Creek, it was used exclusively for all reaches in the basin. Discharge-storage curves were developed from a detailed cross section and structure survey related to the Neuse River basin study Crabtree Creek Hydrologic Engineering Center's River Analysis System (HEC-RAS) model. Natural floodplain cross sections were surveyed at an approximate 1000-ft interval. Regression-based discharge equations were used in the HEC-RAS to establish rating curves of storage volume versus discharge. Sub-reaches were estimated using the following equation:

$$\# subreaches = \frac{L}{v\Delta t}$$

The velocity used for this relationship was determined by solving Manning's equation for normal depth given the 100-year flood discharge, as determined from USGS regional regression equations (AECOM, 2011). Initial condition for each routing reach were set to discharge = inflow.

For the Neuse River mainstem basin HEC-HMS model, Modified-Puls routing methods were used for a limited number of reaches. Five routing reaches near the outlet point of the model used this method due to the sensitivity in storage volume and downstream floodplain conditions. The same methods describe above were used to estimate initial routing reach values. The Neuse River mainstem basin model also used the Muskingum method at four routing reaches in the middle portion of the basin, between Goldsboro and Kinston. Initial Muskingum K values were based on time of concentration estimates using equation (1). Muskingum X values were set low to represent a large degree of hydrograph attenuation. The majority of routing reaches in the Neuse River mainstem basin model used the Muskingum-Cunge method. Reach length and slope dimensions were calculated within HEC-HMS 4.8 and channel characteristics were initially based on the USACE CWMS HEC-RAS model.

For the Hominy Swamp Creek, Big Ditch, and Adkins Branch basin HEC-HMS models, routing methods were based on Muskingum-Cunge. Reach length and slope dimensions were calculated within HEC-HMS 4.8 and channel characteristics were initially based on FEMA effective FIS HEC-RAS modeling.

For all model, initial values were adjusted during calibration to best fit observed data. Only small adjustments were made to the Modified-Puls sub-reach count and Muskingum-Cunge roughness values during calibration.

5.1.2.6 Reservoirs

For the Neuse River mainstem basin HEC-HMS model, a simplified modeling approach was taken to represent observed reservoir releases from Falls Dam during calibration events and assumed operations during design storms. Discharge from the dam was reduced to a minimum flow threshold, or about 100 cfs, during main precipitation events and held constant while conditions were monitored at flow target locations downstream. This mandated operation schema would result in a negligible flow increase (+100 cfs) to the peak discharge associated with downstream basin uncontrolled flow. Per the Falls Lake Water Control Manual, operations allow accommodation for flooding that occur in areas downstream of Clayton, NC and coupled with runoff from uncontrolled drainage areas. Releases from Falls Dam can be reduced to near minimum prior to a storm event to prevent discharges from contributing substantially to those uncontrolled floodwaters. Afterwards and when possible, the flood control space in the reservoir will be evacuated at a rate that will produce up to non-damaging stages downstream. A series of flood releases would be made from Falls Lake once the uncontrolled peak has occurred, and downstream hydrographs have begun receding.

Based on review of Falls Lake operations during historical events, there were considerable delays (~2 weeks) in flood releases following the main precipitation events. It was generally assumed that for this basin study river stages predominantly regulated by Falls Dam releases following design event precipitation will not be associated with water surface elevations that cause peak economic damages. Within the model, Falls Lake releases were simulated as a source element with a constant discharge of 100 cfs. Without the need to simulate a complicated release schedule, the Falls Lake reservoir was represented by a model sink element. USACE water management provided a Falls Lake daily accounts database of reservoir elevation, inflow, outflow, and storage that covered the federal project's history. This dataset was used to determine the ability for the reservoir to successfully capture the full range of inflow, from the 770 square mile portion of the basin above the project, generated for the suite of design storms.

The Crabtree Creek basin HEC-HMS model contained multiple reservoirs. Reservoirs that were included in FEMA detail study streams were assumed to have potential to provide storage during large events and were included in the HEC-HMS model. Basin reservoirs characteristics were determined from survey data (outlet works and spillway dimensions) and GIS-based analysis (elevation-storage area curves). Reservoirs that were included in the Crabtree Creek basin HEC-HMS model are listed in Table 24.

Reservoir elements were not applicable within the smaller basin models, Hominy Swamp Creek, Big Ditch, or Adkins Branch. In these subbasins, either no impoundment structure existed or was at a small enough scale to be considered negligible in causing significant flood effects .

Table 24. Crabtree Creek basin HEC-HMS Modeled Reservoirs (AECOM)

Table 2-4: Routing Reservoirs Used in the HEC-HMS Models

Structure	Sub-shed (Stream)
I-440 Culvert	Big Branch (Big Branch)
Cedar Hills Lake Dam	Big Branch (Big Branch)
Lassiter Mill Dam	Crabtree Creek (Crabtree Creek)
Lake Crabtree Dam	Crabtree Creek (Crabtree Creek)
Lake Lynn Dam	Hare Snipe Creek
Vet School Pond	House Creek
I-440 Culvert	House Creek
US-70 Culvert	Little Brier Creek (Basin 18, Stream 16)
I-540 Culvert	Little Brier Creek (Little Brier Creek)
TW Alexander Dr Culver	Little Brier Creek (Little Brier Creek)
Railroad Culvert	Marsh Creek (Marsh Creek (B18, S17))
Millbrook Tributary Dam 1	Marsh Creek (Millbrook Tributary to Marsh Creek)
Millbrook Tributary Dam 2	Marsh Creek (Millbrook Tributary to Marsh Creek)
Millbrook Tributary Dam 3	Marsh Creek (Millbrook Tributary to Marsh Creek)
Beaman Lake Dam	Marsh Creek (New Hope Tributary to Marsh Creek)
New Hope Church Road Dam	Marsh Creek (New Hope Tributary to Marsh Creek)
New Hope Tributary Dam 1	Marsh Creek (New Hope Tributary to Marsh Creek)
New Hope Tributary Dam 2	Marsh Creek (New Hope Tributary to Marsh Creek)
Long Street Culvert	Mine Creek (East Fork Mine Creek)
Newton Road Culvert	Mine Creek (East Fork Mine Creek)
Woodbend Drive Dam	Mine Creek (East Fork Mine Tributary)
Lead Mine Road Culvert	Mine Creek (Lynn Road Tributary)
Shelley Lake Dam	Mine Creek (Mine Creek)
Beaverdam Creek (Basin 12, Stream 1) Dam 1	Neuse Tribs (Beaverdam Creek (B12, S1))
Beaverdam Creek (Basin 12, Stream 1) Dam 2	Neuse Tribs (Beaverdam Creek (B12, S1))
Beaverdam Creek (Basin 15, Stream 21) Dam 1	Neuse Tribs (Beaverdam Creek (B15, S21))
Hodges Creek Dam 1	Neuse Tribs (Powell Creek)
Hodges Creek Dam2	Neuse Tribs (Powell Creek)
Gresham Lake/US 1	Perry Creek (Perry Creek (B15, S26))
I-540 Culvert	Perry Creek (Perry Creek (B15, S26))
Hunting Ridge Road	Perry Creek (Perry Creek (B15, S26))
North Ridge CC Lake Dam	Perry Creek (Perry Creek (B15, S26))
Reedy Creek Road	Richland Creek (Richland Creek (B18, S3))
Big Lake	Sycamore Creek (Sycamore Creek)
Sycamore Lake	Sycamore Creek (Sycamore Creek)
Grove Barton Road Culvert	Turkey Creek (Basin 18, Stream 4)
Lake Anne	Turkey Creek (Turkey Creek)
Lake Dunaway	Turkey Creek (Turkey Creek)
Lake 3	Turkey Creek (Turkey Creek)
Yates Mill Pond Dam	Yates Branch

5.1.3 Calibration And Validation

Four rainfall events were chosen for the Neuse River Mainstem basin HEC-HMS model calibration and validation. Three events were used for calibration and one for validation. The three calibration scenarios included historic Hurricane Matthew (2016) and Hurricane Florence (2018), and a September 2019 widespread rainfall event. 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. An April 2017 rainfall event was chosen for validation. While there have been older historic rainfall events that have impacted the basin, due to difficultly in consistent calibration data and flow records affected by construction of Falls Lake in the early 1980s, it was determined more appropriate to focus on recent flooding events that also better reflect the model's assumption of existing conditions. Summary of events used for calibration and validation is listed in Table 25.

Table 25. Calibration and Validation Rainfall Events for Neuse River Mainstem Basin HEC-HMS Model

Event	Precipitation Source	Average Rainfall Depth (in)			Event Classification
		Upper Neuse	Middle Neuse	Lower Neuse	
October 7-10, 2016	NOAA XMRG	7.9	9.8	6.4	Calibration
September 13-15, 2018	NOAA XMRG	6.5	11.4	13.5	Calibration
September 5-7, 2019	NOAA XMRG	4.2	6.1	6.3	Calibration
April 24-26, 2017	NOAA XMRG	7.1	4.9	3.2	Validation

Next Generation Weather Radar (NEXRAD) Stage IV hourly gridded precipitation data from NWS was obtained from USACE SAW water management. All calibration events occurred during the Fall season, which is historically when most significant tropical systems have impacted the Neuse River basin. The validation event occurred in the Spring season and is typical of frontal weather systems that cause major thunderstorms and associated heavy rainfall. Locations of streamflow gage sites used in calibration efforts are shown in Figure 25 and listed in Table 26 below.

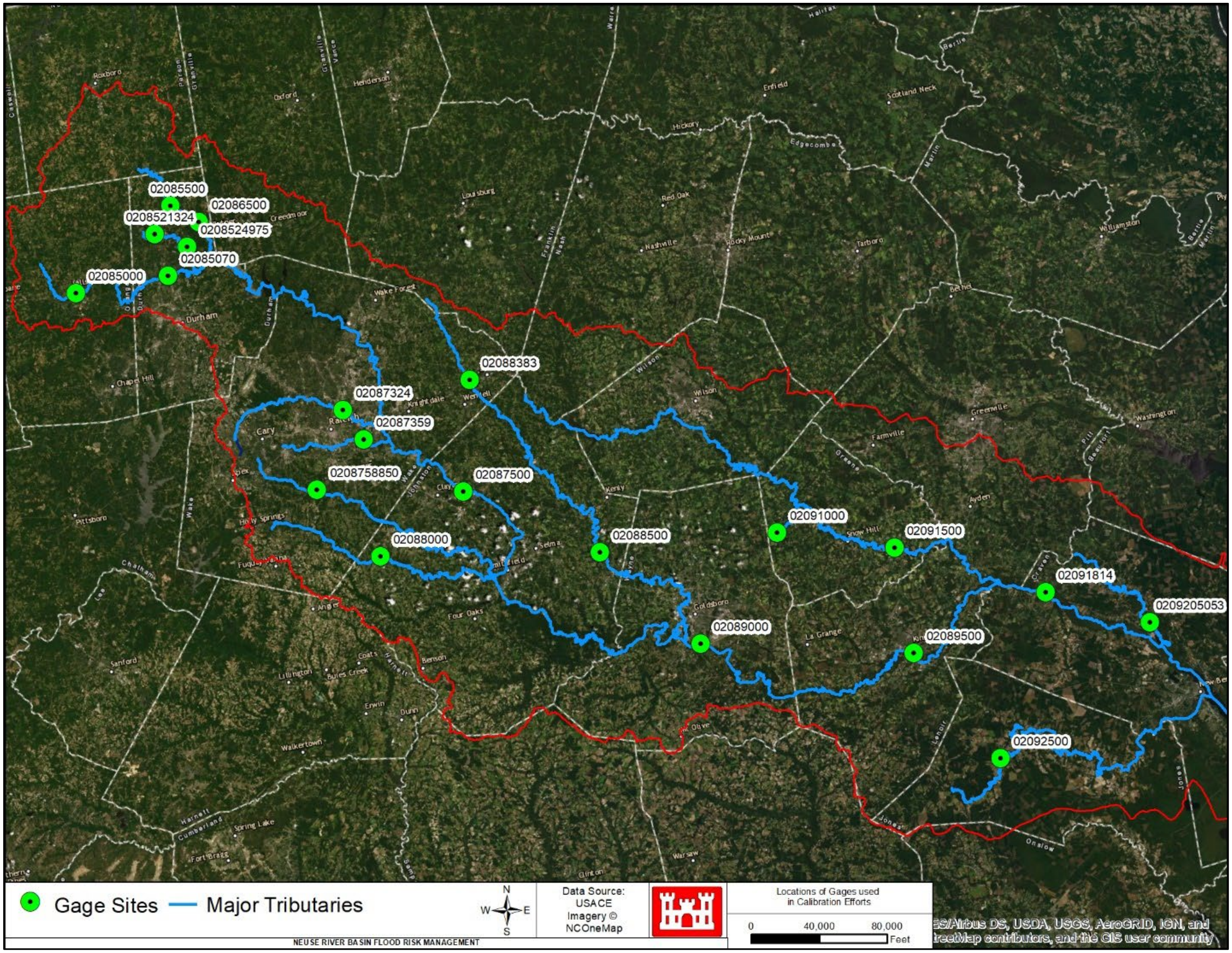

Figure 25. Streamflow gage locations for Calibration Efforts

Table 26. Streamflow Number Identifications for Calibration Efforts

Station Number	Station Name
02085000	Eno River at Hillsborough
02085070	Eno River at Durham
0208521324	Little River at SR 1461 nr Orange Factory
0208524975	Little River below Little River Trib at Fairntos
02085500	Flat River at Bahama
02086500	Flat River below Dam nr Bahama
02087324	Crabtree Creek at US1 at Raleigh
02087359	Walnut Creek at Sunnybrook Drive at Raleigh
02087500	Neuse River nr Clayton
0208758850	Swift Creek nr McCullars Crossroads
02088000	Middle Creek nr Clayton
02088500	Little River nr Princeton
02089000	Neuse River nr Goldsboro
02089500	Neuse River at Kinston
02091000	Nahunta Swamp nr Shine
02091500	Contentnea Creek at Hookerton
02091814	Neuse River nr Fort Barnwell
0209205053	Swift Creek at NC Hwy 43 nr Streets Ferry
02092500	Trent River nr Trenton
02088383	Little River nr Zebulon

Calibration to observed data was based on selection of widespread rainfall events as described above. Overall, comprehensive event coverage for the entire Neuse River basin was limited due to it large area. As listed in Table 25 above, even for Hurricanes Matthew and Florence, there were inconsistences in rainfall amounts across the different geographic regions in the basin. Outside of these major tropical events, the varying intensity associated with frontal-based rainfall events meant that out-of-bank flooding for large portions of the Neuse River mainstem was difficult to capture in a single, historical scenario. Results for the calibration and validation events at select USGS gages are shown in Figure 26 through Figure 86.

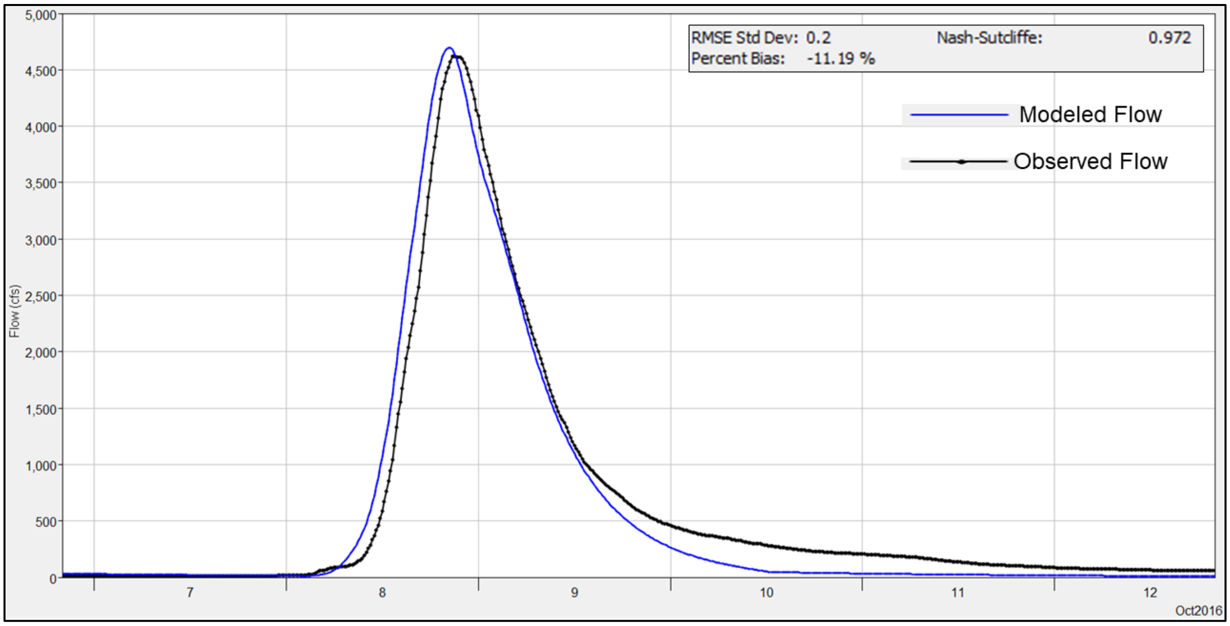

Figure 26. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Eno River at Hillsborough, NC Gage

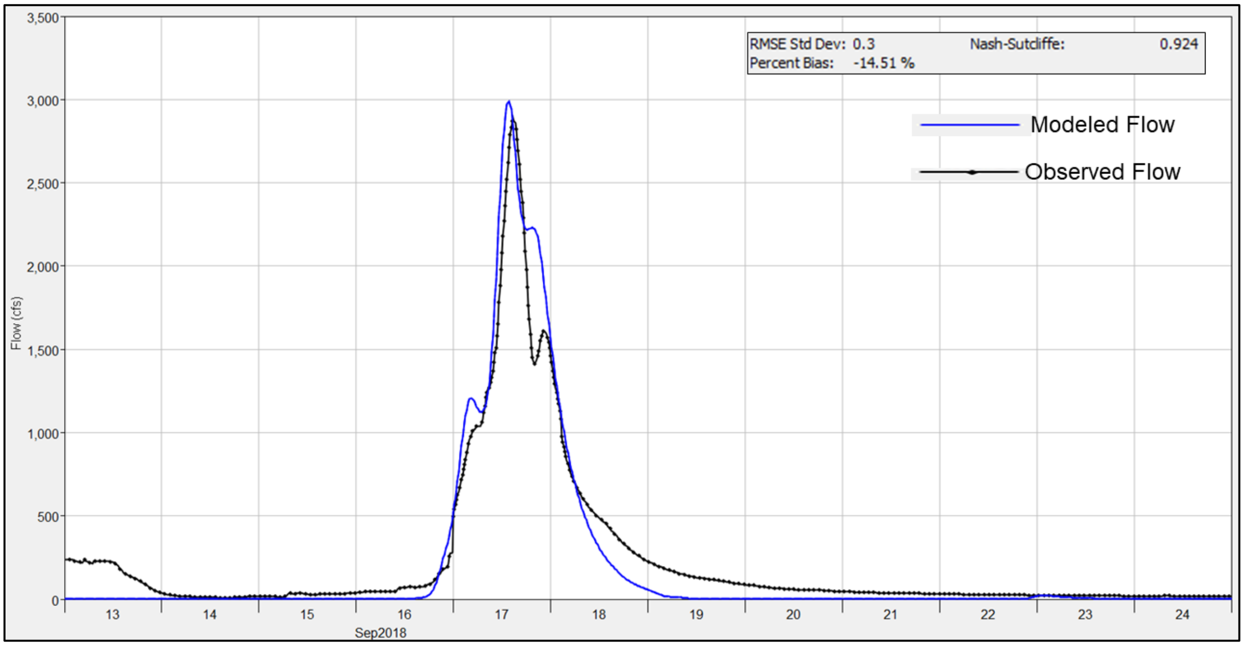

Figure 27. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Eno River at Hillsborough, NC Gage

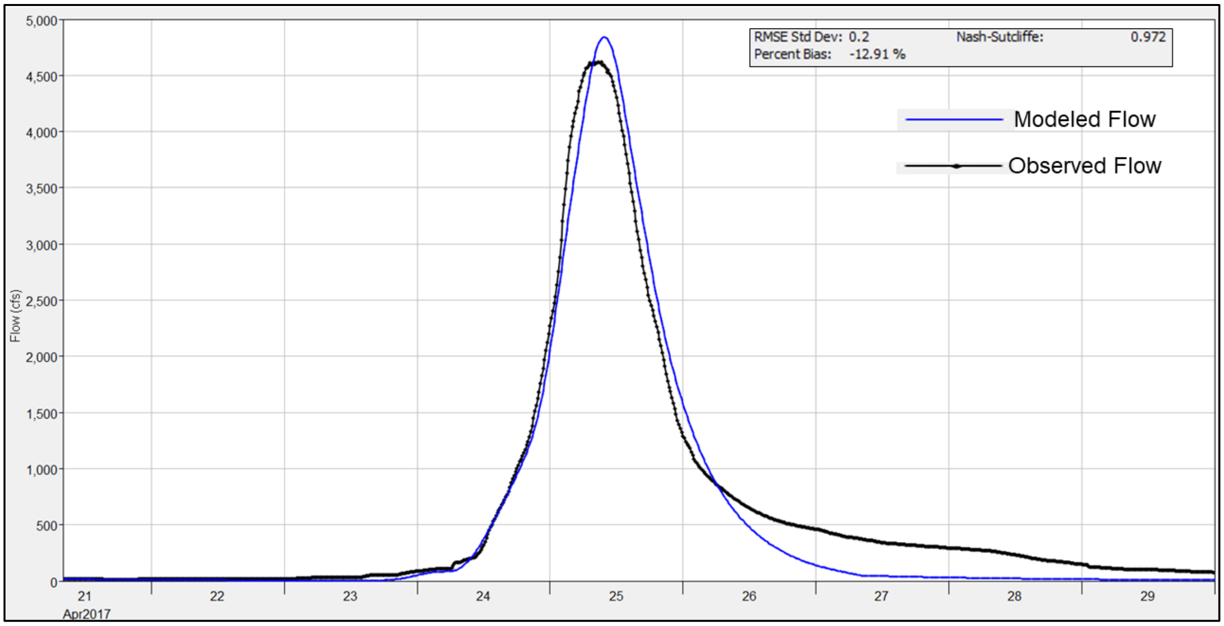

Figure 28. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Eno River at Hillsborough, NC Gage

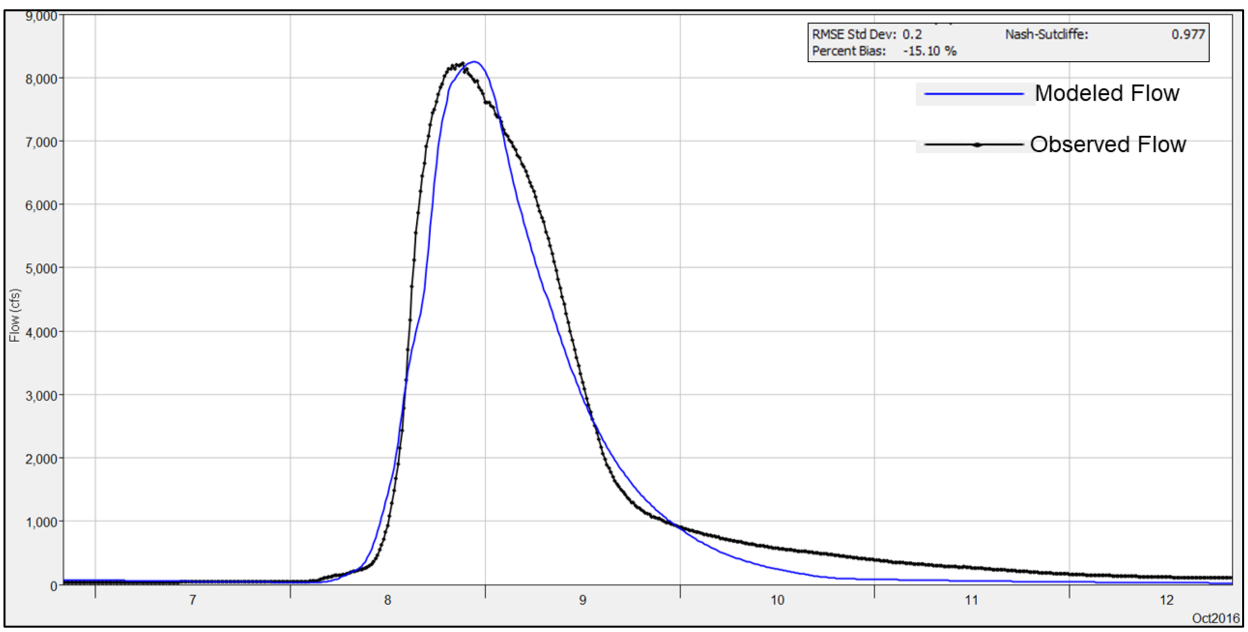

Figure 29. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Eno River near Durham, NC gage

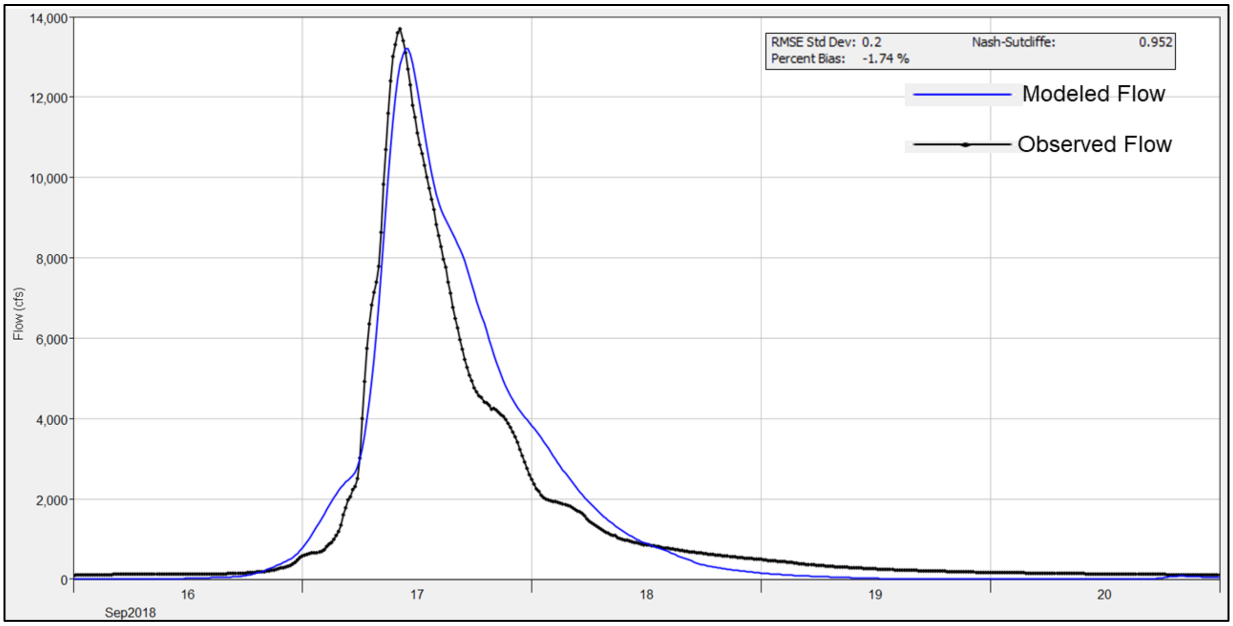

Figure 30. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Eno River near Durham, NC gage

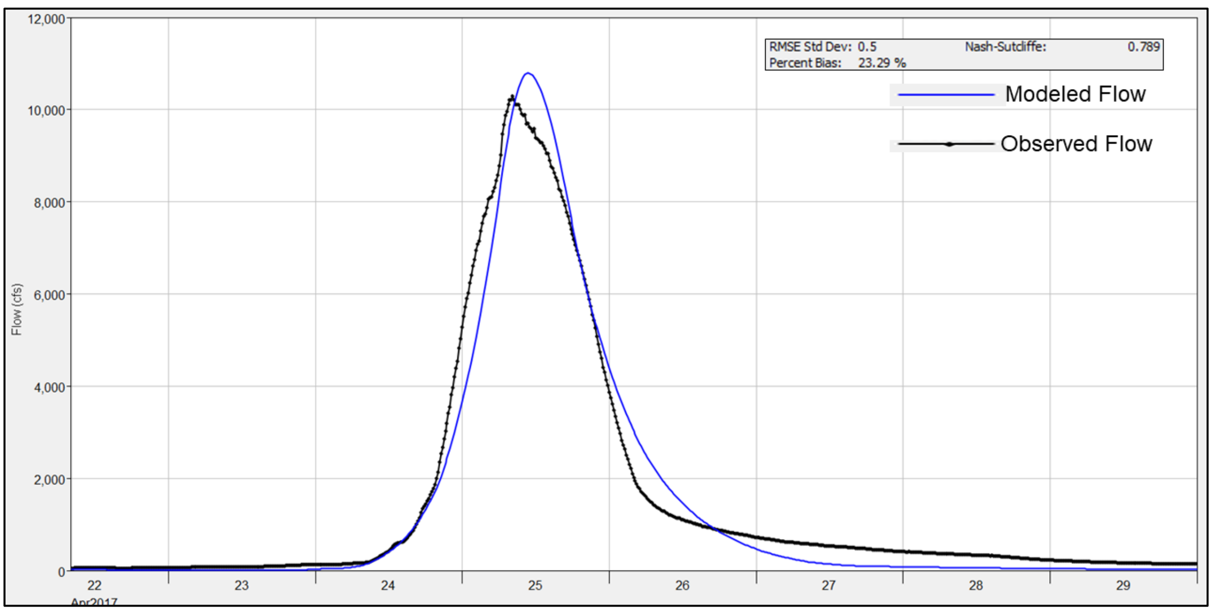

Figure 31. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Eno River near Durham, NC gage

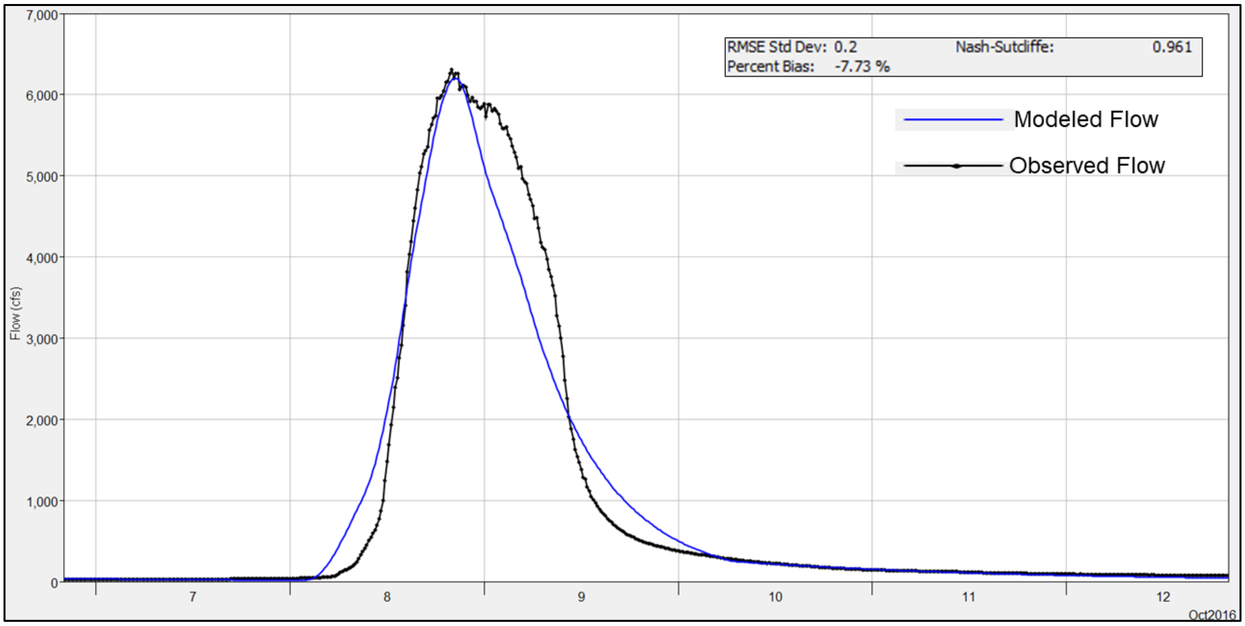

Figure 32. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Little River near Orange Factory, NC

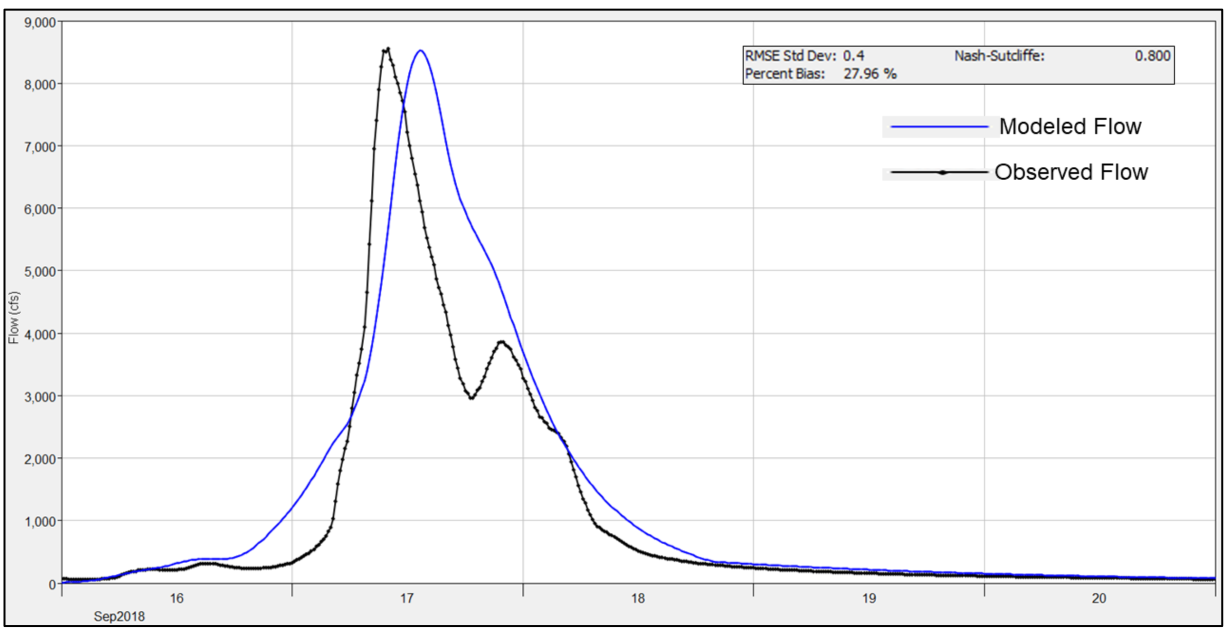

Figure 33. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Little River near Orange Factory, NC

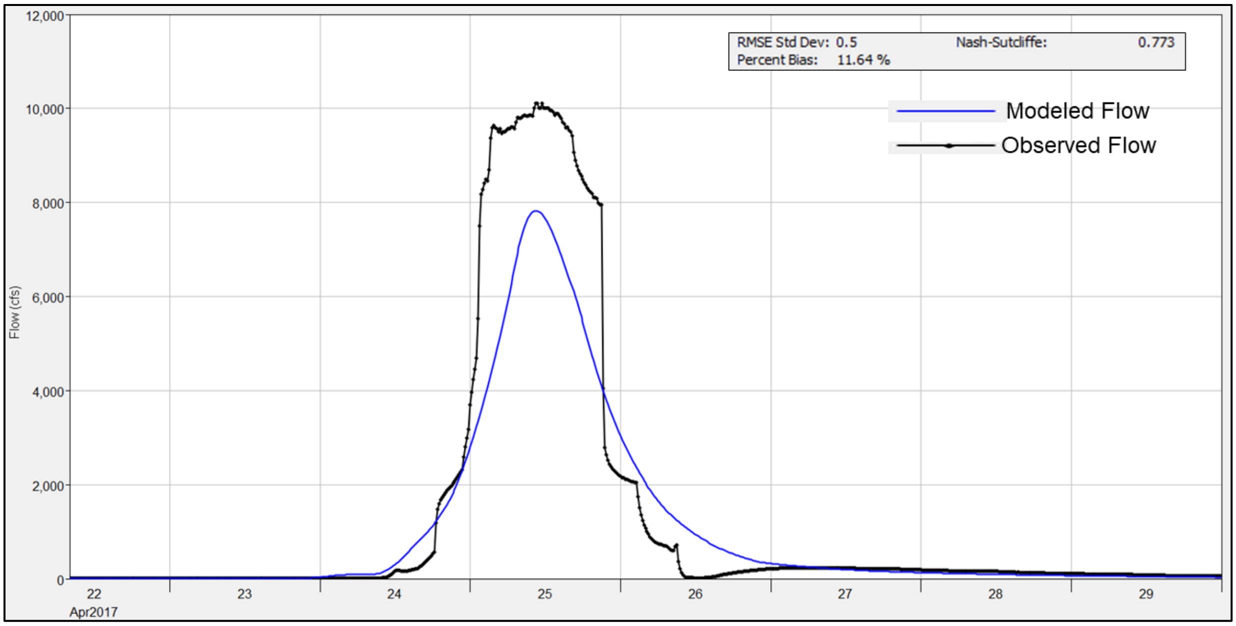

Figure 34. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Little River near Orange Factory, NC

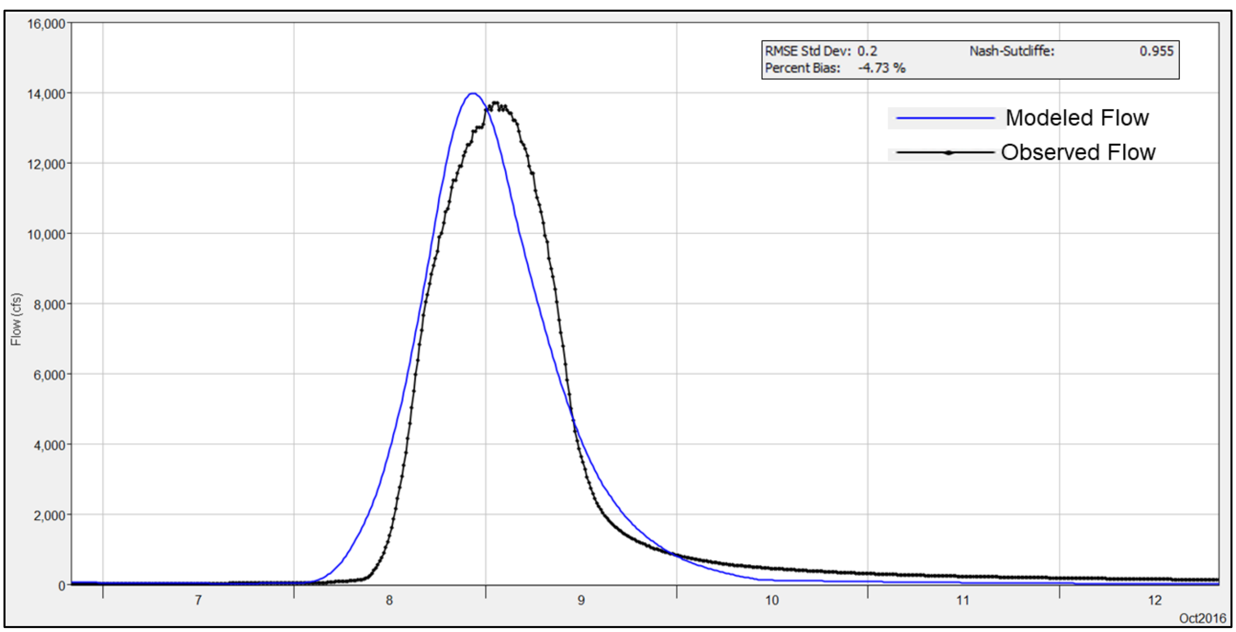

Figure 35. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Flat River at Bahama, NC Gage

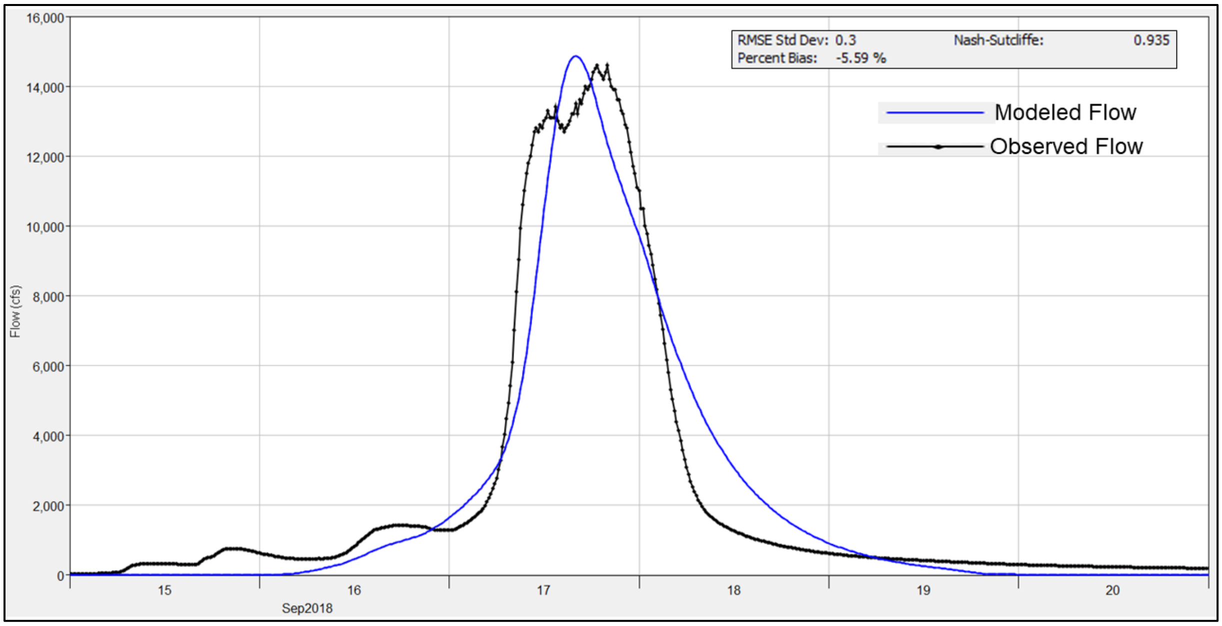

Figure 36. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Flat River at Bahama, NC Gage

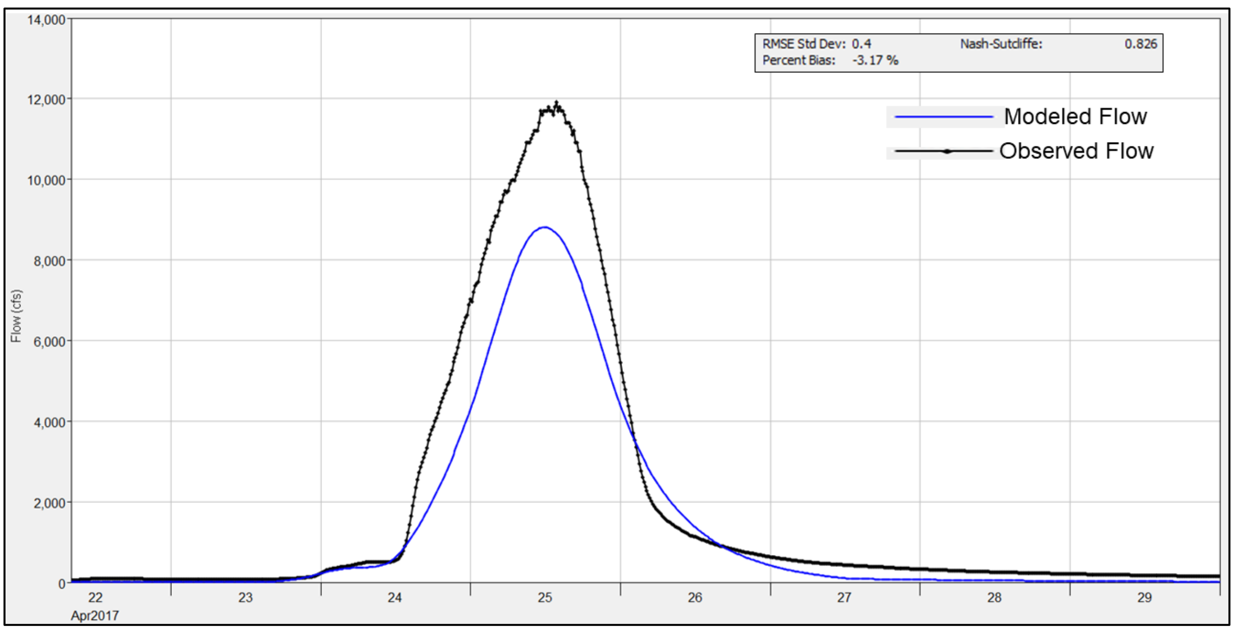

Figure 37. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Flat River at Bahama, NC Gage

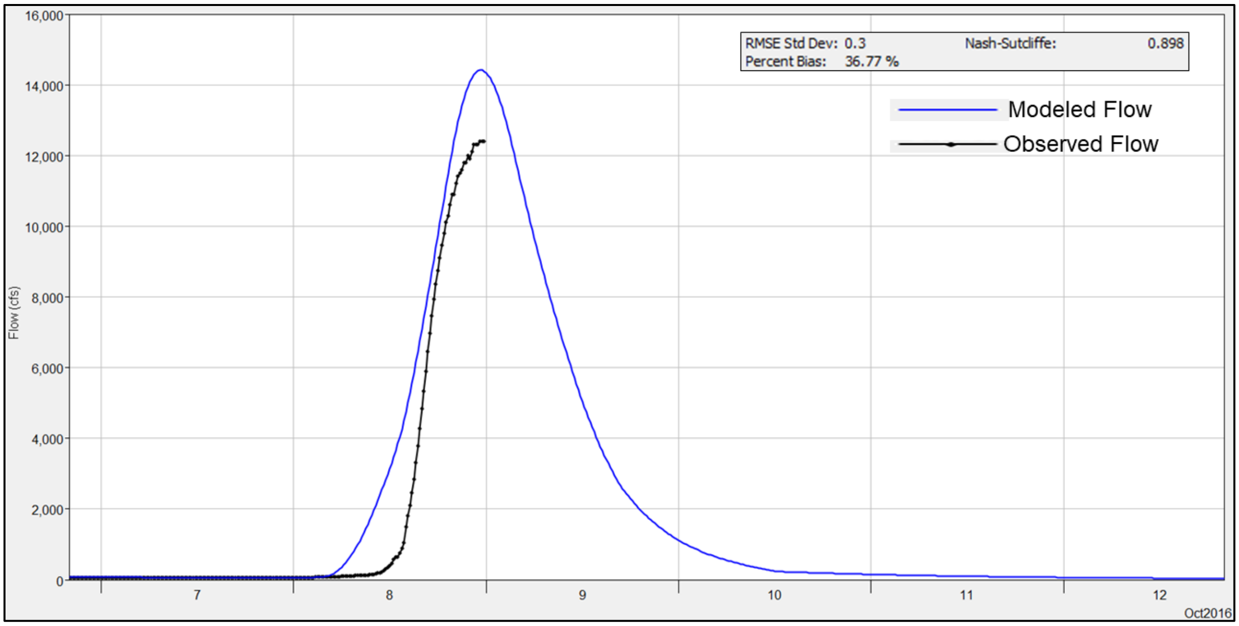

Figure 38. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Flat River at Dam nr Bahama, NC Gage

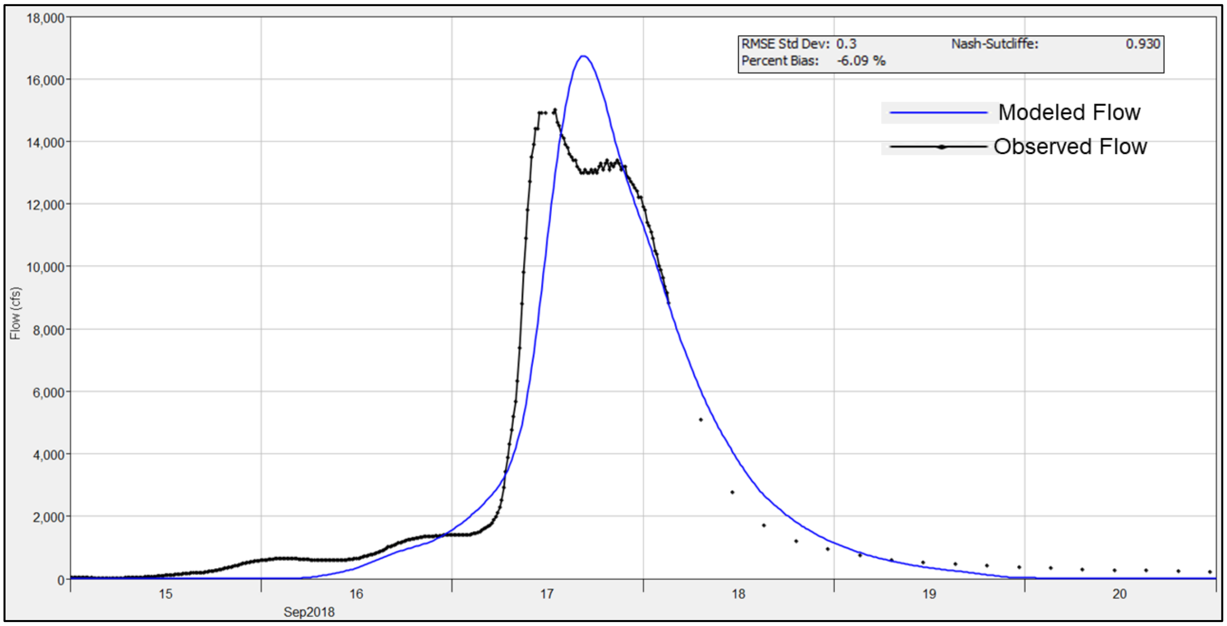

Figure 39. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Flat River at Dam nr Bahama, NC Gage

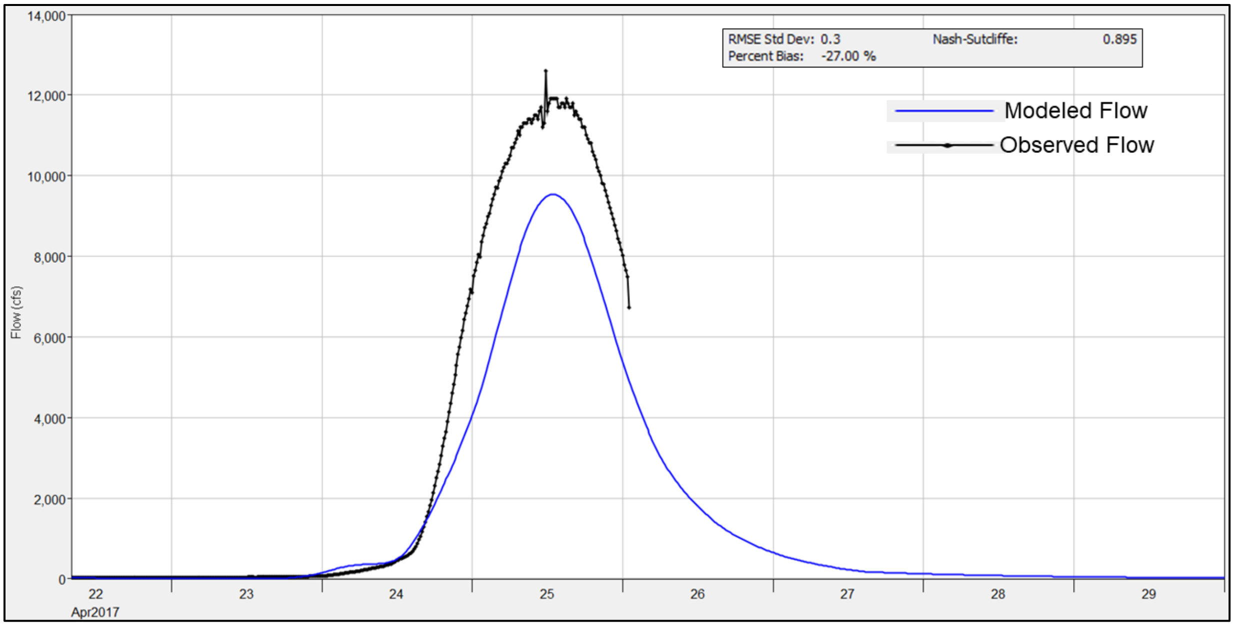

Figure 40. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Flat River at Dam nr Bahama, NC Gage

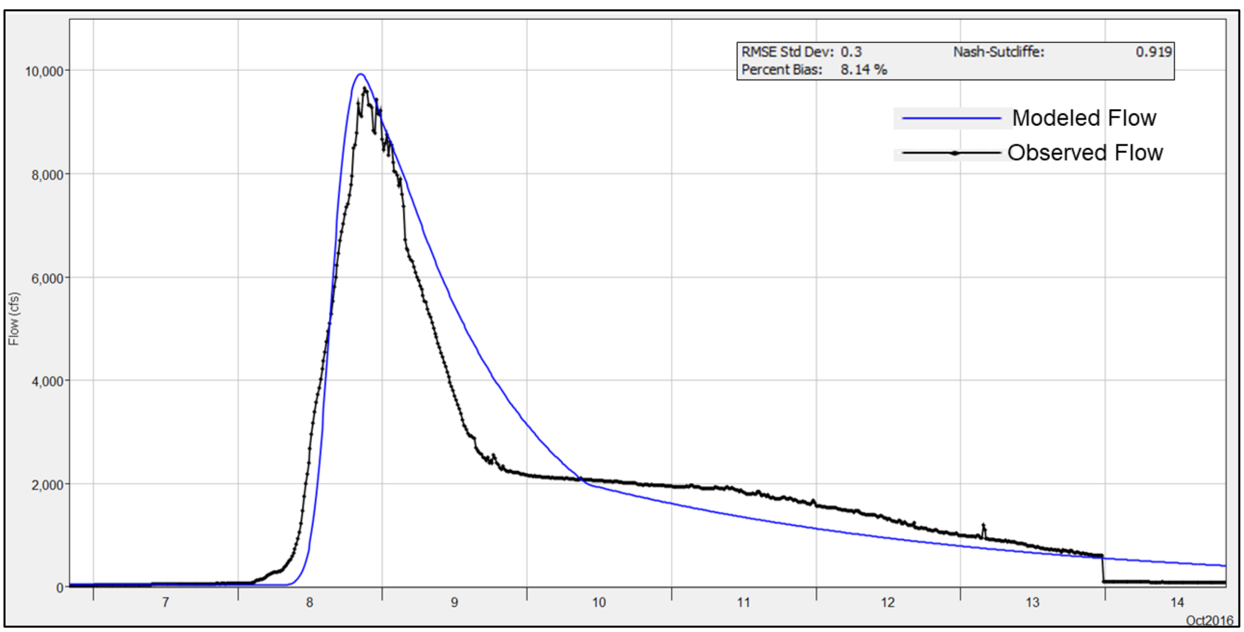

Figure 41. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Crabtree Creek at US-1 Gage

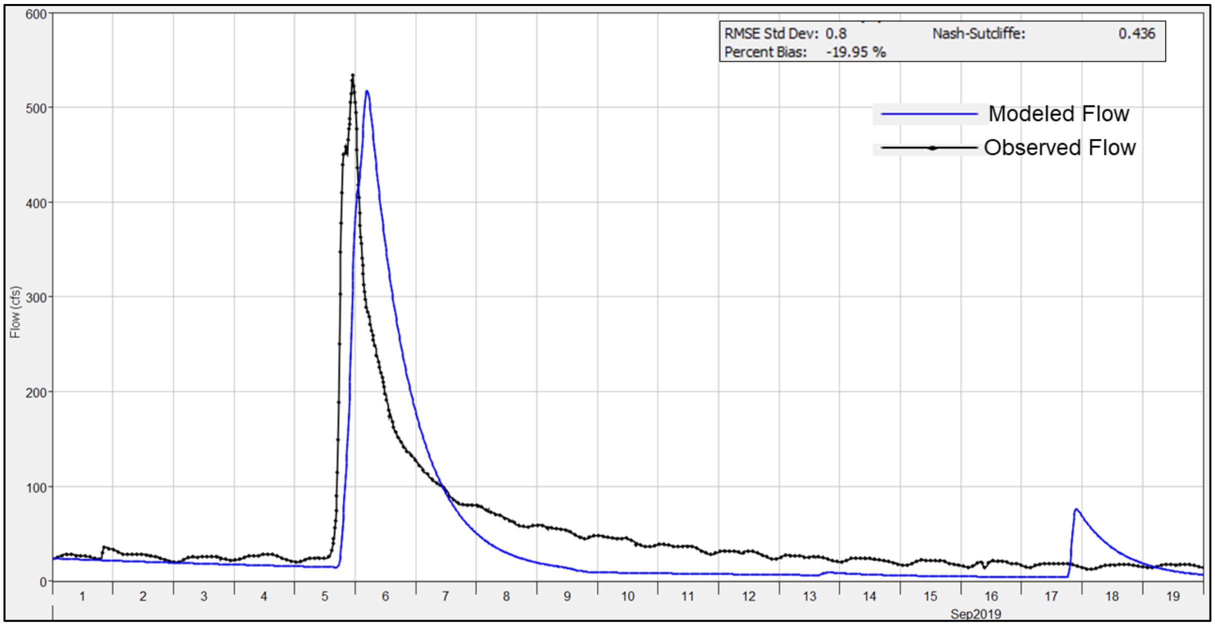

Figure 42. Neuse River Mainstem Basin HEC-HMS September 2019 Calibration at Crabtree Creek at US-1 Gage

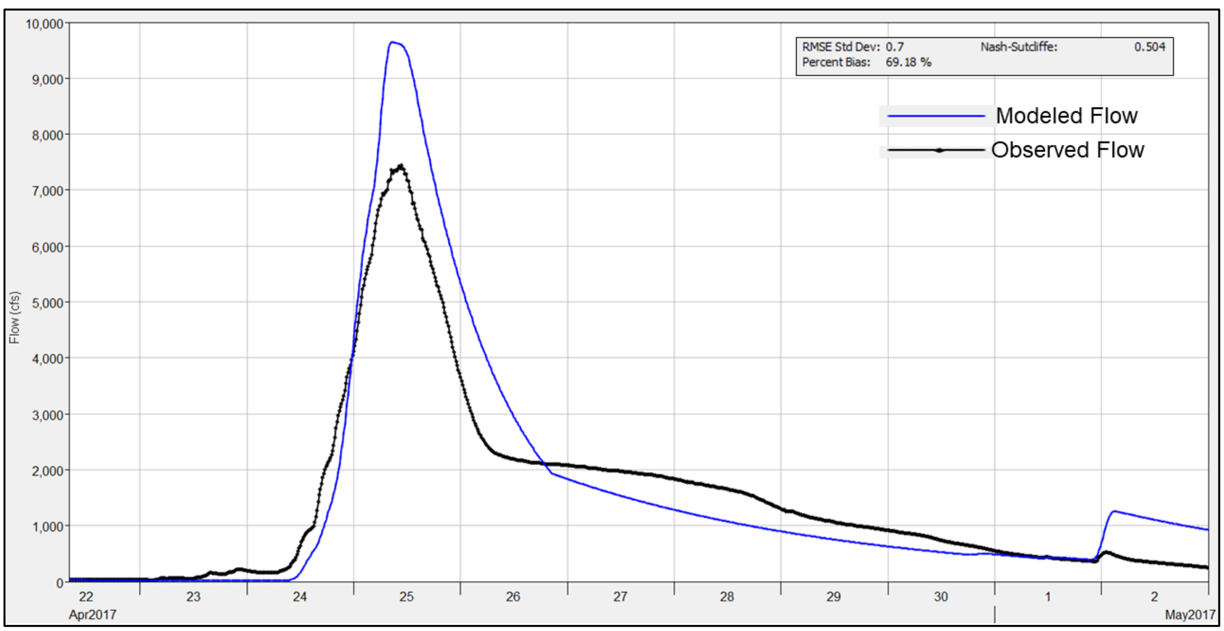

Figure 43. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Crabtree Creek at US-1 Gage

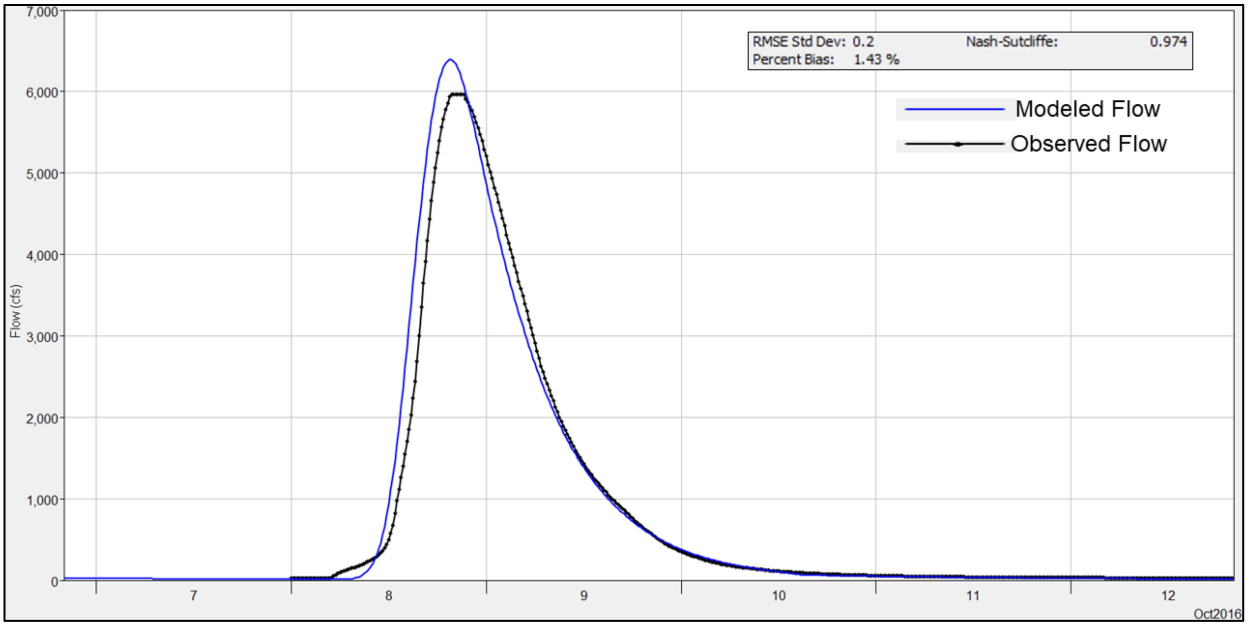

Figure 44. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Walnut Creek at Sunnybrook Dr Gage

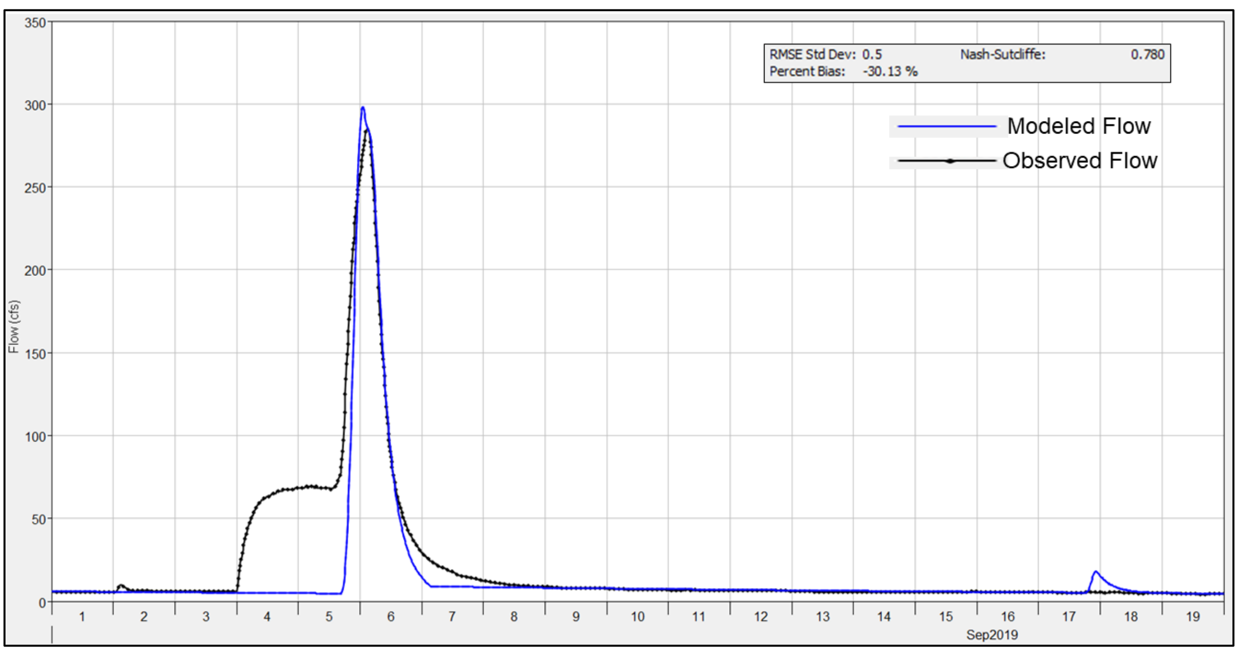

Figure 45. Neuse River Mainstem Basin HEC-HMS September 2019 Calibration at Walnut Creek at Sunnybrook Dr Gage

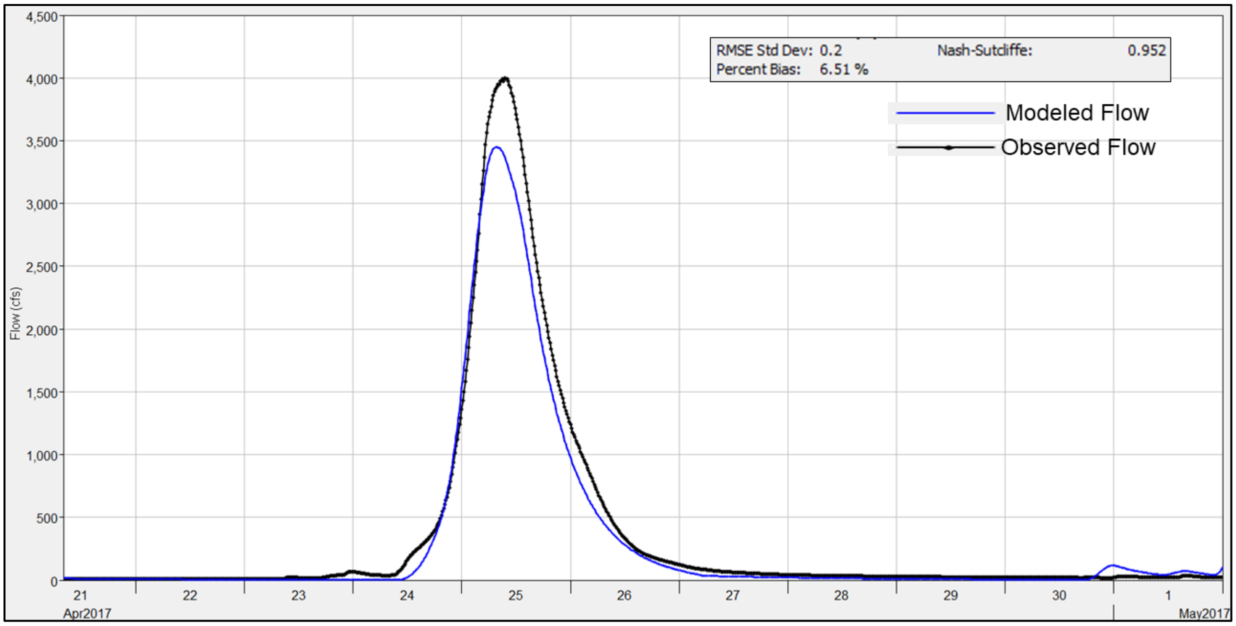

Figure 46. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Walnut Creek at Sunnybrook Dr Gage

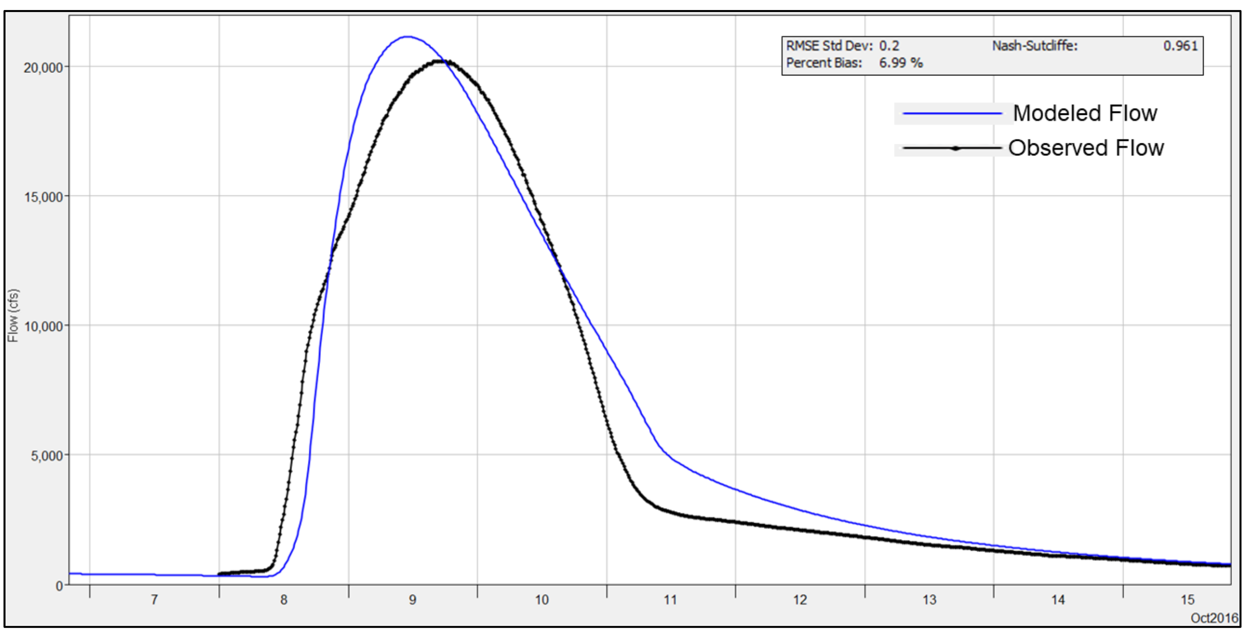

Figure 47. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Neuse River near Clayton, NC Gage

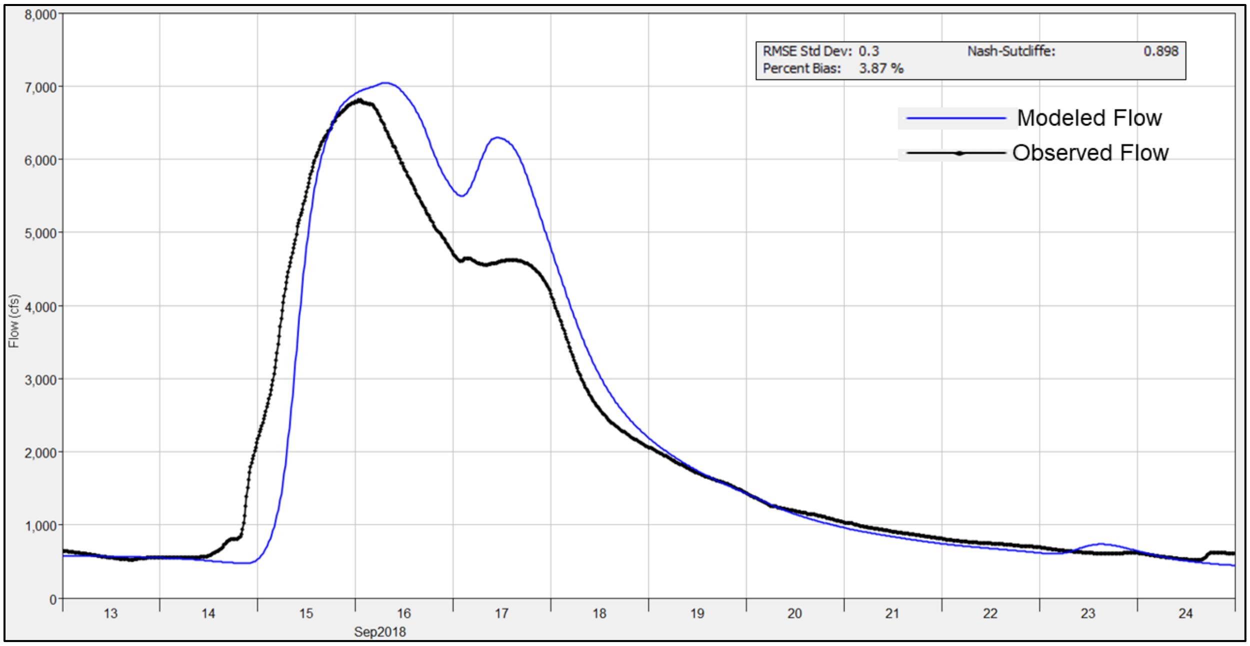

Figure 48. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Neuse River near Clayton, NC Gage

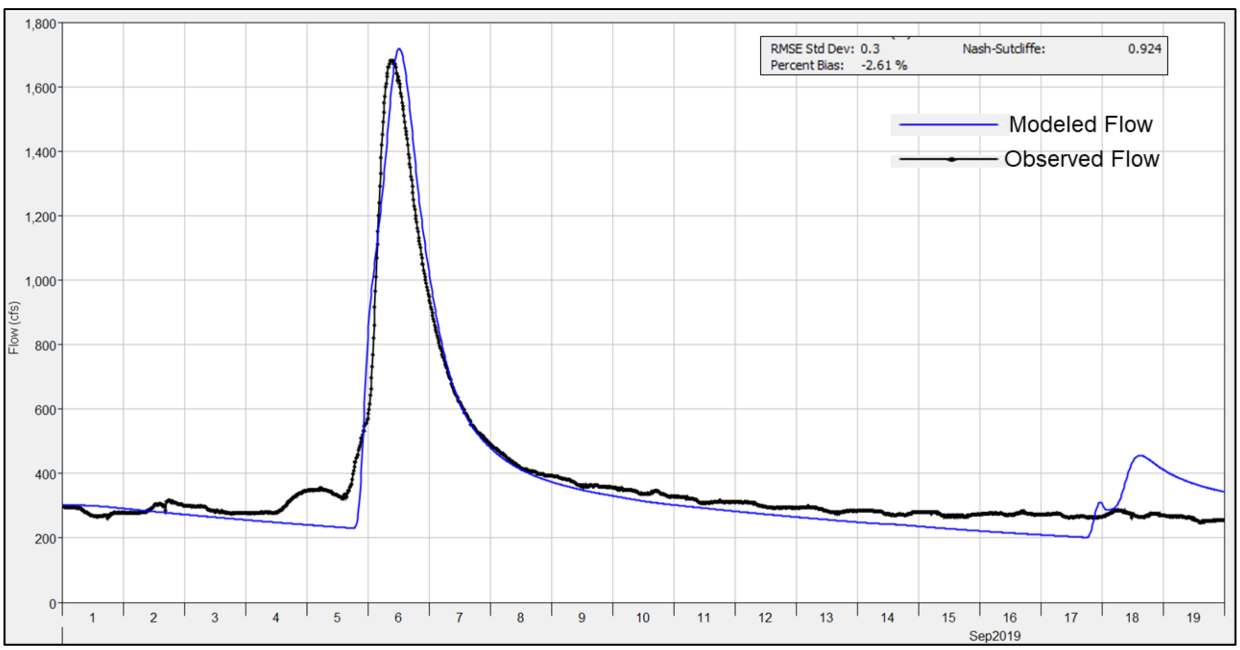

Figure 49. Neuse River Mainstem Basin HEC-HMS September 2019 Calibration at Neuse River near Clayton, NC Gage

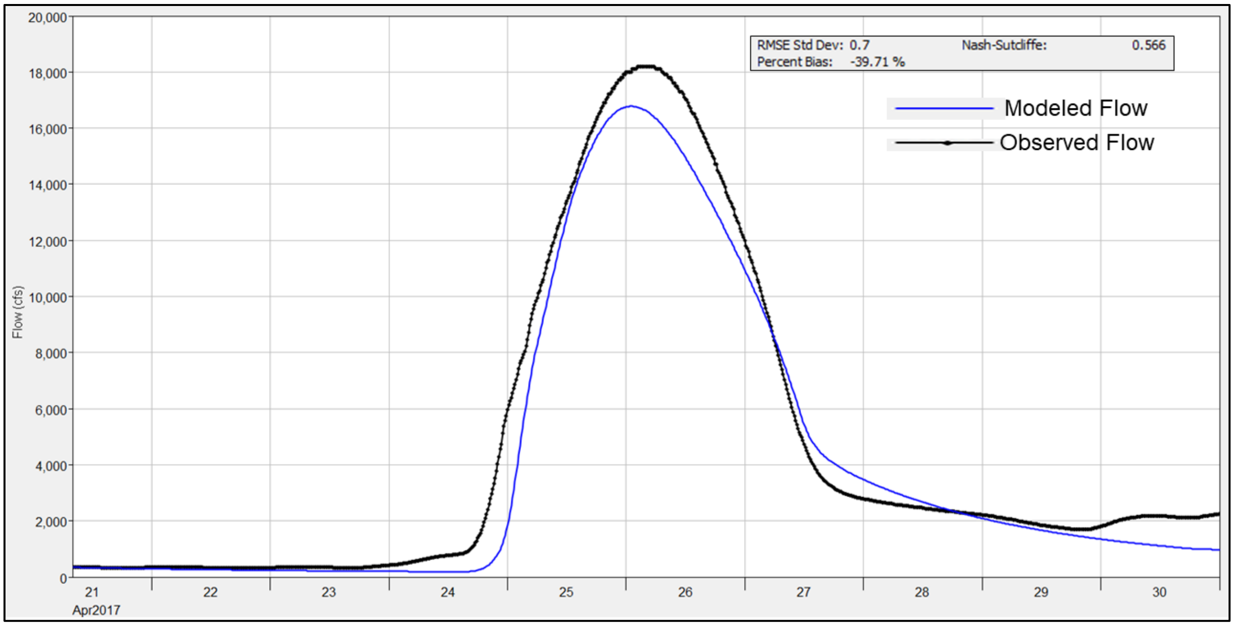

Figure 50. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Neuse River near Clayton, NC Gage

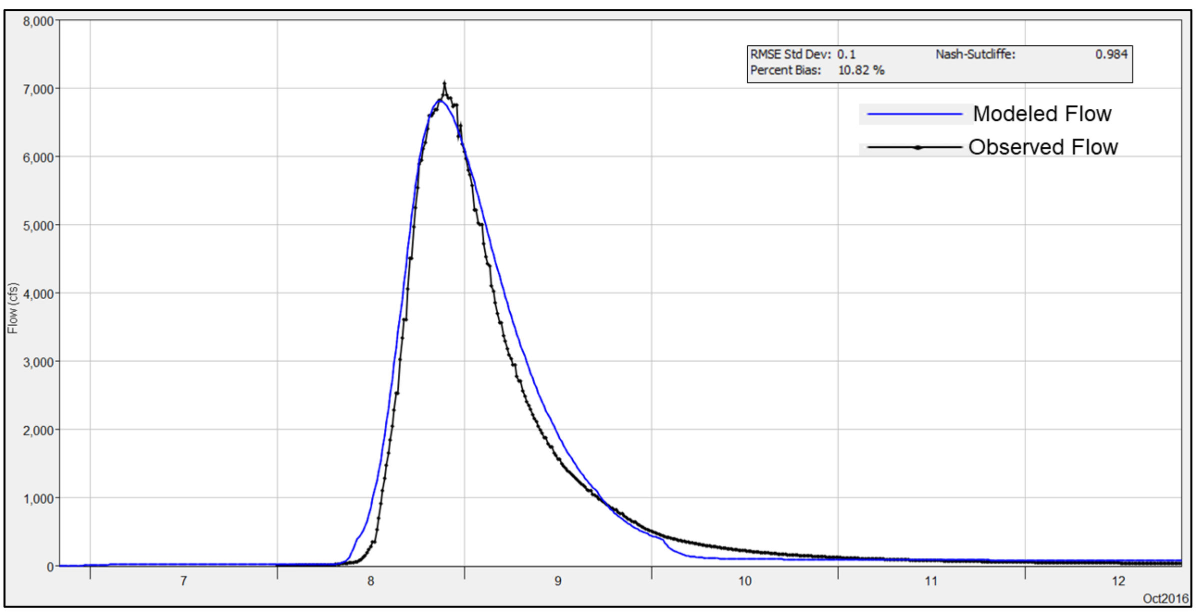

Figure 51. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Swift Creek near McCullars Crossroads, NC Gage

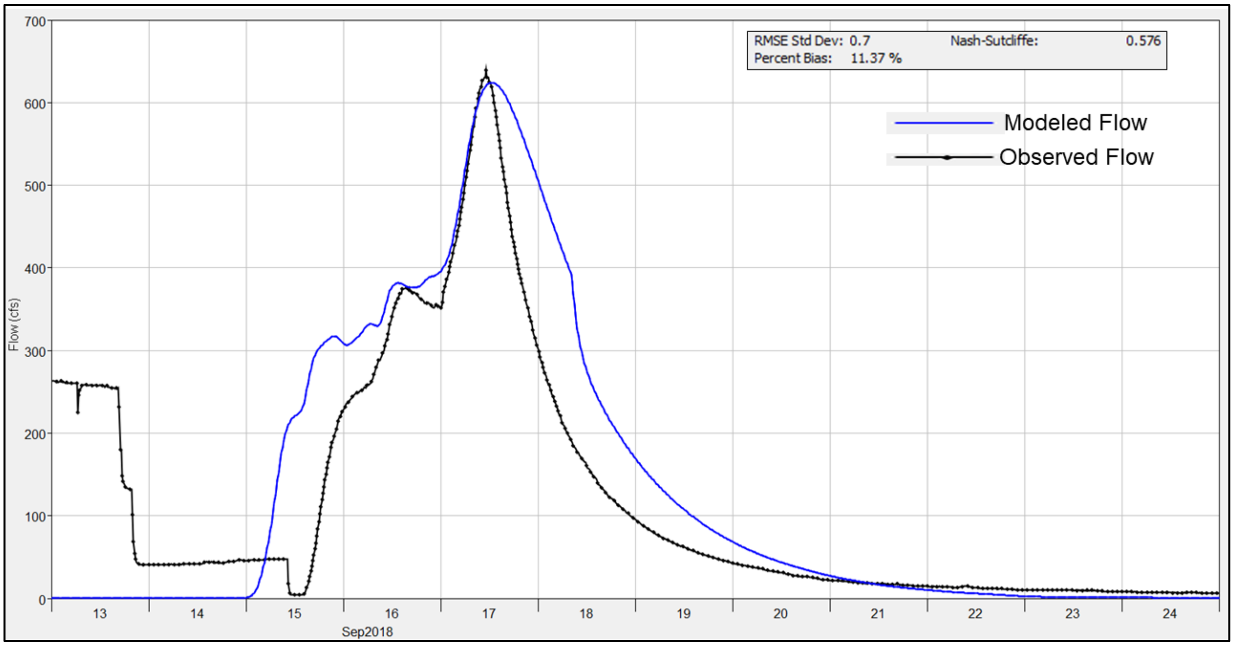

Figure 52. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Swift Creek near McCullars Crossroads, NC Gage

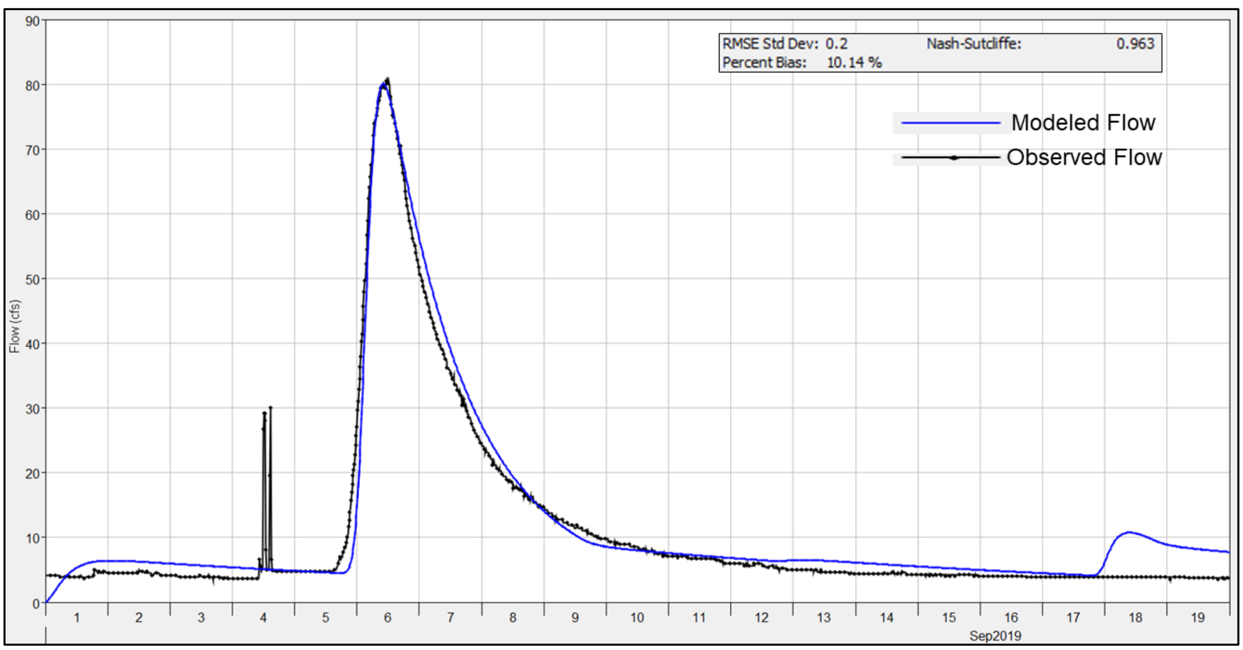

Figure 53. Neuse River Mainstem Basin HEC-HMS September 2019 Calibration at Swift Creek near McCullars Crossroads, NC Gage

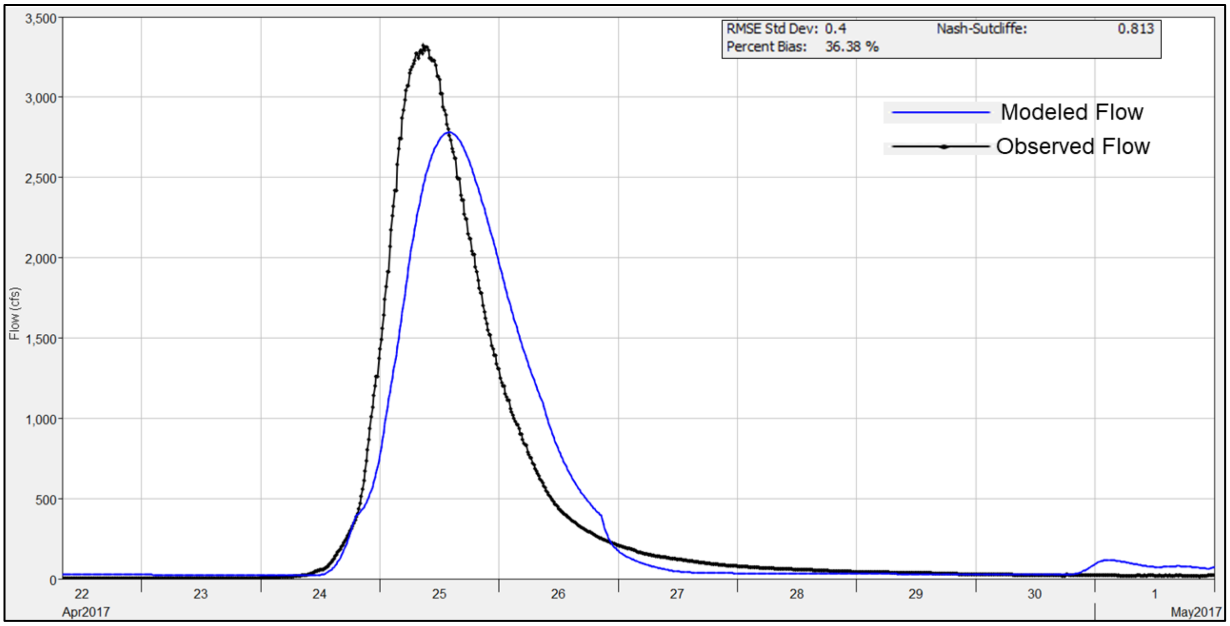

Figure 54. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Swift Creek near McCullars Crossroads, NC Gage

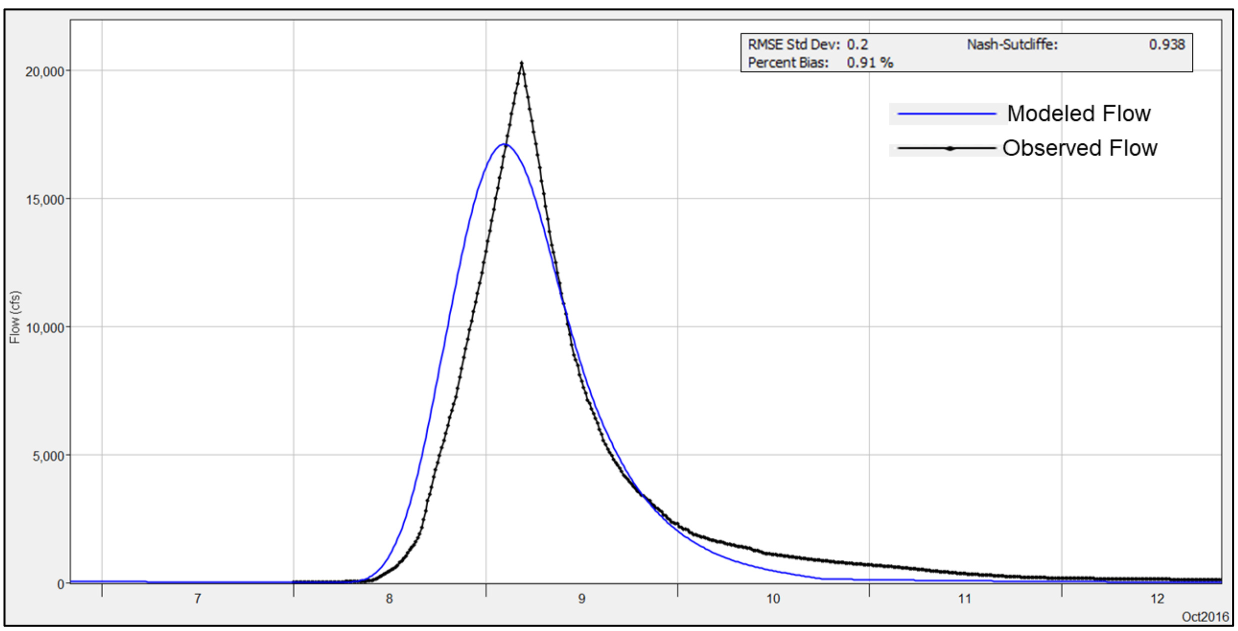

Figure 55. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Middle Creek near Clayton, NC Gage

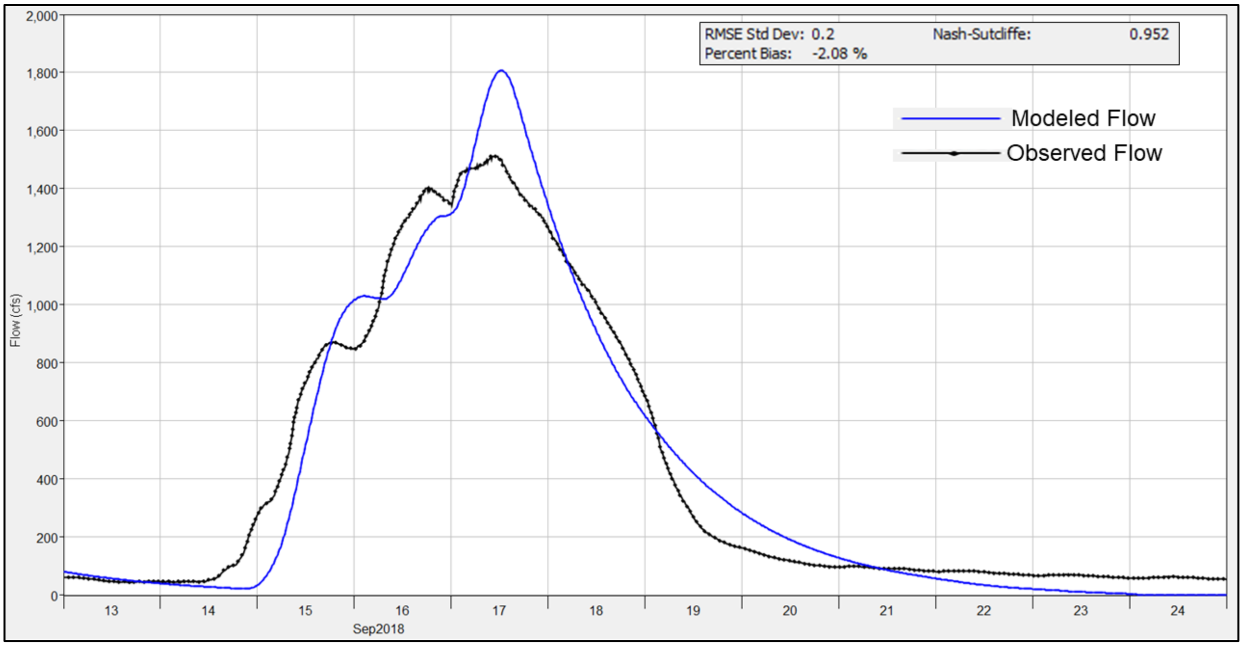

Figure 56. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Middle Creek near Clayton, NC Gage

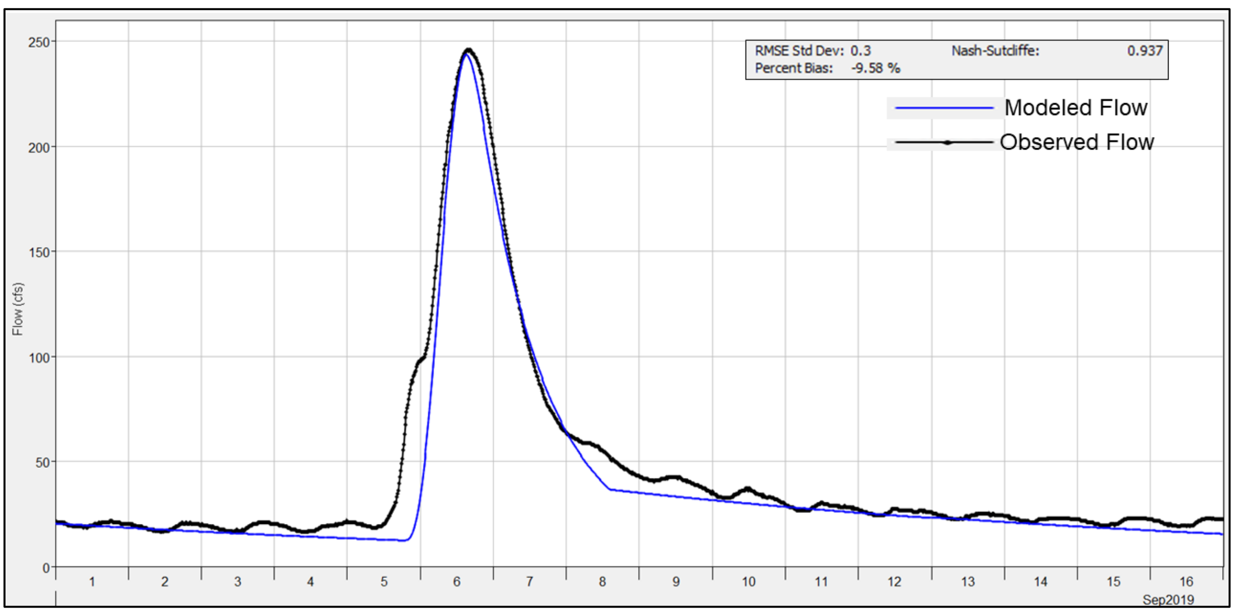

Figure 57. Neuse River Mainstem Basin HEC-HMS September 2019 Calibration at Middle Creek near Clayton, NC Gage

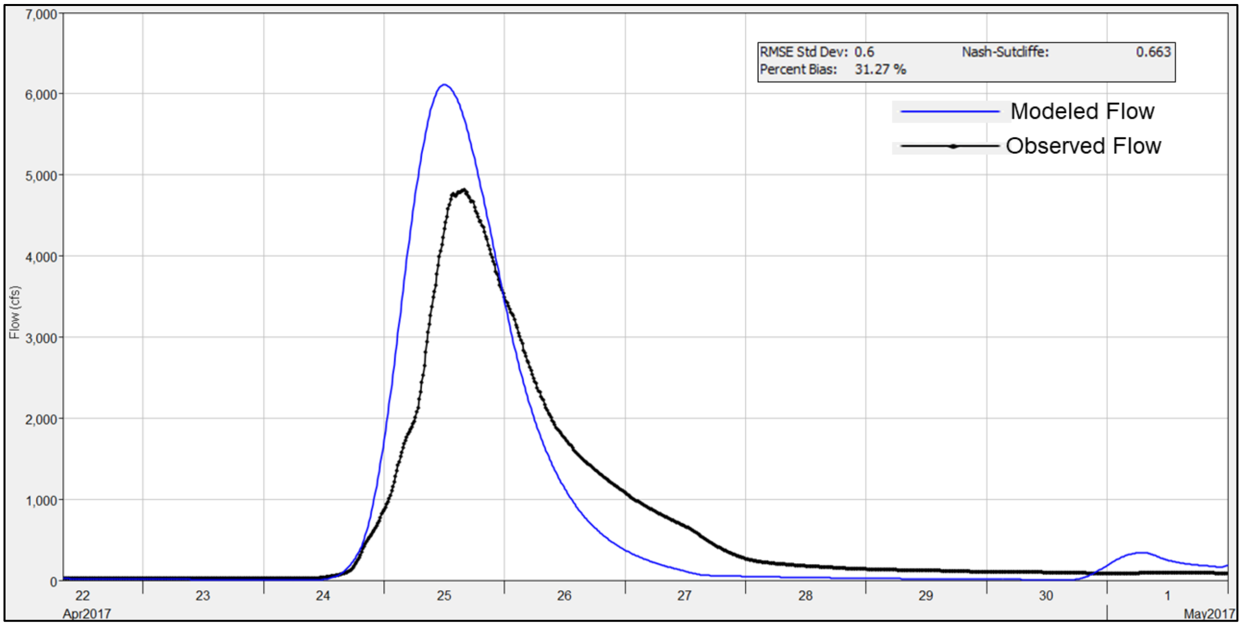

Figure 58. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Middle Creek near Clayton, NC Gage

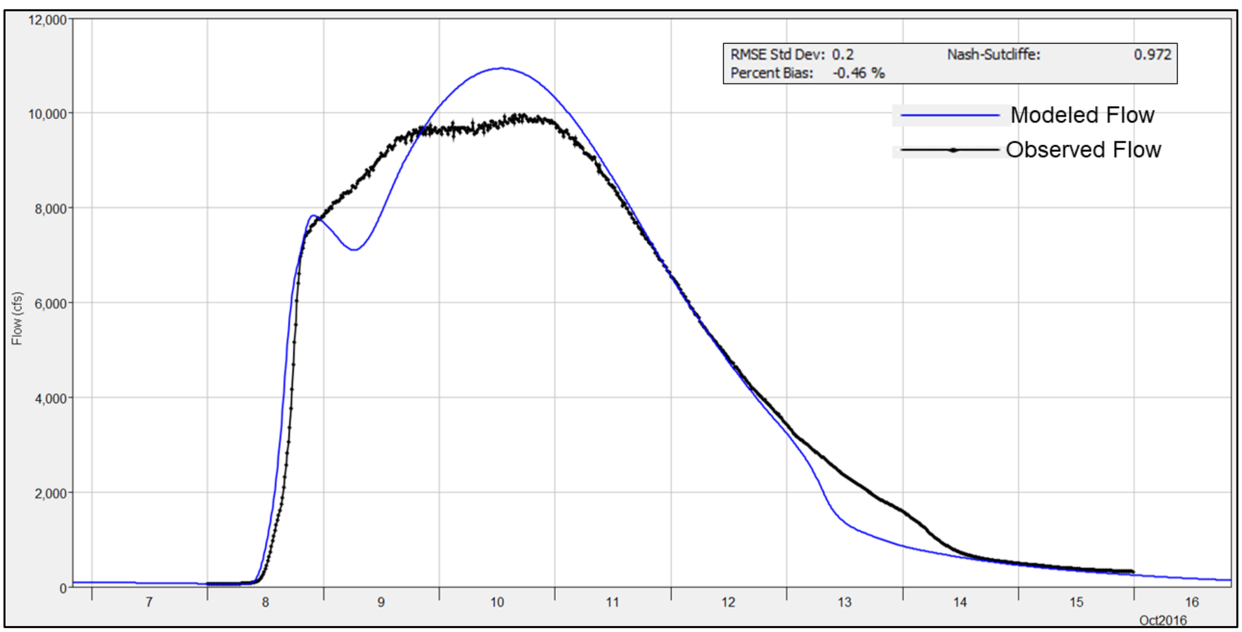

Figure 59. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration Little River near Princeton, NC Gage

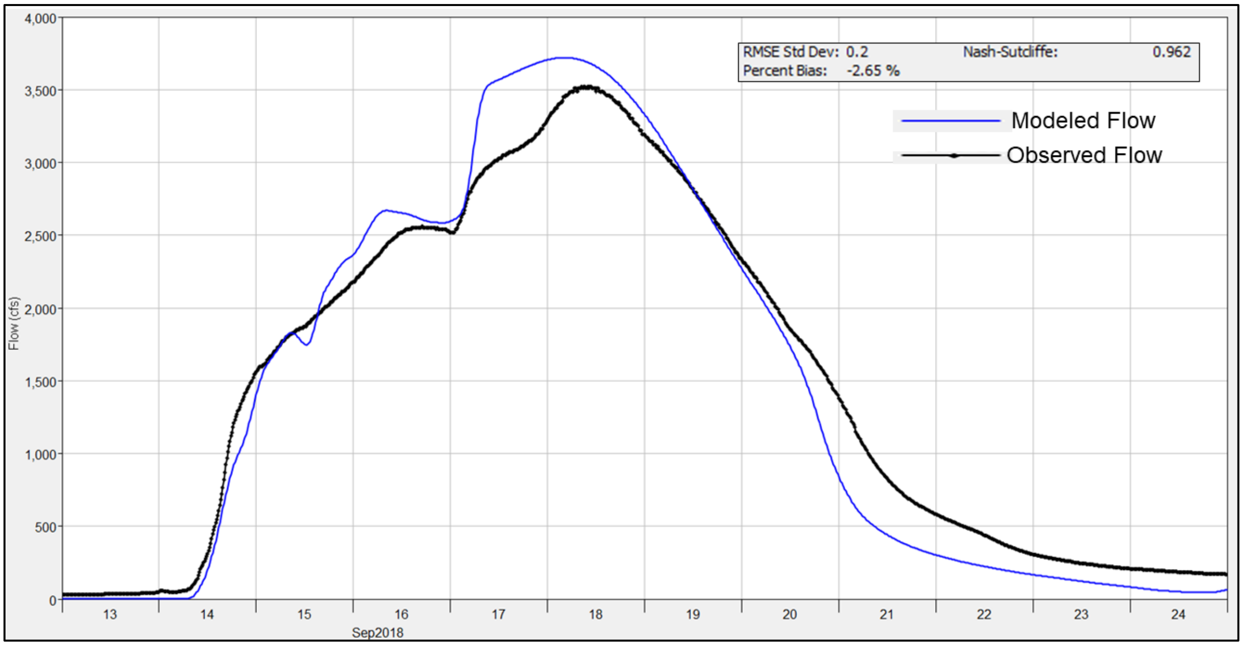

Figure 60. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration Little River near Princeton, NC Gage

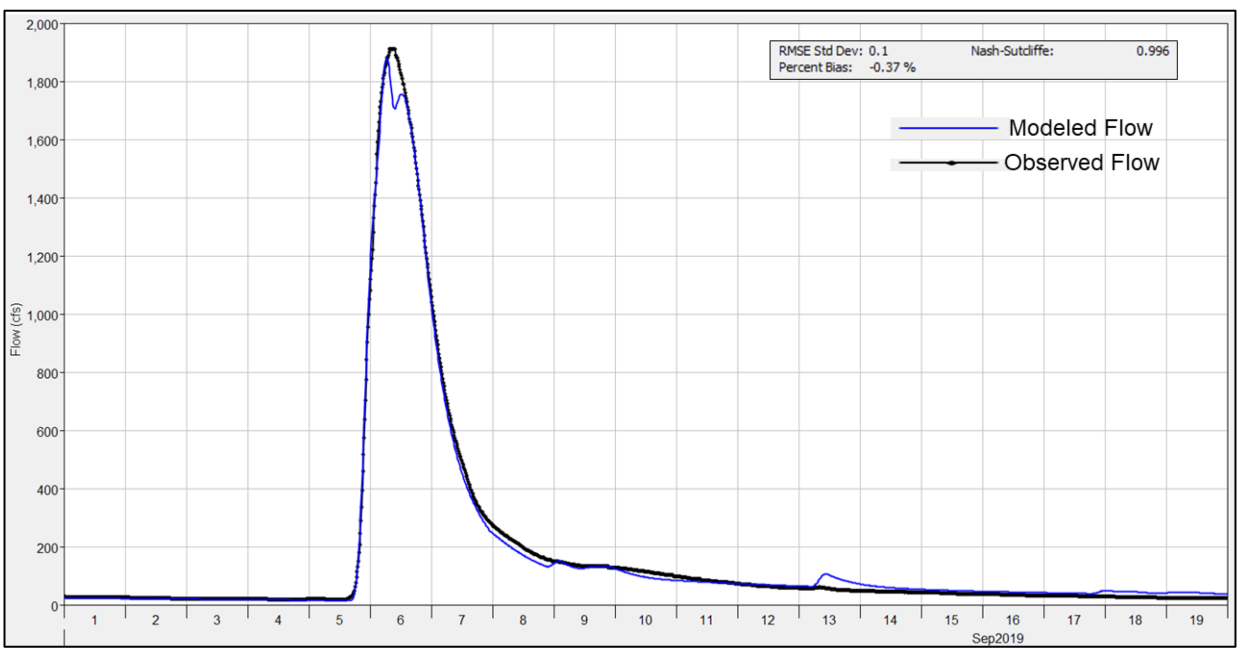

Figure 61. Neuse River Mainstem Basin HEC-HMS September 2019 Calibration Little River near Princeton, NC Gage

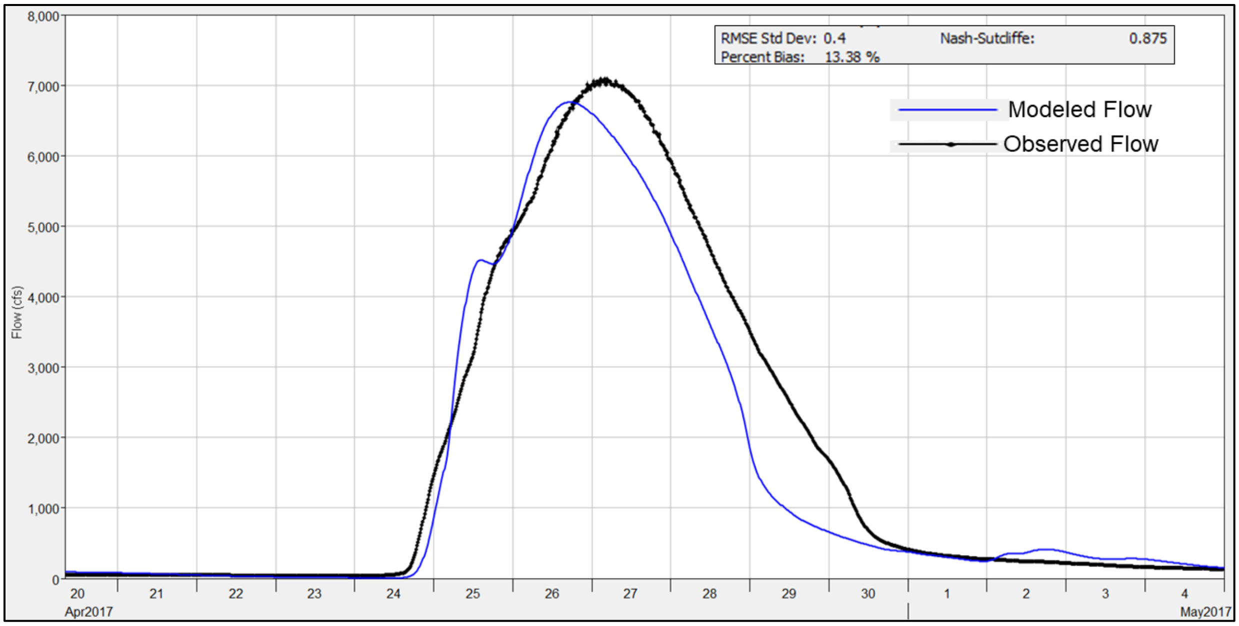

Figure 62. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Little River near Princeton, NC Gage

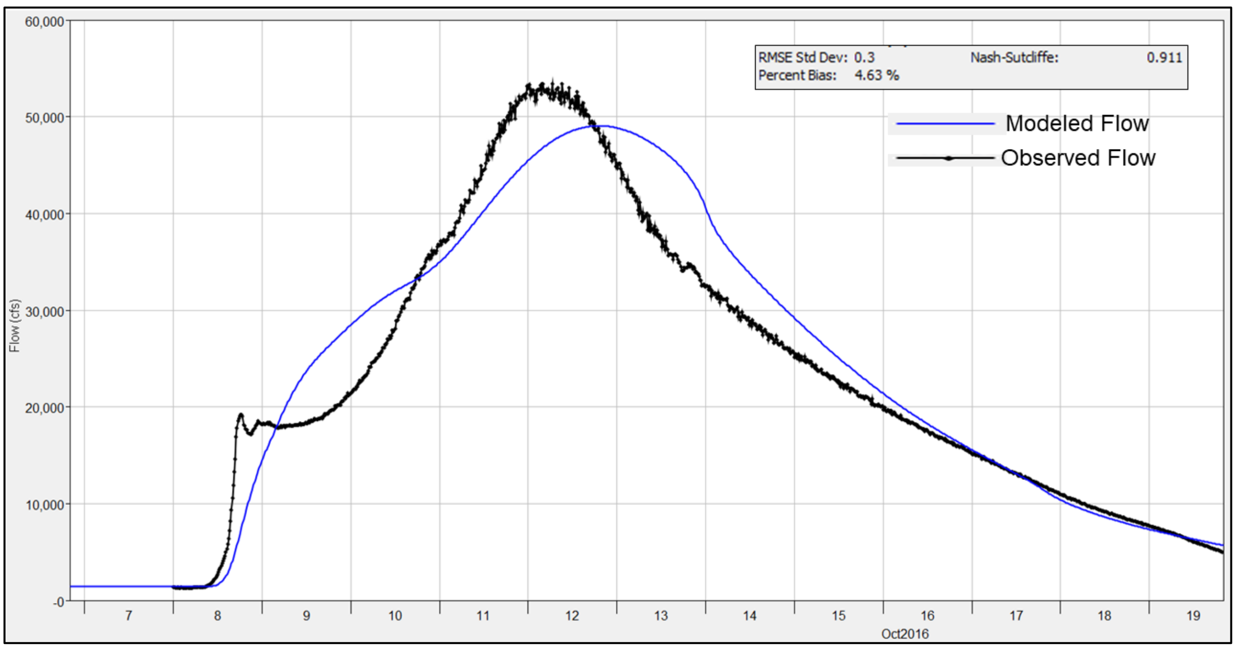

Figure 63. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Neuse River near Goldsboro, NC Gage

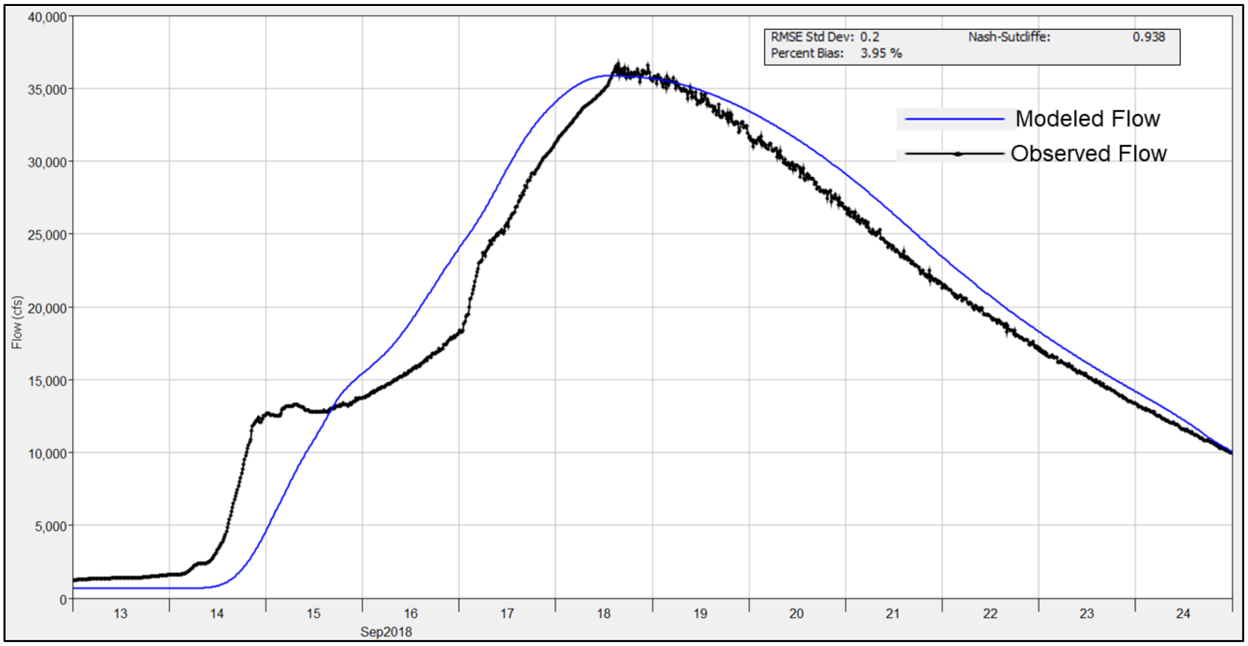

Figure 64. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Neuse River near Goldsboro, NC Gage

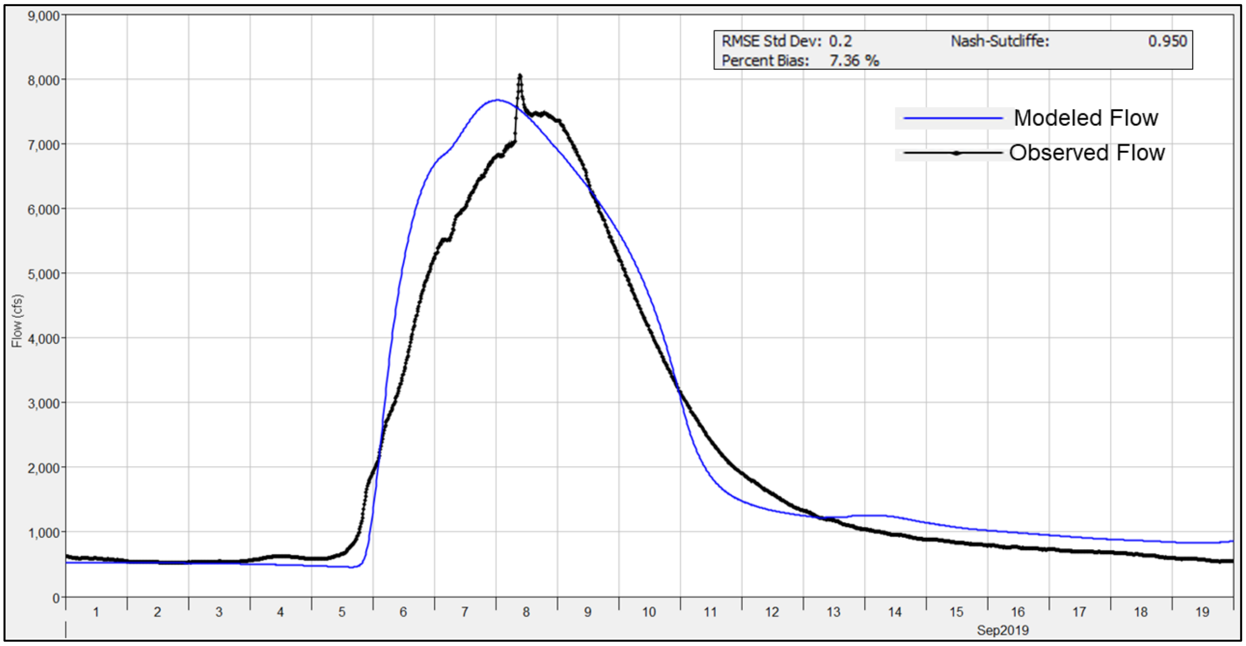

Figure 65. Neuse River Mainstem Basin HEC-HMS September 2019 Calibration at Neuse River near Goldsboro, NC Gage

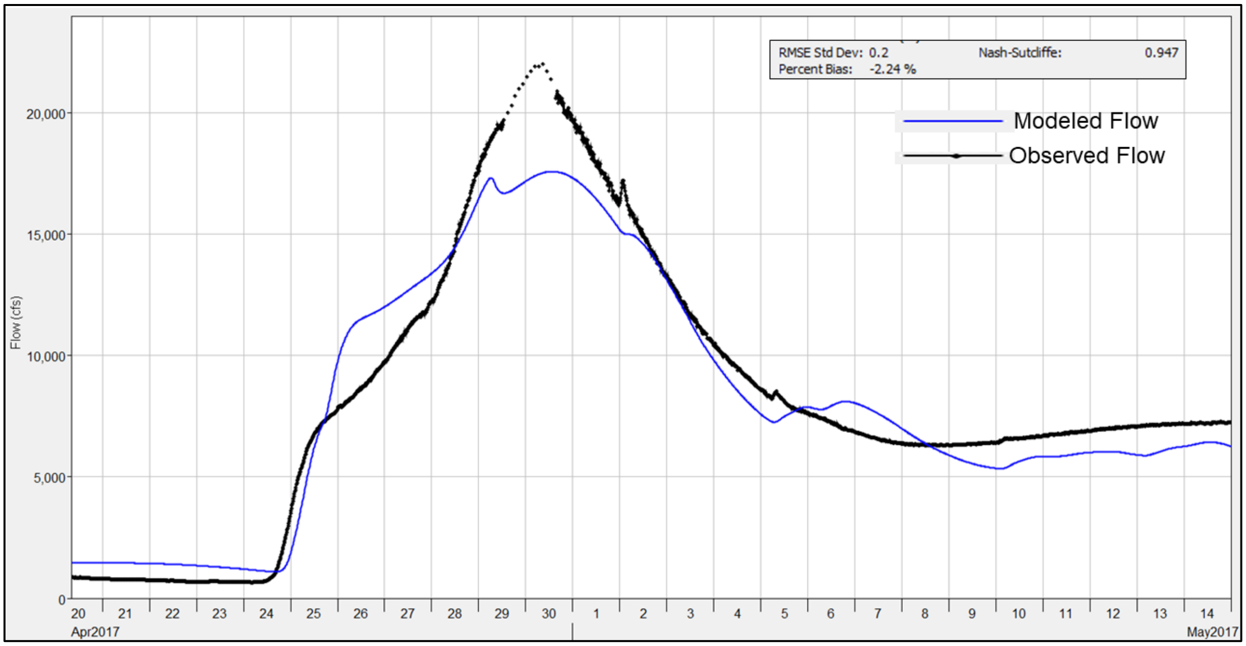

Figure 66. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Neuse River near Goldsboro, NC Gage

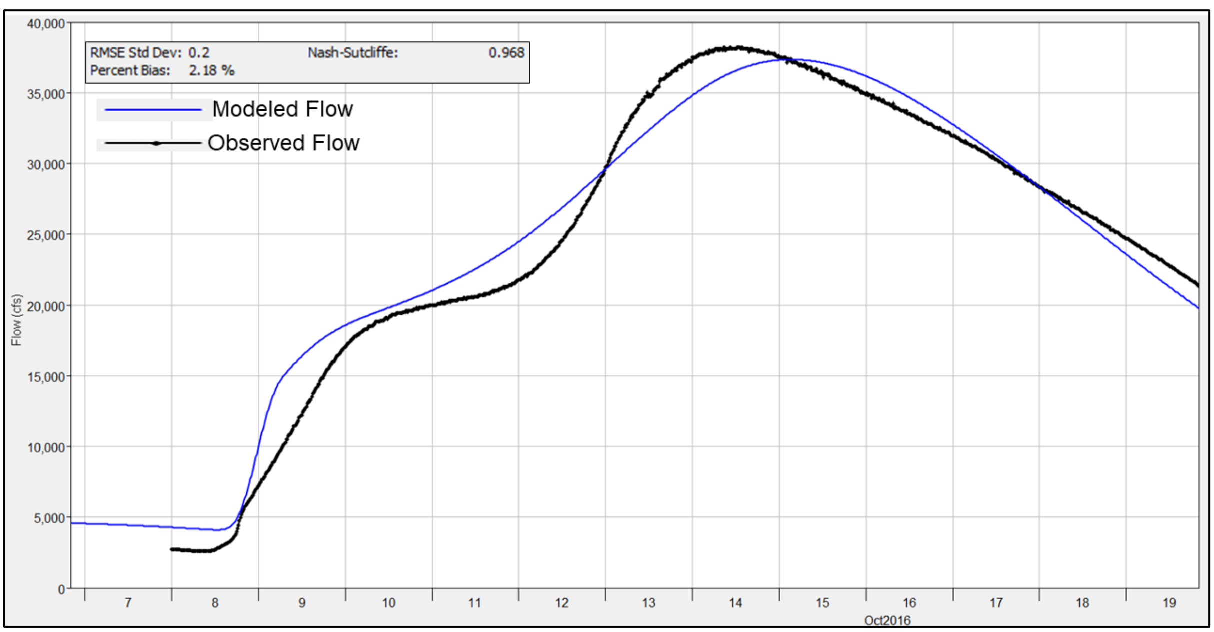

Figure 67. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Neuse River at Kinston, NC Gage

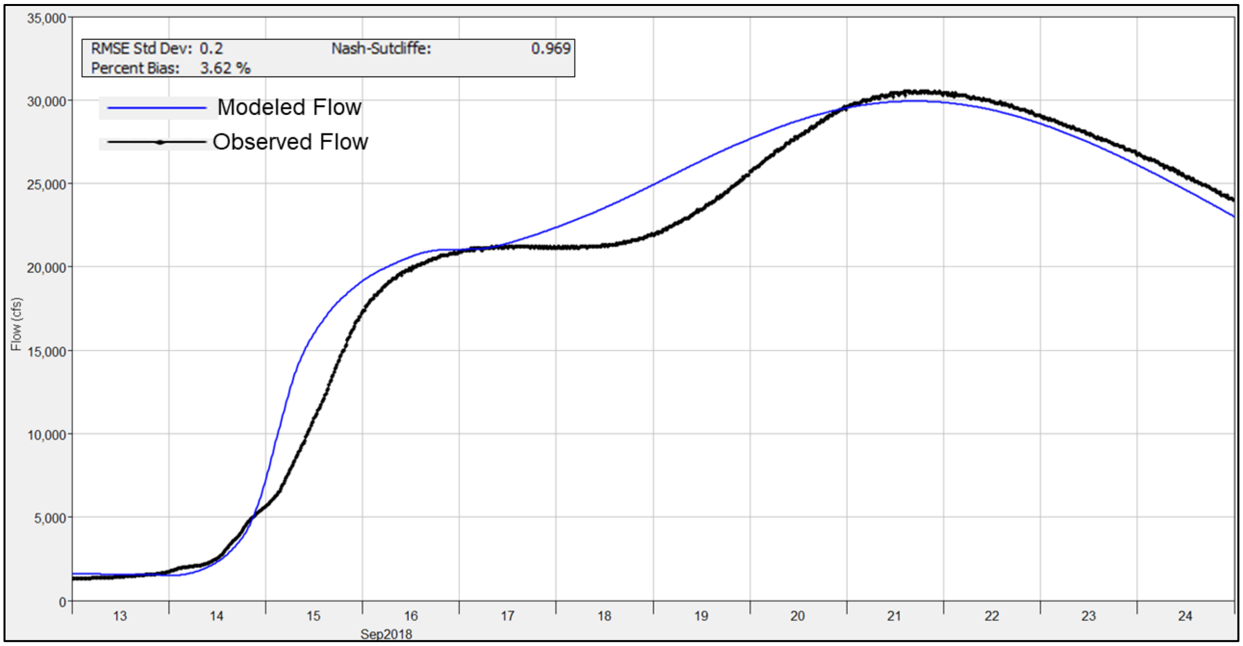

Figure 68. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Neuse River at Kinston, NC Gage

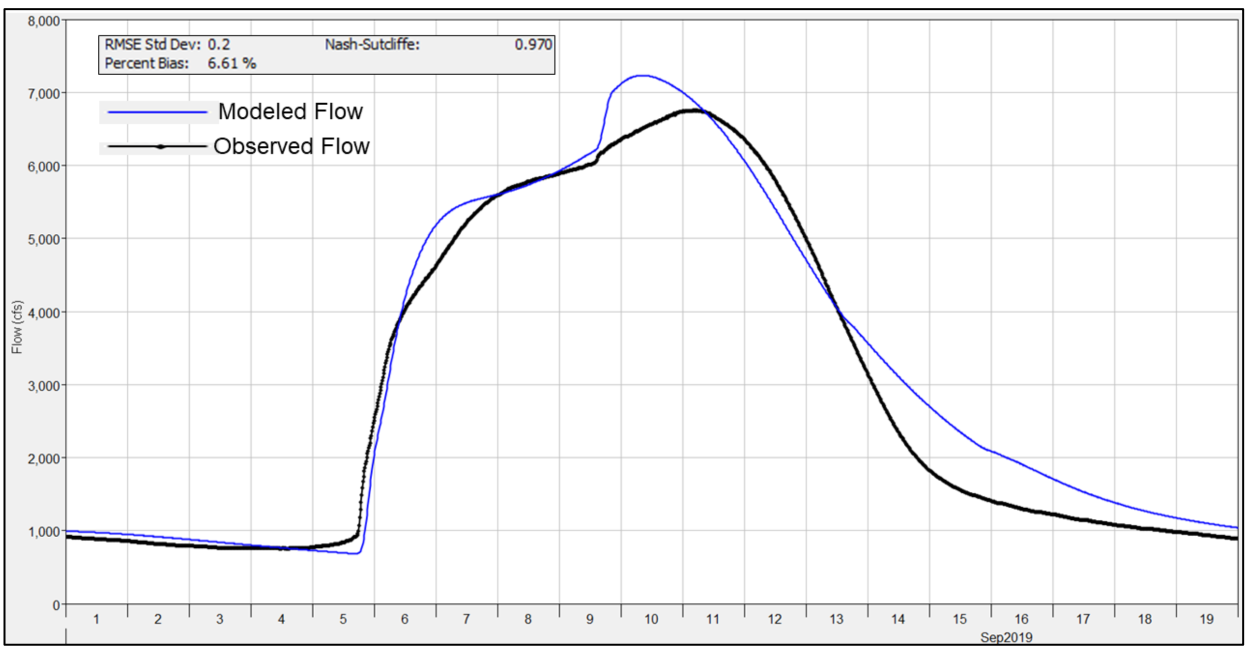

Figure 69. Neuse River Mainstem Basin HEC-HMS September 2019 Calibration at Neuse River at Kinston, NC Gage

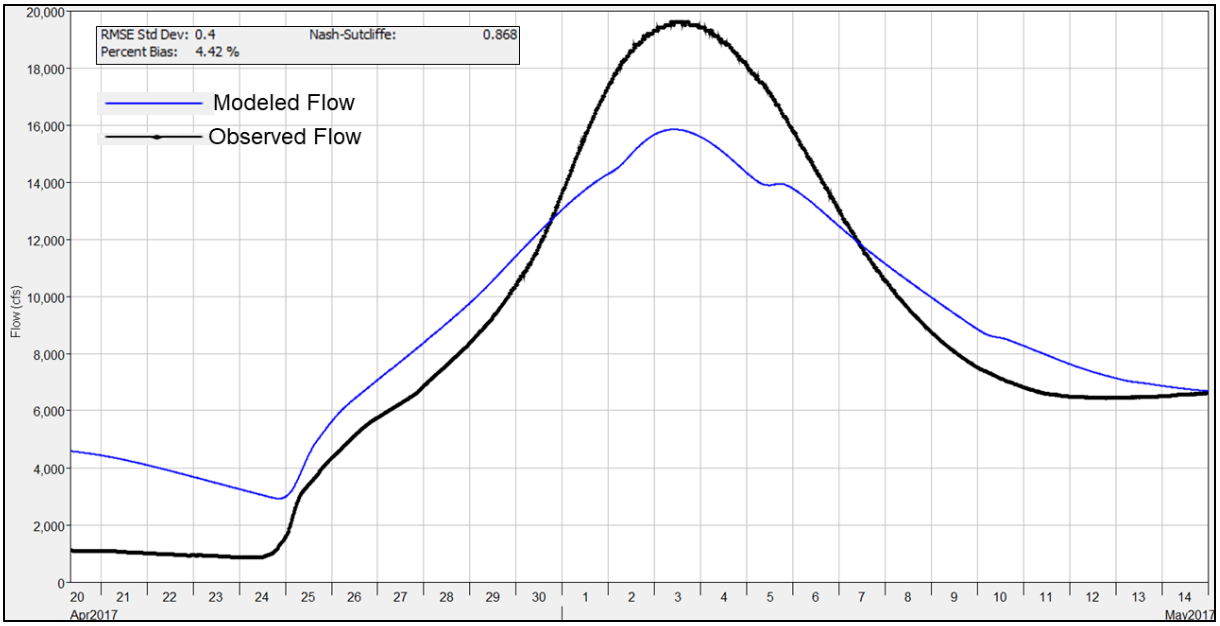

Figure 70. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Neuse River at Kinston, NC Gage

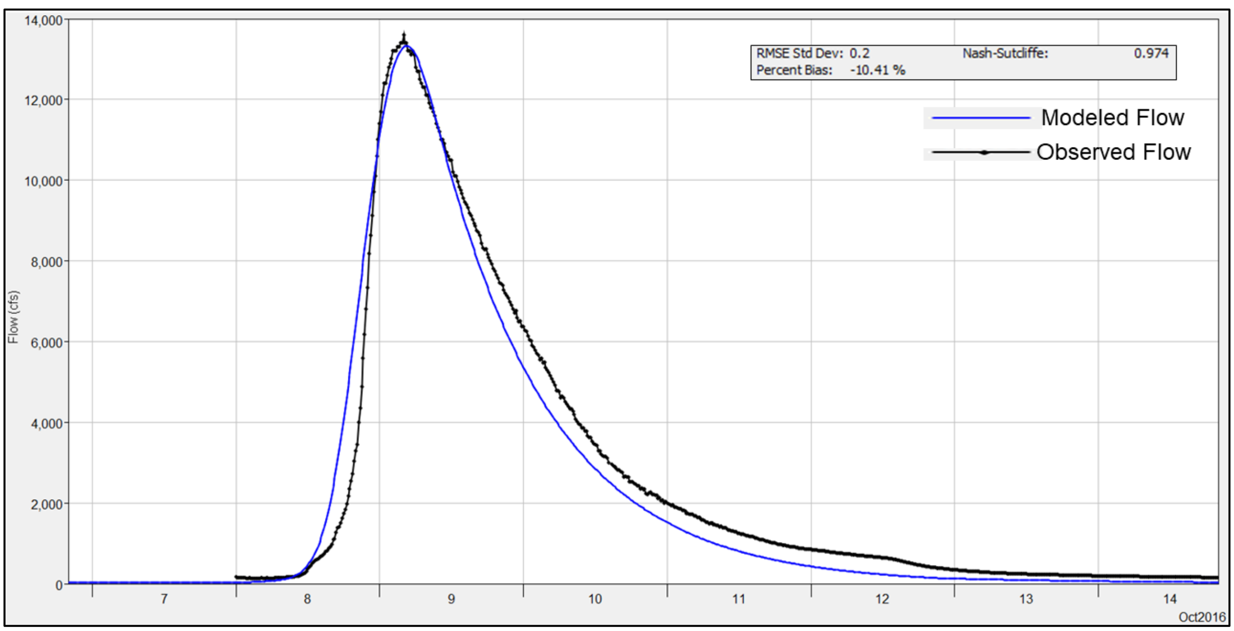

Figure 71. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Nahunta Swamp near Shine, NC Gage

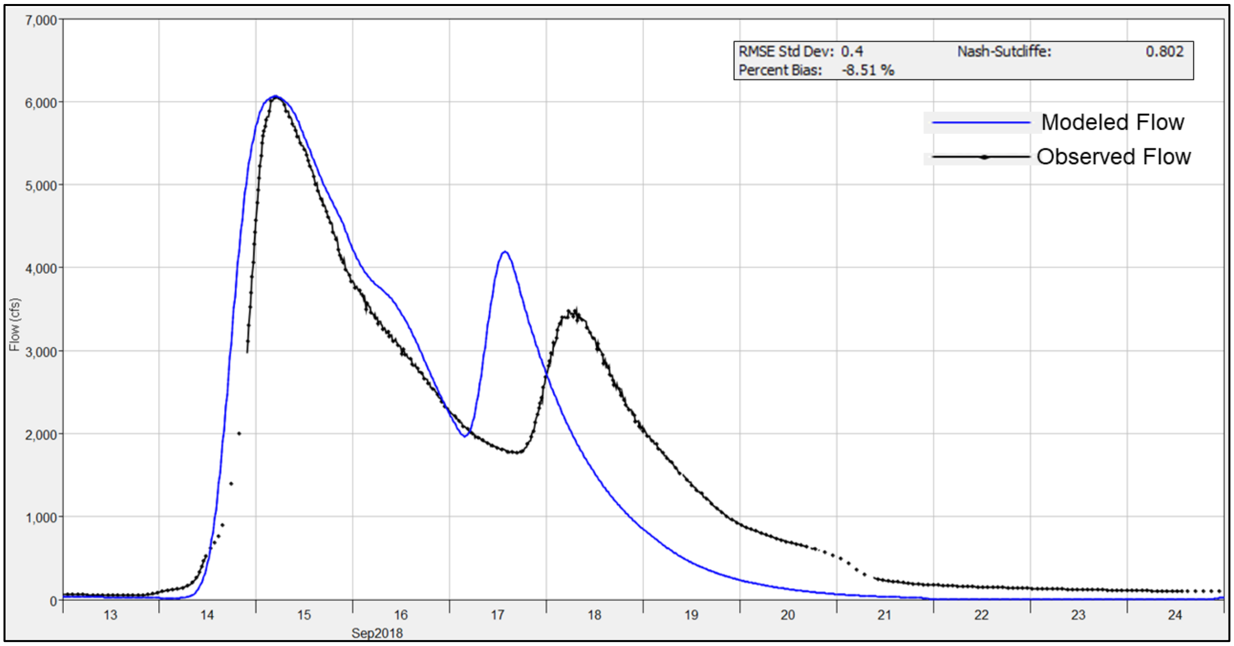

Figure 72. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Nahunta Swamp near Shine, NC Gage

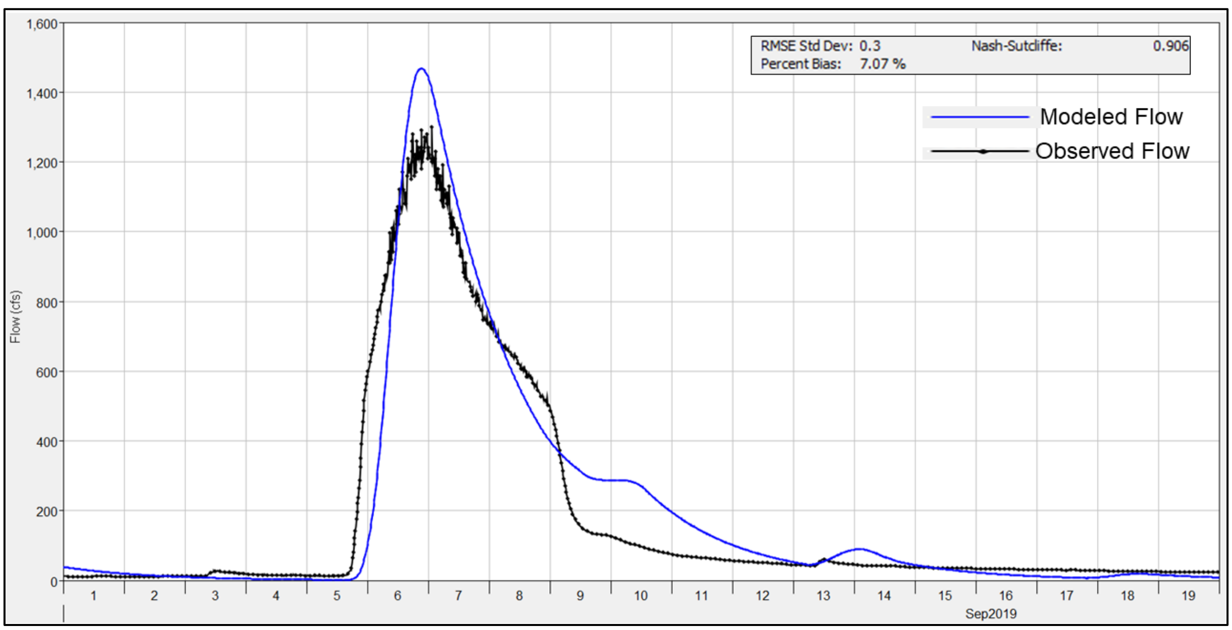

Figure 73. Neuse River Mainstem Basin HEC-HMS September 2019 Calibration at Nahunta Swamp near Shine, NC Gage

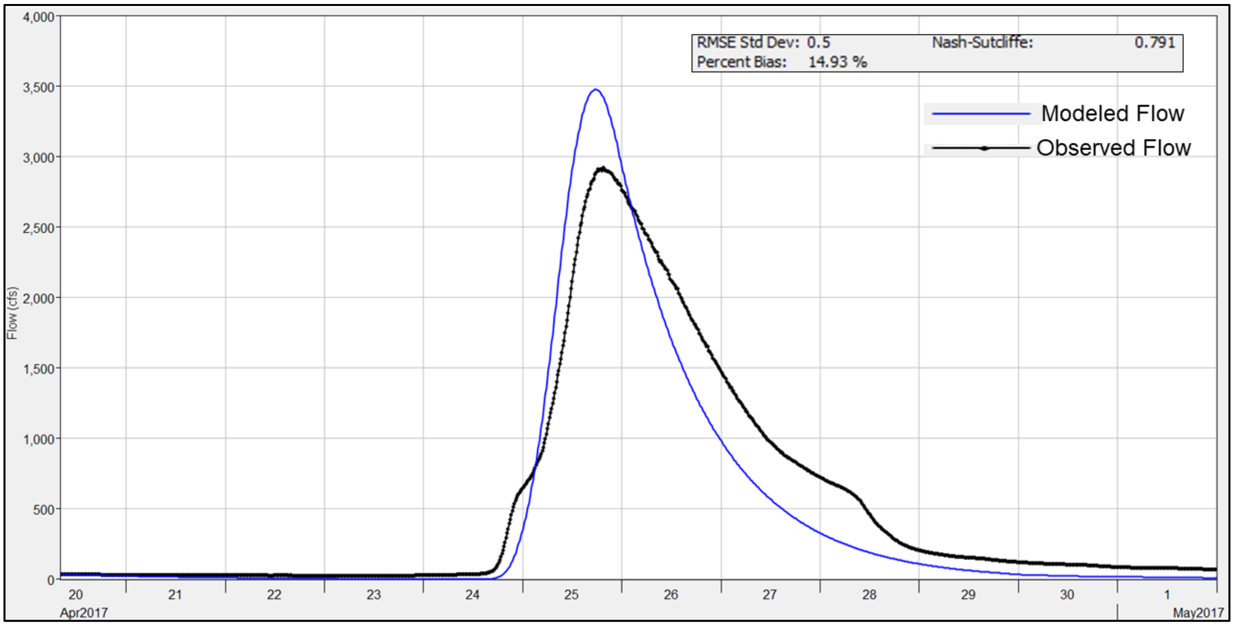

Figure 74. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Nahunta Swamp near Shine, NC Gage

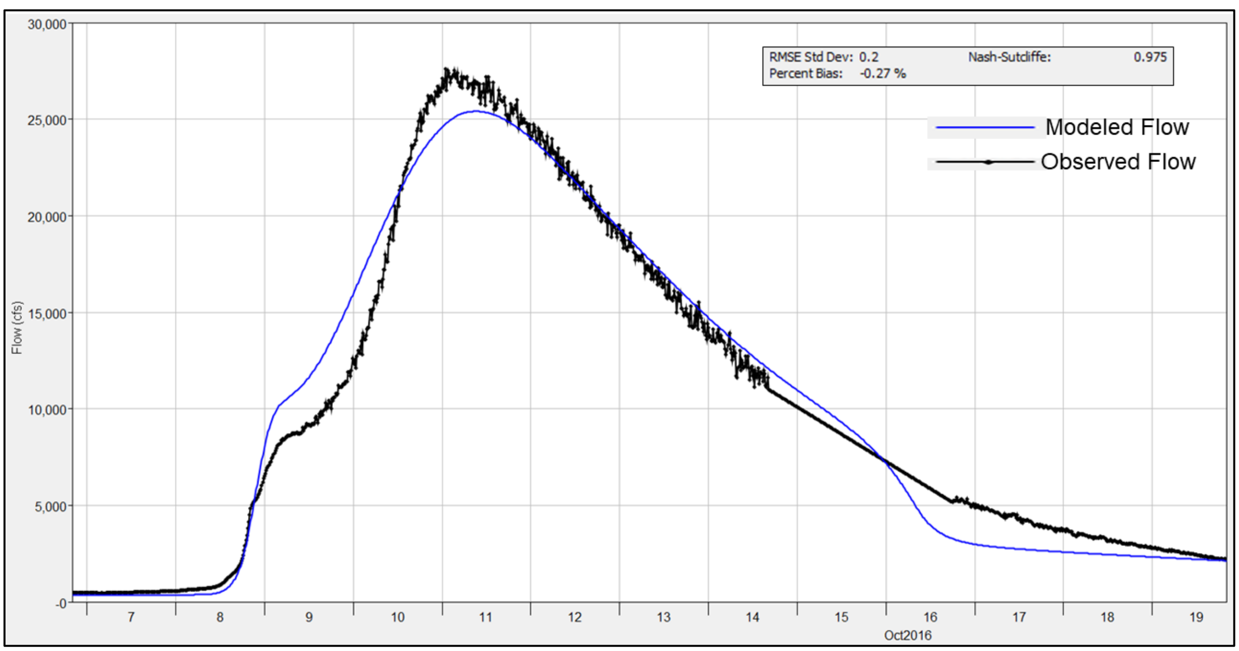

Figure 75. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Contentnea Creek at Hookerton, NC Gage

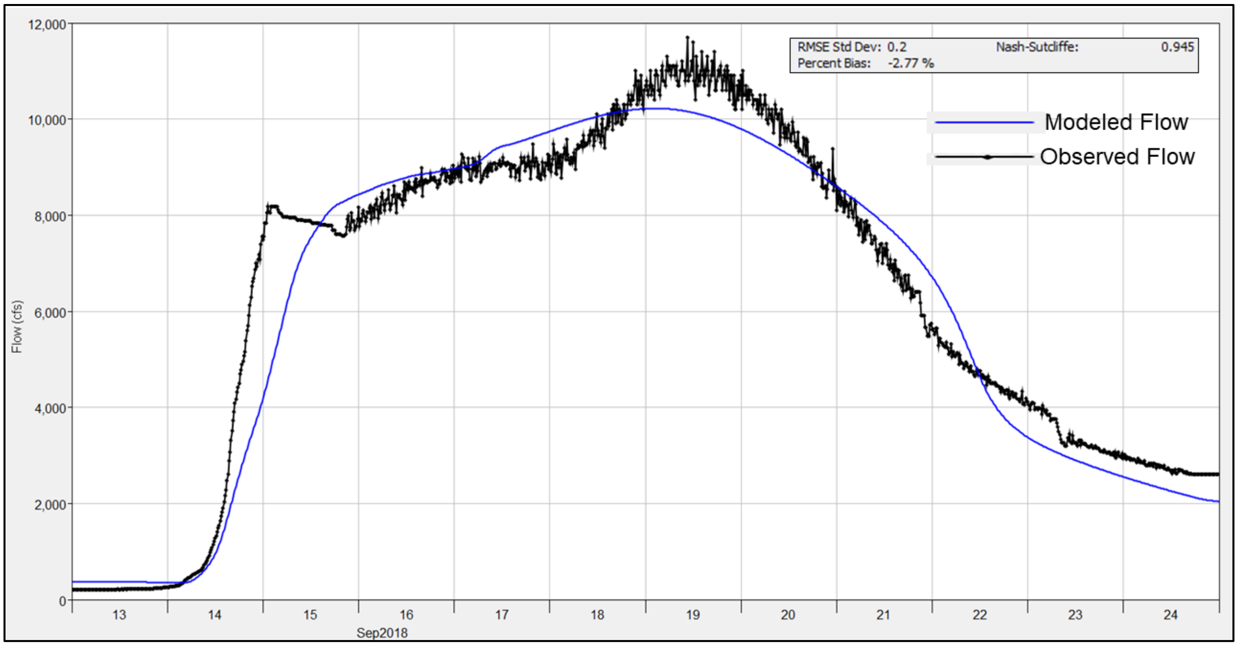

Figure 76. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Contentnea Creek at Hookerton, NC Gage

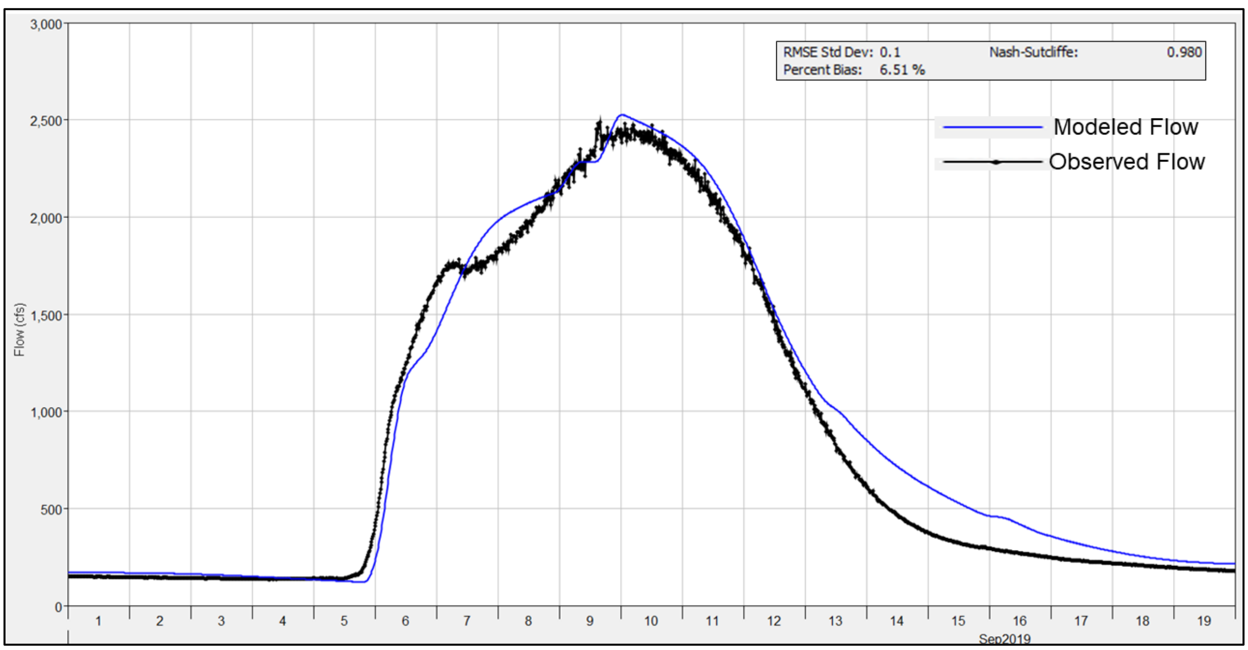

Figure 77. Neuse River Mainstem Basin HEC-HMS September 2019 Calibration at Contentnea Creek at Hookerton, NC Gage

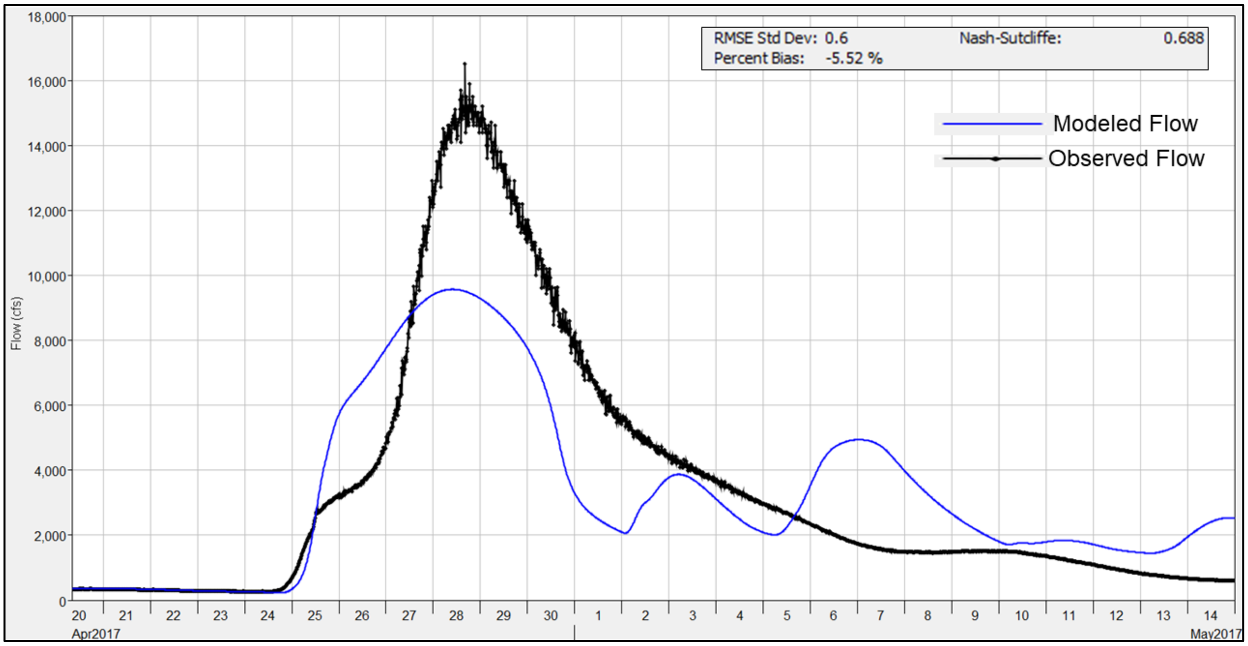

Figure 78. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Contentnea Creek at Hookerton, NC Gage

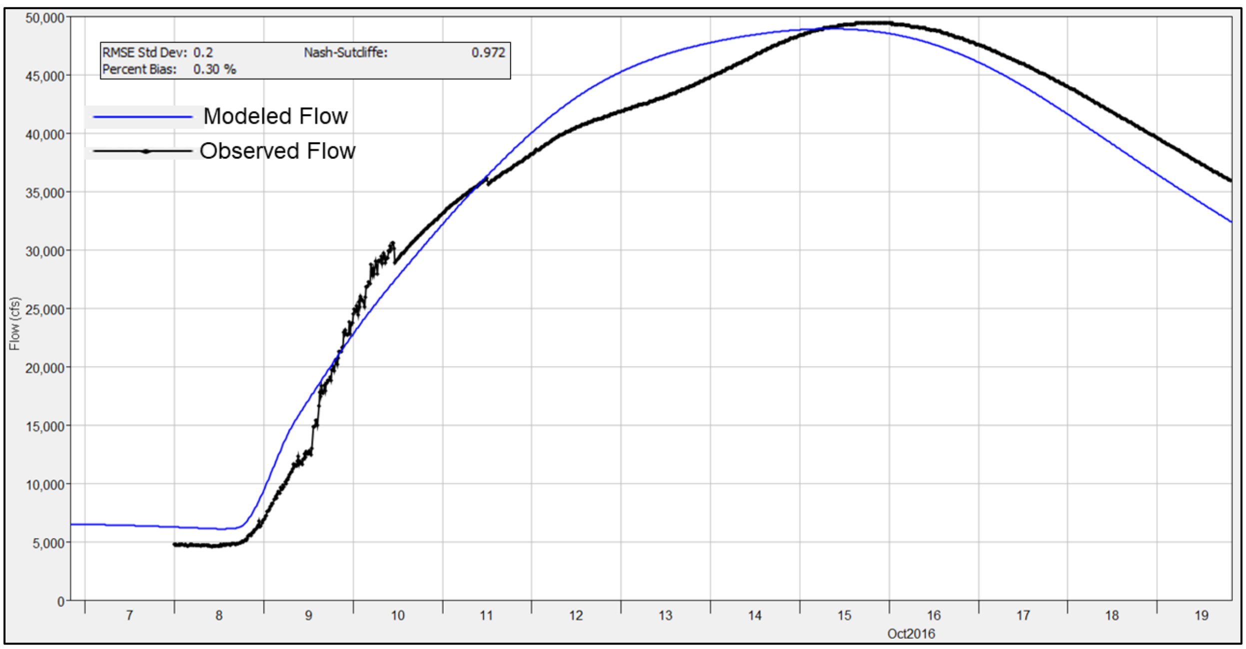

Figure 79. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Neuse River near Fort Barnwell, NC Gage

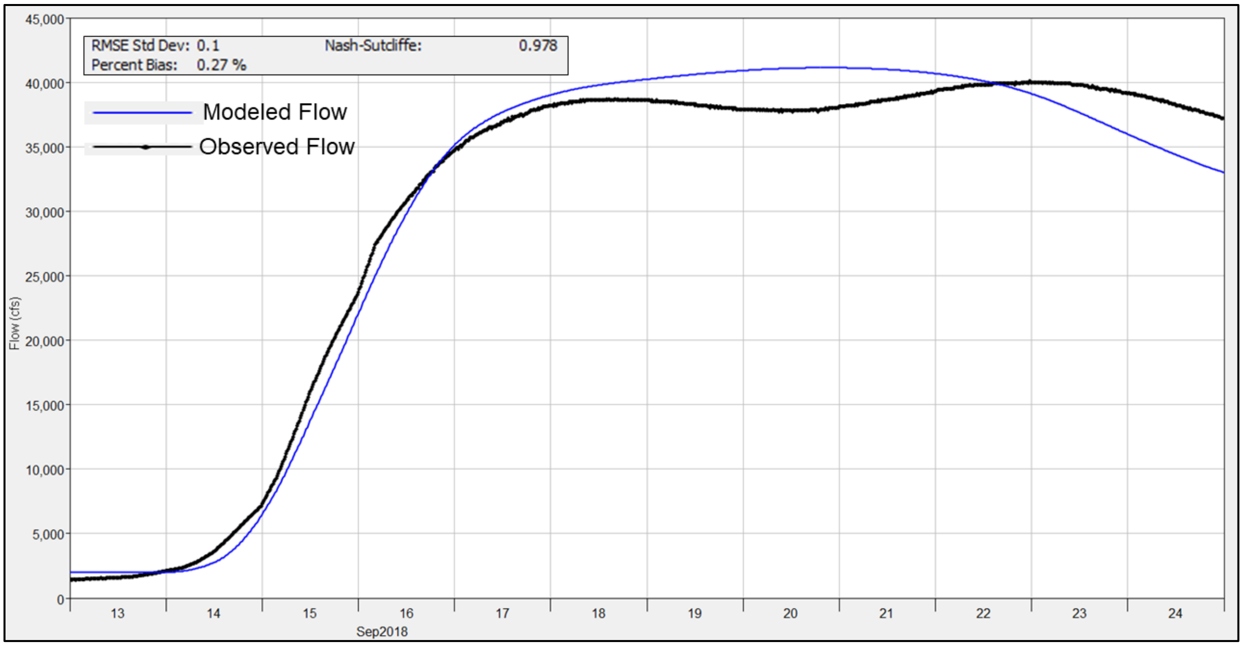

Figure 80. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Neuse River near Fort Barnwell, NC Gage

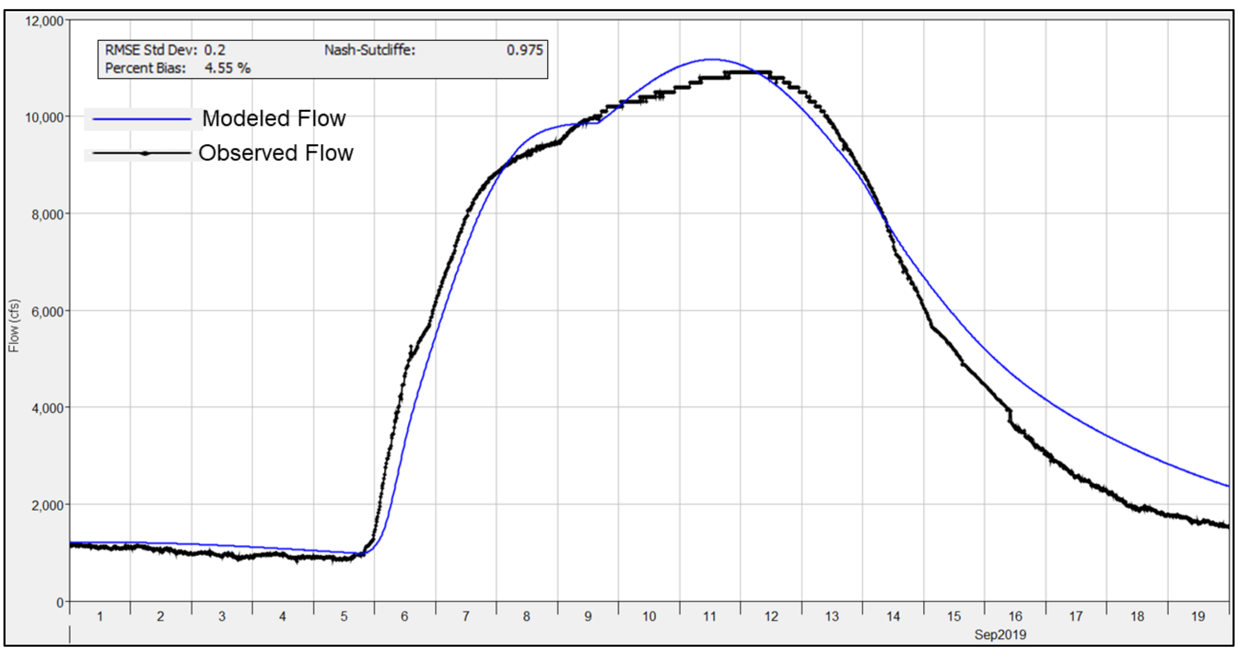

Figure 81. Neuse River Mainstem Basin HEC-HMS September 2019 Calibration at Neuse River near Fort Barnwell, NC Gage

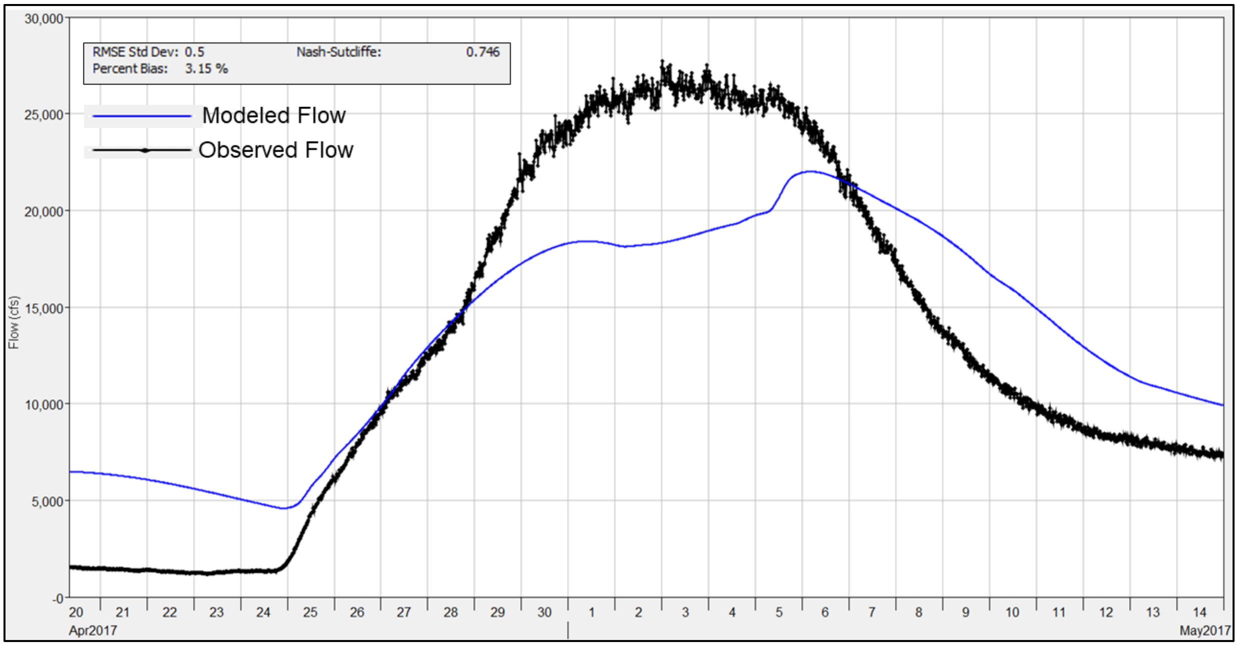

Figure 82. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Neuse River near Fort Barnwell, NC Gage

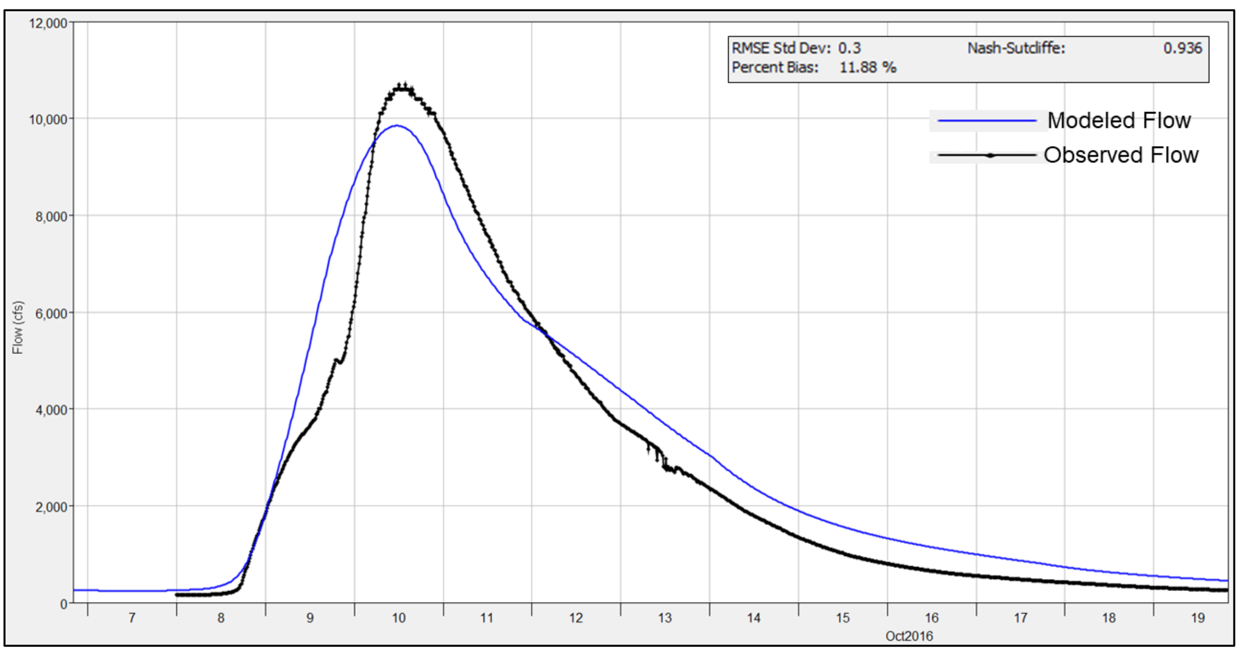

Figure 83. Neuse River Mainstem Basin HEC-HMS Hurricane Matthew Calibration at Trent River near Trenton, NC Gage

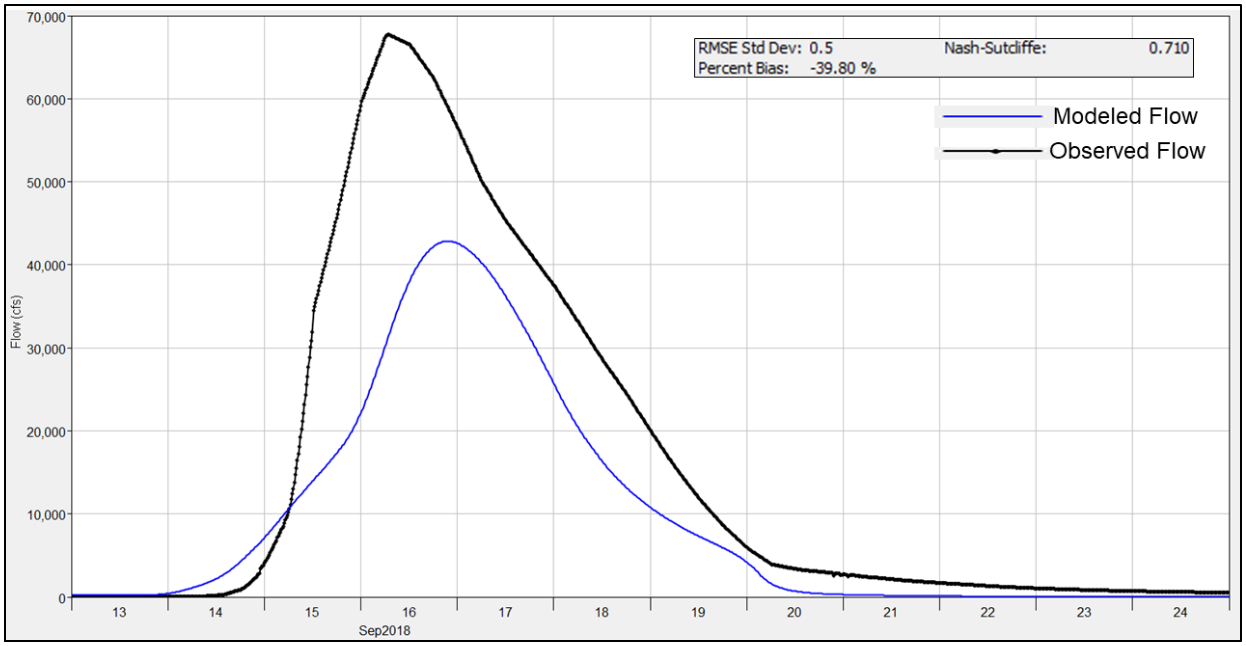

Figure 84. Neuse River Mainstem Basin HEC-HMS Hurricane Florence Calibration at Trent River near Trenton, NC Gage

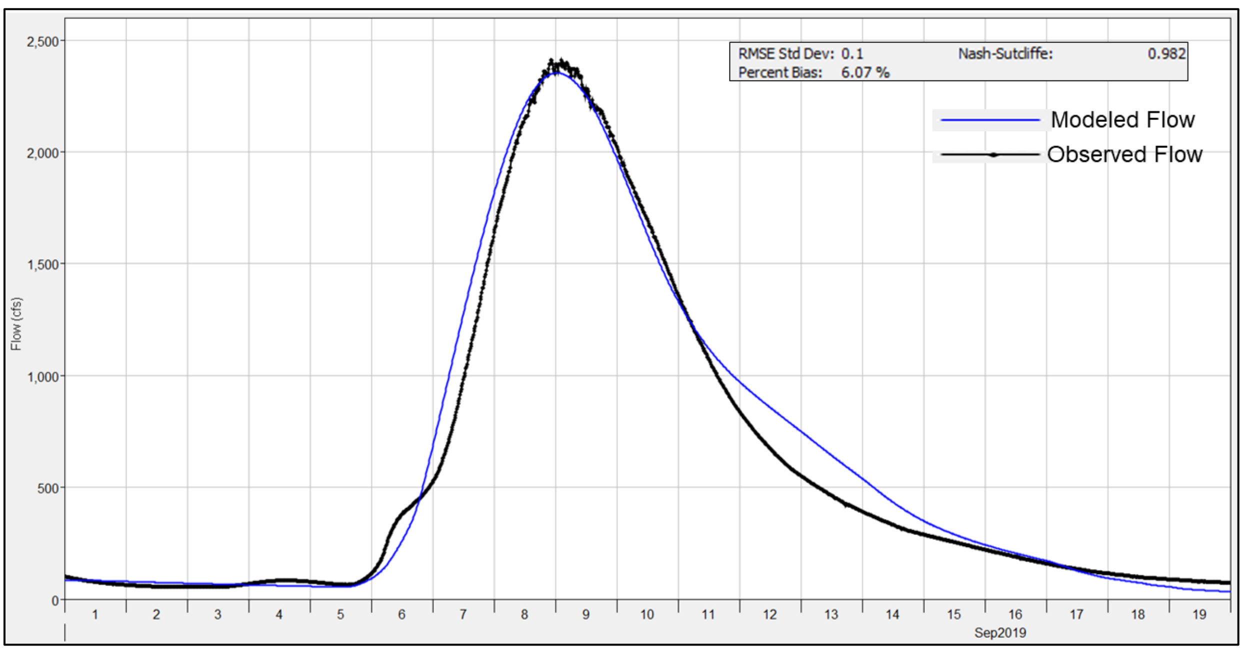

Figure 85. Neuse River Mainstem Basin HEC-HMS September 2019 Calibration at Trent River near Trenton, NC Gage

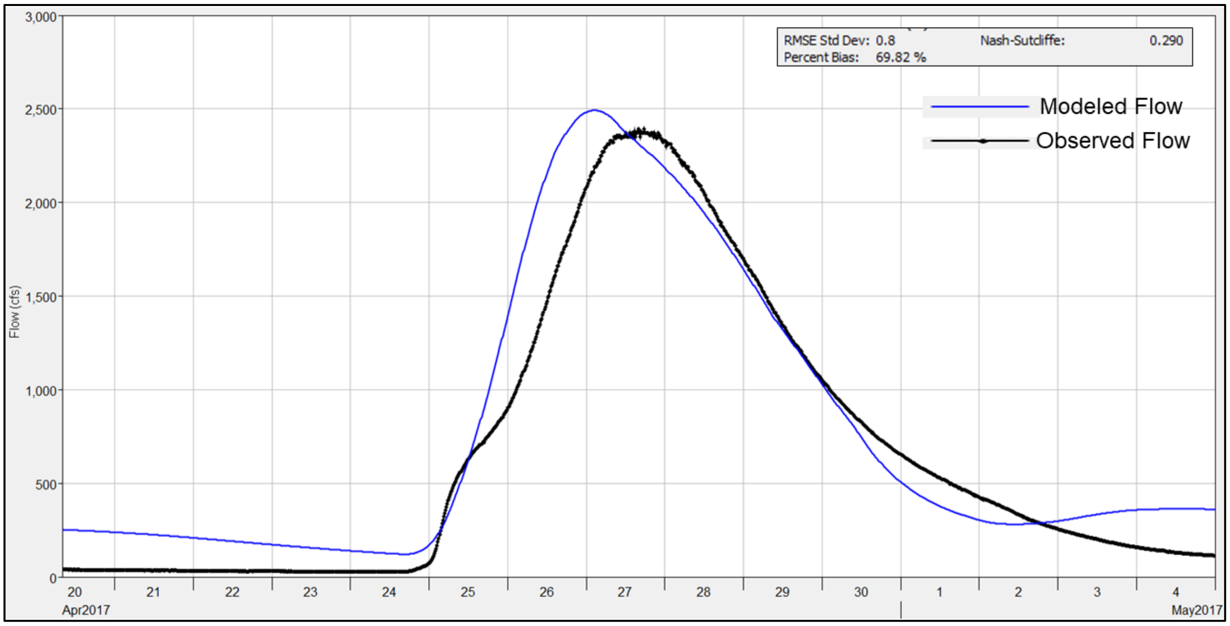

Figure 86. Neuse River Mainstem Basin HEC-HMS April 2017 Validation at Trent River near Trenton, NC Gage

Adjusted model loss, transform, and select routing parameter values for each of the calibrated events as well as final selected values are listed in Table 27 through Table 33 below.

Table 27. Adjusted Neuse River Mainstem Basin HEC-HMS Model Loss Method Curve Number Parameter for Calibration Events and Final Selected Value

Subbasin	Curve Number				
	Initial	Matthew-Calibrated	Florence-Calibration	2019-Calibrated	Final
B10	65.5	68.7	55.0	70.1	66.9
B11	65.5	83.8	72.0	65.5	74.0
B15	65.9	65.9	56.0	59.3	67.5
B16	64.7	73.7	58.2	64.7	71.2
B19	64.8	87.4	58.5	60.8	69.2
B21	61.4	75.8	59.9	60.7	67.6
B23	65.0	65.0	63.7	65.0	65.5
B24	69.4	69.4	59.4	81.1	70.0
B25	65.4	65.4	45.8	52.3	57.2
B26	68.2	78.4	70.6	68.5	72.2
B28a	66.9	73.5	61.0	70.6	70.7
B28b	66.8	74.3	66.8	79.0	71.4
B29a	63.0	63.0	53.6	56.7	66.3
B29b	69.1	94.1	55.8	64.7	71.5
B30	62.0	62.0	64.1	67.7	67.9
B31	64.4	70.9	58.0	64.4	66.7
B32	69.5	76.4	63.4	73.3	71.8
B35	62.0	62.0	64.1	62.3	63.5
B37	67.1	63.8	53.7	67.1	66.0
B39a	71.1	64.0	71.1	71.1	75.2
B39b	74.5	67.0	74.5	74.5	72.3
B40	67.1	74.1	70.4	65.2	71.0
B41	67.4	77.6	69.8	67.8	71.1
B43	66.2	72.9	72.9	65.6	68.0
B44	64.5	74.2	52.0	59.4	64.9
B46	70.0	70.0	70.0	70.0	70.5
B47	64.1	76.9	69.2	67.2	67.9
B49	65.7	75.5	52.9	56.6	65.1
B5	68.3	84.0	68.3	68.3	77.4
B50	63.5	69.9	48.9	55.9	60.3
B52	64.4	65.9	62.6	64.6	64.8
B53	65.4	71.9	59.7	69.0	68.1

B54	65.9	60.3	57.2	59.1	59.2
B55	62.6	68.9	57.2	66.1	66.4
B56	63.1	63.1	65.3	63.4	64.3
B59	69.6	80.1	72.1	70.0	71.7
B6	70.3	70.3	70.3	70.3	69.5
B60	65.5	75.3	67.8	65.8	66.0
B60b	60.9	70.0	63.0	61.2	62.9
B61	58.6	67.4	60.6	58.9	60.8
B62	60.9	67.0	56.9	67.0	65.9
B62d	64.8	64.6	54.9	64.6	63.5
B62f	60.0	66.0	56.1	66.0	68.1
B62h	64.7	63.3	53.8	63.3	62.4
B63a	62.1	71.3	67.8	69.9	68.7
B63d	67.3	56.1	53.3	55.0	57.9
B64	69.5	65.3	62.0	64.0	67.3
B66a	64.3	64.3	64.3	64.3	66.0
B67	65.0	73.0	73.0	73.0	79.5
B68a	65.0	71.9	77.7	77.7	71.4
B68b	65.0	69.2	69.2	69.2	68.3
B68c	75.0	65.6	65.6	65.6	66.5
B68d	65.0	68.3	68.3	68.3	70.7
B68e	65.0	94.6	99.0	68.5	78.8
B68f	75.0	95.1	94.8	64.6	76.2
B69	65.0	65.6	65.6	65.6	63.7

Table 28. Adjusted Neuse River Mainstem Basin HEC-HMS Model Transform Method Time of Concentration Parameter for Calibration Events and Final Selected Value

Subbasin	Time of Concentration (hr)				
	Initial	Matthew-Calibrated	Florence-Calibrated	2019-Calibrated	Final
B10	5.4	2.5	2.5	6.1	6.3
B11	7.1	5.5	5.9	6.9	7.1
B15	10.7	11.3	11.3	4.5	14.8
B16	6.4	5.7	5.7	11.3	7.3
B19	8.9	15.2	10.2	22.8	16.6
B21	8.6	17.0	14.0	18.7	15.0
B23	4.5	6.4	6.4	6.4	6.4
B24	8.8	4.7	4.7	2.7	3.8
B25	4.9	9.0	22.5	9.0	12.2
B26	11.3	15.7	33.4	10.4	19.9
B28a	3.7	7.4	4.4	3.7	11.1
B28b	5.6	5.7	8.7	3.2	5.8
B29a	11.3	12.8	12.8	5.1	8.8
B29b	6.9	7.4	24.4	7.4	9.7
B30	10.5	8.3	35.4	11.1	17.7
B31	7.5	2.1	3.7	10.7	5.9
B32	5.3	6.9	4.9	3.4	5.6
B35	13.3	7.5	30.2	9.4	18.0
B37	4.1	8.9	6.2	8.9	10.1
B39a	10.4	31.1	9.9	9.9	27.0
B39b	10.5	28.5	9.1	9.1	17.0
B40	13.5	12.0	14.5	12.0	12.6
B41	12.6	16.9	36.0	11.2	24.8
B43	8.2	3.5	4.5	5.3	7.9
B44	13.7	13.2	33.0	26.4	23.0
B46	1.3	7.5	7.5	7.5	13.7
B47	13.8	15.1	32.1	30.0	18.5
B49	14.4	10.5	26.3	21.0	18.3
B5	8.5	9.8	8.9	8.9	10.9
B50	9.9	10.0	7.0	10.0	9.7
B52	7.9	6.0	11.9	6.0	7.5
B53	9.3	11.2	6.2	5.6	8.0

B54	7.1	3.1	6.7	3.1	4.6
B55	8.2	10.0	8.0	5.0	6.6
B56	9.2	12.1	25.8	8.1	17.4
B59	13.2	15.3	32.6	10.2	18.0
B6	13.4	14.0	14.0	14.0	12.7
B60	14.0	23.3	49.7	15.5	17.5
B60b	8.9	14.2	30.2	9.4	16.6
B61	2.9	4.7	15.2	4.7	7.4
B62	11.9	10.8	10.8	10.8	10.6
B62d	9.1	14.4	14.4	14.4	14.0
B62f	7.5	10.0	10.0	10.0	12.8
B62h	10.3	12.9	12.9	5.4	11.9
B63a	9.1	17.9	21.4	17.9	17.5
B63d	8.3	13.6	16.3	13.6	12.3
B64	7.5	13.7	16.4	9.6	11.5
B66a	14.4	8.3	8.3	8.3	8.7
B67	10.6	21.4	21.4	21.4	34.2
B68a	15.7	18.5	50.1	50.1	31.1
B68b	11.8	41.7	41.7	41.7	34.6
B68c	8.3	35.1	35.1	35.1	36.8
B68d	6.5	24.5	24.5	24.5	30.9
B68e	8.0	52.4	72.9	81.6	54.2
B68f	6.4	14.2	57.0	95.8	53.4
B69	3.5	13.2	13.2	13.2	10.9

Table 29. Adjusted Neuse River Mainstem Basin HEC-HMS Model Transform Method Storage Coefficient Parameter for Calibration Events and Final Selected Value

Subbasin	Storage Coefficient (hr)				
	Initial	Matthew-Calibrated	Florence-Calibrated	2019-Calibrated	Final
B10	10.0	4.5	25.5	29.5	9.8
B11	13.1	9.6	8.3	12.8	9.8
B15	19.7	37.6	37.6	13.1	34.3
B16	11.9	8.4	7.3	21.0	10.1
B19	16.5	19.0	20.6	36.5	21.9
B21	15.9	8.4	30.5	20.9	10.7
B23	8.4	11.9	3.6	11.9	11.8
B24	16.3	21.8	21.8	16.6	20.1
B25	9.1	25.0	25.0	16.7	23.3
B26	20.9	34.8	41.7	23.2	35.6
B28a	6.9	7.8	7.8	7.8	11.4
B28b	10.4	9.2	9.2	5.6	9.6
B29a	21.0	33.8	33.8	11.8	32.2
B29b	12.8	29.9	11.1	29.9	28.9
B30	19.5	36.9	44.2	24.6	35.9
B31	13.8	9.9	5.9	19.7	10.2
B32	9.8	6.3	6.3	6.3	7.9
B35	24.6	20.9	37.7	20.9	24.8
B37	7.5	19.8	11.6	16.5	21.6
B39a	19.3	61.1	18.3	18.3	47.0
B39b	19.5	56.0	16.8	16.8	45.8
B40	25.0	21.5	59.2	49.5	29.4
B41	23.3	46.8	56.1	11.2	44.2
B43	15.2	20.1	40.1	11.1	19.7
B44	25.3	59.6	89.4	43.9	58.1
B46	2.4	13.8	13.8	13.8	15.8
B47	25.5	41.0	29.5	36.4	36.8
B49	26.6	64.1	96.1	47.2	62.4
B5	15.7	7.4	9.9	16.5	10.3
B50	18.3	34.7	54.3	47.1	38.8
B52	14.6	12.0	33.0	12.0	13.7
B53	17.1	37.9	12.9	9.5	30.9

B54	13.1	5.9	16.3	5.9	7.2
B55	15.2	23.1	17.1	5.8	23.3
B56	17.1	26.9	32.2	17.9	27.2
B59	24.5	28.2	33.9	18.8	28.3
B6	24.8	25.9	25.9	25.9	24.3
B60	25.9	51.8	62.1	34.5	46.5
B60b	16.5	31.4	37.7	11.0	27.3
B61	5.4	16.9	15.2	8.5	15.0
B62	21.9	8.8	8.8	8.8	8.9
B62d	16.8	14.4	14.4	14.4	15.1
B62f	13.8	10.5	10.5	10.5	19.5
B62h	19.0	21.5	21.5	30.5	27.9
B63a	16.8	36.7	101.0	25.7	38.0
B63d	15.4	27.9	76.6	19.5	24.9
B64	13.8	25.9	71.2	51.8	26.8
B66a	26.6	15.4	15.4	15.4	15.7
B67	19.6	32.9	32.9	32.9	30.3
B68a	29.0	34.2	33.7	33.7	31.4
B68b	21.7	38.6	38.6	38.6	37.2
B68c	15.3	21.7	21.7	21.7	23.3
B68d	12.0	16.5	16.5	16.5	18.2
B68e	14.8	12.7	6.0	50.4	22.8
B68f	11.9	11.6	5.9	39.7	19.9
B69	6.5	24.5	24.5	24.5	22.6

Table 30. Adjusted Neuse River Mainstem Basin HEC-HMS Model Select Routing Method Slope Parameter for Calibration Events and Final Selected Value

Reach	Slope (ft/ft)				
	Initial	Matthew-Calibrated	Florence-Calibrated	2019-Calibrated	Final
B32R	0.001	0.00099	0.0005	0.00099	0.001
B56R	0.0003	0.0002	0.0003	0.0003	0.0002
B35R	0.0009	0.0007	0.00088	0.00088	0.0007
B30R	0.0004	0.0006	0.0004	0.0004	0.0006
B46R	0.0003	0.0001	0.0003	0.0003	0.0001
B63aR	0.0001	0.00015	0.00011	0.00011	0.0002
B63cR	0.0001	0.00015	0.00011	0.00011	0.0002
B64R	0.0002	0.00015	0.00015	0.0001	0.0002

Table 31. Adjusted Neuse River Mainstem Basin HEC-HMS Model Select Routing Method Channel Manning's N Parameter for Calibration Events and Final Selected Value

Reach	Channel Manning's N				
	Initial	Matthew-Calibrated	Florence-Calibrated	2019-Calibrated	Final
B49R	0.05	0.035	0.035	0.1	0.035
B25R	0.05	0.035	0.035	0.08	0.035
B50R	0.05	0.035	0.035	0.1	0.035
B52R	0.1	0.1	0.04	0.12	0.1
B54R	0.1	0.1	0.04	0.12	0.1
B53R	0.04	0.05	0.035	0.05	0.05
B32R	0.04	0.04	0.035	0.035	0.04
B28aR	0.04	0.04	0.035	0.035	0.04
B55R	0.04	0.04	0.04	0.035	0.04
B56R	0.04	0.06	0.04	0.04	0.06
B35R	0.035	0.035	0.035	0.04	0.035
B30R	0.035	0.035	0.035	0.04	0.035
B46R	0.05	0.1	0.05	0.05	0.1
B59aR	0.04	0.04	0.04	0.05	0.04
B59R	0.04	0.1	0.1	0.08	0.1
B60aR	0.04	0.04	0.08	0.04	0.04
B60R	0.04	0.08	0.1	0.06	0.08
B29aR	0.04	0.04	0.07	0.04	0.04
B43R	0.04	0.04	0.08	0.05	0.04
B47R	0.04	0.04	0.04	0.08	0.04
B63aR	0.07	0.07	0.1	0.03	0.07
B63cR	0.07	0.07	0.1	0.03	0.07

Table 32. Adjusted Neuse River Mainstem Basin HEC-HMS Model Select Routing Method Left Overbank Manning's N Parameter for Calibration Events and Final Selected Value

Reach	Left Manning's N				
	Initial	Matthew-Calibrated	Florence-Calibrated	2019-Calibrated	Final
B55R	0.18	0.25	0.18	0.25	0.25
B56R	0.12	0.25	0.12	0.12	0.25
B35R	0.1	0.25	0.1	0.1	0.25
B30R	0.1	0.25	0.1	0.1	0.25
B46R	0.2	0.25	0.2	0.2	0.25

Table 33. Adjusted Neuse River Mainstem Basin HEC-HMS Model Select Routing Method Right Overbank Manning's N Parameter for Calibration Events and Final Selected Value

Reach	Right Manning's N				
	Initial	Matthew-Calibrated	Florence-Calibrated	2019-Calibrated	Final
B55R	0.18	0.25	0.18	0.25	0.25
B56R	0.12	0.25	0.12	0.12	0.25
B35R	0.1	0.25	0.1	0.1	0.25
B30R	0.1	0.25	0.1	0.1	0.25
B46R	0.2	0.25	0.2	0.2	0.25

The Crabtree Creek basin HEC-HMS model underwent calibration to two historic events, Tropical Storm Alberto in June 2006 and Hurricane Matthew in October 2016.

The calibration to the Tropical Storm Alberto event was limited in scope. Tropical Storm Alberto produced the highest recorded peak streamflow at the USGS Crabtree Creek at Ebenezer Church Rd near Raleigh, NC gage (0208726005), 3rd highest at the USGS Crabtree Creek at HWY 70 at Raleigh, NC gage (02087275), and 4th highest at the USGS Crabtree Creek at US 1 at Raleigh, NC gage (02087324). The June 2006 event was simulated using the Gage Weights method in the meteorological model. The following precipitation gages were used in this analysis: USGS 0208732534 Pigeon House Cr at Cameron Village at Raleigh, NC, USGS 02087359 Walnut Creek at Sunnybrook Drive nr Raleigh, NC, USGS 02087182 Falls Lake above Dam nr Falls, NC, USGS 0208732885 Marsh Creek near New Hope, NC, and KRDU Raleigh-Durham International Airport. Total precipitation amounts for the event ranged from 5.5 inches to 7.8 inches. Total rainfall duration was approximately 12 hours.

Event calibration was performed at three gage locations, (1) USGS 0208726005, (2) USGS 02087275, and (3) 02087324. Calibration was focused on matching observed peak flow recorded at these sites. Summarized results of this calibration are listed in Table 34.

Table 34. Summarized Results of Crabtree Creek HMS Tropical Storm Calibration

Gage ID	Gage Location	Observed Flow (cfs)	Computed Flow (cfs)
208726005	Crabtree Creek at Ebenezer Church Rd near Raleigh, NC	8120.4	7690.5
2087275	Crabtree Creek at HWY 70 at Raleigh, NC	8650	11216.3
2087324	Crabtree Creek at US 1 at Raleigh, NC	8173.9	13564.4

Calibration to the Hurricane Matthew event was conducted in a similar way. The gage weights meteorological method was also used for this event. Several more precipitation gages were included due to better coverage of collected data. In addition to the gages used for the Tropical Storm Alberto calibration, the following gages were supplemented: USGS 355020078465645 Raingage at Lake Crabtree Co. Park Nr Morrisville, USGS 02087275 Crabtree Creek at Hwy 70 At Raleigh, NC, USGS 02087322 Crabtree Cr At Old Wake Forest Rd At Raleigh, NC, USGS 355856078492945 Raingage at Ltl Lick Cr at NC Hwy 98 Oak Grove, NC, USGS 0208735012 Rocky Branch Below Pullen Road at

Raleigh, NC, and USGS 02087580 Swift Creek Near Apex, NC. Total precipitation amounts for the event ranged from 5.6 inches to 9.0 inches. Total rainfall duration was approximately 24 hours.

Event calibration was performed at three gage locations, (1) USGS 0208726005, (2) USGS 02087275, and (3) 02087324. Calibration was focused on matching observed peak flow recorded at these sites. Summarized results of this calibration are listed in Table 35.

Table 35. Summarized Results of Crabtree Creek HMS Hurricane Matthew Calibration

Gage ID	Gage Location	Observed Flow (cfs)	Computed Flow (cfs)	Flow Variance (%)	Observed Volume (ac-ft)	Computed Volume (ac-ft)	Volume Variance (%)
208726005	Crabtree Creek at Ebenezer Church Rd near Raleigh, NC	5740	5991	4.4	18692	18012	-3.6
2087275	Crabtree Creek at HWY 70 at Raleigh, NC	6350	9007	41.8	22819	23121	1.3
2087324	Crabtree Creek at US 1 at Raleigh, NC	9650	12419	28.7	27732	29844	7.6

There are no historical or current streamflow records for use in calibration of the Adkins Branch basin HEC-HMS model. For Hominy Swamp Creek basin, there are streamflow records available from two historical gage sites, USGS 02090512 Hominy Swamp at Phillips St at Wilson, NC and USGS 0209050750 Hominy Swamp at Forest Hills Road near Wilson, NC. Neither gage had a period of record adequate for calibration purposes. Therefore, no event calibration was carried out for the Hominy Swamp Creek basin HEC-HMS model. The Big Ditch basin had one historical USGS gage site, USGS 02088682 Big Ditch at Retha St at Goldsboro, NC, that recorded peak flow from 1951 to 1984; however, it was not utilized for calibration due to lack of calibration rainfall data and the large span of time between gage records and existing conditions.

5.1.4 Calibration/Validation Results And Discussion

The primary means of calibration were through subbasin parameter adjustments. Adjustments were made with respect to simulating both the peak flow and volume of event hydrographs to best fit observed gage data. This required balancing flow and volume throughout the basin. Calibration was mostly successful with a few exceptions. It was determined that the September 2019 calibration event did not provide adequate rainfall coverage in the upper basin, above Falls Lake. Observed gage flows were too low to simulate accurately due to lack of rainfall-producing runoff and the presence of some flow regulation by reservoirs above Falls Lake. Therefore, calibration within this region was weighted more towards the larger Hurricane events.

A phenomenon that has historically occurred was also seen during modeling, in that significant flood hydrograph attenuation appeared to be taking place within the reach of the Neuse River mainstem between the USGS Goldsboro and Kinston gages. This disparity was quantified by the peak flow at Kinston being substantially less than the peak upstream at Goldsboro. For reference, the drainage area at the Kinston gage is about 300 square miles more than at Goldsboro, yet during Hurricane Matthew and Florence, observed flow at Kinston was only roughly 75% of the record peak at Goldsboro. USGS has suggested this reduction in flow between the two gage is likely indicative of storage within the intervening drainage basin (analogous to a detention pound) (USGS, 2016).

Within the Neuse River mainstem basin HEC-HMS model, the highly urban Crabtree Creek and Walnut Creek watersheds were unable to be adequately calibrated within the Hurricane Florence simulation. Attempts to match observed peak flow or volume resulted in unreasonable subbasin parameter values. This occurrence was most likely due to the relatively minor runoff response across these two subbasins, which were not significantly impacted by the precipitation footprint of Hurricane Florence. Furthermore, streamgage records near the outlets of Crabtree Creek and Walnut Creek suggested flow regulation that appeared more prominent during average flow conditions. As such, both outlets of these watersheds were simulated with a sink element within HEC-HMS and observed flow was set to their respective USGS streamflow gages. This method was chosen to reduce uncertainty in final calibrated parameters by not including the Hurricane Florence-specific values for these two subbasins. While this resulted in one less calibration event for these particular subbasins, the Crabtree Creek HEC-HMS model was assumed to be more appropriate for plan formulation within its particular watershed. Notably, the other calibration and validation events were able to better replicate observed data within reasonable subbasin parameter values. This issue was not unexpected given the rough approximation of these complex watersheds as a single subbasin in the Neuse River mainstem basin model. High Nash-Sutcliffe values seen in the figures above were representative of well-calibrated models for the calibration events.

Previous USACECWMS (daily operations), Modeling, Mapping and Consequences (MMC) Probable Maximum Flood analysis (PMF), and State efforts for HEC-HMS calibration had similar technical issues with successfully calibrating and validating flow to the Crabtree Creek, Goldsboro, Kinston, and Hookerton USGS gages.

The April 2017 validation event included additional rainfall that occurred roughly 10 days following the main event and the model’s inability to accurately simulate this secondary event resulted in a lowered Nash-Sutcliffe value. Validation of the model was done using weighted average parameters from calibration for the transform and losses parameters with emphasis placed on the 2016 Hurricane Matthew event calibration due to its comprehensive, basinwide impact footprint and the choice for scaling of the design events, as discussed below in Section 5.1.5. Some minor adjustments to the final routing reach parameters were also made based on a Hurricane Matthew-weighted average calibration. A summary of HEC-HMS calibration and validation results are listed in Table 36 through Table 39.

Table 36. Summarized Results of HMS Hurricane Matthew Calibration

Gage ID	Gage Location	Observed Flow (cfs)	Computed Flow (cfs)	Flow Variance (%)	Observed Volume (ac-ft)	Computed Volume (ac-ft)	Volume Variance (%)
2085000	ENO RIVER AT HILLSBOROUGH, NC	4,620	4,696	1.6	7,705	6,843	-11.2
2085070	ENO RIVER NEAR DURHAM, NC	8,220	8,243	0.3	16,479	13,990	-15.1
208521324	LITTLE RIVER AT SR1461 NEAR ORANGE FACTORY, NC	6,310	6,203	-1.7	11,562	10,669	-7.7
208524975	LITTLE R BL LITTLE R TRIB AT FAIRNTOSH, NC	7,590	7,172	-5.5	12,549	12,388	-1.3
2085500	FLAT RIVER AT BAHAMA, NC	13,700	13,976	2.0	23,712	22,592	-4.7
2086500	FLAT RIVER AT DAM NEAR BAHAMA, NC	12,400	14,412	16.2	7347.4*	23,770	--
2087324	CRABTREE CREEK AT US 1 AT RALEIGH, NC	9,650	9,930	2.9	28,993	31,353	8.1
2087359	WALNUT CREEK AT SUNNYBROOK DRIVE NR RALEIGH, NC	5,960	6,393	7.3	8,671	8,838	1.9
208758850	SWIFT CREEK NEAR MCCULLARS CROSSROADS, NC	7,060	6,807	-3.6	9,663	11,130	15.2

2087500	NEUSE RIVER NEAR CLAYTON, NC	20,200	21,156	4.7	95,978	103,550	7.9
2088000	MIDDLE CREEK NEAR CLAYTON, NC	20,300	17,127	-15.6	28,400	28,801	1.4
2088383	LITTLE RIVER NEAR ZEBULON, NC	9,370	7,687	-18.0	25,252	25,577	1.3
2088500	LITTLE RIVER NEAR PRINCETON, NC	9,960	10,941	9.8	75,186	75,679	0.7
2089000	NEUSE RIVER NEAR GOLDSBORO, NC	53,400	49,052	-8.1	564,809	594,332	5.2
2089500	NEUSE RIVER AT KINSTON, NC	38,200	37,350	-2.2	592,224	615,403	3.9
2091000	NAHUNTA SWAMP NEAR SHINE, NC	13,600	13,328	-2.0	36,629	32,945	-10.1
2091500	CONTENTNE A CREEK AT HOOKERTON, NC	27,600	25,403	-8.0	262,669	261,956	-0.3
2091814	NEUSE RIVER NEAR FORT BARNWELL, NC	49,400	48,923	-1.0	856,472	873,815	2.0
2092500	TRENT RIVER NEAR TRENTON, NC	10,700	9,848	-8.0	67,263	75,817	12.7

* Missing gage records

Table 37. Summarized Results of HMS Hurricane Florence Calibration

Gage ID	Gage Location	Observed Flow (cfs)	Computed Flow (cfs)	Flow Variance (%)	Observed Volume (ac-ft)	Computed Volume (ac-ft)	Volume Variance (%)
2085000	ENO RIVER AT HILLSBOROUGH , NC	2,890	2,988	3.4	5,448	4,660	-14.5
2085070	ENO RIVER NEAR DURHAM, NC	13,700	13,202	-3.6	15,825	15,551	-1.7
208521324	LITTLE RIVER AT SR1461 NEAR ORANGE FACTORY, NC	8,550	8,524	-0.3	11,219	14,356	28.0
208524975	LITTLE R BL LITTLE R TRIB AT FAIRNTOSH, NC	13,600	14,298	5.1	12,171	19,435	59.7
2085500	FLAT RIVER AT BAHAMA, NC	14,600	14,868	1.8	28,696	27,093	-5.6
2086500	FLAT RIVER AT DAM NEAR BAHAMA, NC	15,000	16,736	11.6	23,007	30,341	31.9
2087324	CRABTREE CREEK AT US 1 AT RALEIGH, NC	2,630	--	--	21,100	--	--
2087359	WALNUT CREEK AT SUNNYBROOK DRIVE NR RALEIGH, NC	838	--	--	3,532	--	--
208758850	SWIFT CREEK NEAR MCCULLARS CROSSROADS, NC	639	624	-2.3	2,828	3,153	11.5
2087500	NEUSE RIVER NEAR CLAYTON, NC	6,810	7,043	3.4	50,498	52,457	3.9
2088000	MIDDLE CREEK NEAR CLAYTON, NC	1,510	1,806	19.6	10,189	9,978	-2.1

2088383	LITTLE RIVER NEAR ZEBULON, NC	1,290	1,210	-6.2	5,890	5,699	-3.2
2088500	LITTLE RIVER NEAR PRINCETON, NC	3,520	3,720	5.7	35,176	34,246	-2.6
2089000	NEUSE RIVER NEAR GOLDSBORO, NC	36,700	35,858	-2.3	455,049	473,024	4.0
2089500	NEUSE RIVER AT KINSTON, NC	30,500	29,932	-1.9	480,889	498,303	3.6
2091000	NAHUNTA SWAMP NEAR SHINE, NC	6,060	6,062	0.0	32,014	31,185	-2.6
2091500	CONTENTNEA CREEK AT HOOKERTON, NC	11,700	10,222	-12.6	151,312	147,130	-2.8
2091814	NEUSE RIVER NEAR FORT BARNWELL, NC	40,100	41,132	2.6	712,970	714,938	0.3
2092500	TRENT RIVER NEAR TRENTON, NC	67,700	42,835	-36.7	376,738	226,778	-39.8

-- gage not included in calibration

Table 38. Summarized Results of HMS September 2019 Calibration

Gage ID	Gage Location	Observed Flow (cfs)	Computed Flow (cfs)	Flow Variance (%)	Observed Volume (ac-ft)	Computed Volume (ac-ft)	Volume Variance (%)
2085000	ENO RIVER AT HILLSBOROUGH, NC	--	--	--	--	--	--
2085070	ENO RIVER NEAR DURHAM, NC	--	--	--	--	--	--
208521324	LITTLE RIVER AT SR1461 NEAR ORANGE FACTORY, NC	--	--	--	--	--	--
208524975	LITTLE R BL LITTLE R TRIB AT FAIRNTOSH, NC	--	--	--	--	--	--
2085500	FLAT RIVER AT BAHAMA, NC	--	--	--	--	--	--
2086500	FLAT RIVER AT DAM NEAR BAHAMA, NC	--	--	--	--	--	--
2087324	CRABTREE CREEK AT US 1 AT RALEIGH, NC	534	518	-3.1	1,841	1,473	-20.0
2087359	WALNUT CREEK AT SUNNYBROOK DRIVE NR RALEIGH, NC	285	298	4.6	791	553	-30.1
208758850	SWIFT CREEK NEAR MCCULLARS CROSSROADS, NC	81	80	-1.4	408	450	10.1
2087500	NEUSE RIVER NEAR CLAYTON, NC	1,680	1,718	2.3	14,181	13,809	-2.6
2088000	MIDDLE CREEK NEAR CLAYTON, NC	246	244	-1.0	1,492	1,349	-9.6

2088383	LITTLE RIVER NEAR ZEBULON, NC	48	83	73.6	574	355	-38.2
2088500	LITTLE RIVER NEAR PRINCETON, NC	1,910	1,881	-1.5	6,389	6,366	-0.4
2089000	NEUSE RIVER NEAR GOLDSBORO, NC	8,060	7,670	-4.8	79,854	85,731	7.4
2089500	NEUSE RIVER AT KINSTON, NC	6,760	7,231	7.0	112,295	119,718	6.6
2091000	NAHUNTA SWAMP NEAR SHINE, NC	1,300	1,467	12.9	6,748	7,226	7.1
2091500	CONTENTNEA CREEK AT HOOKERTON, NC	2,490	2,527	1.5	32,595	34,718	6.5
2091814	NEUSE RIVER NEAR FORT BARNWELL, NC	10,900	11,170	2.5	198,945	208,078	4.6
2092500	TRENT RIVER NEAR TRENTON, NC	2,410	2,353	-2.4	21,953	23,286	6.1

-- gage not included in calibration

Table 39. Summarized Results of HMS April 2017 Validation

Gage ID	Gage Location	Observed Flow (cfs)	Computed Flow (cfs)	Flow Variance (%)	Observed Volume (ac-ft)	Computed Volume (ac-ft)	Volume Variance (%)
2085000	ENO RIVER AT HILLSBOROUGH , NC	4,320	4,841	12.1	10,749	17,085	59.0
2085070	ENO RIVER NEAR DURHAM, NC	10,300	10,792	4.8	26,700	39,643	48.5
208521324	LITTLE RIVER AT SR1461 NEAR ORANGE FACTORY, NC	7,200	6,102	-15.3	16,144	21,960	36.0
208524975	LITTLE R BL LITTLE R TRIB AT FAIRNTOSH, NC	10,100	7,825	-22.5	20,713	28,752	38.8
2085500	FLAT RIVER AT BAHAMA, NC	11,900	8,808	-26.0	31,914	38,594	20.9
2086500	FLAT RIVER AT DAM NEAR BAHAMA, NC	12,600	9,530	-24.4	24,717	42,767	73.0
2087324	CRABTREE CREEK AT US 1 AT RALEIGH, NC	7,440	9,644	29.6	37,130	62,809	69.2
2087359	WALNUT CREEK AT SUNNYBROOK DRIVE NR RALEIGH, NC	4,000	3,448	-13.8	8,171	8,703	6.5
208758850	SWIFT CREEK NEAR MCCULLARS CROSSROADS, NC	3,320	2,780	-16.3	7,760	10,583	36.4
2087500	NEUSE RIVER NEAR CLAYTON, NC	18,200	16,777	-7.8	227,094	136,930	-39.7
2088000	MIDDLE CREEK NEAR CLAYTON, NC	4,820	6,110	26.8	15,254	20,023	31.3
2088383	LITTLE RIVER NEAR ZEBULON, NC	6,050	3,562	-41.1	13,903	16,907	21.6
2088500	LITTLE RIVER NEAR PRINCETON, NC	7,080	6,757	-4.6	53,160	60,267	13.4

2089000	NEUSE RIVER NEAR GOLDSBORO, NC	22,000	17,585	-20.1	373,630	398,715	6.7
2089500	NEUSE RIVER AT KINSTON, NC	19,600	15,844	-19.2	425,394	444,184	4.4
2091000	NAHUNTA SWAMP NEAR SHINE, NC	2,920	3,474	19.0	16,190	18,607	14.9
2091500	CONTENTNEA CREEK AT HOOKERTON, NC	16,500	9,568	-42.0	170,519	161,085	-5.5
2091814	NEUSE RIVER NEAR FORT BARNWELL, NC	27,700	22,000	-20.6	651,010	671,439	3.1
2092500	TRENT RIVER NEAR TRENTON, NC	2,390	2,492	4.3	23,654	40,162	69.8

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 40, Table 41, and Table 42 below. For the Hurricane Matthew calibration event, the computed hydrograph time of peak at Goldsboro (02089000) and Kinston (02089500) gages were off by >12 hours when compared to observed data. 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. Based on a closer match between computed and observed time of peak for gages elsewhere along the Neuse River mainstem, the larger discrepancy between Goldsboro and Kinston is unlikely to lead to significant concerns related to plan formulation. For the Hurricane Florence calibration event, the computed hydrograph time of peak at the Fort Barnwell gage (02091814) missed the observed peak by roughly 2 days. It was noted that the observed hydrograph experienced two separate peaks, an initial peak occurring on mid-day September 19 and a second, slightly higher peak on the midnight of September 22. Based on its close proximity to the downstream boundary of the study model and a closer time of peak match for gages along the Neuse River mainstem at Goldsboro and Kinston, as well as at the Contentnea Creek at Hookerton gage (02091500), the larger difference at Fort Barnwell was not considered a significant concern to plan formulation. For the September 2019 calibration event, the potential effects of floodplain storage and attenuation between Goldsboro and Kinston was similar to the Hurricane Matthew event.

Table 40. Time of Peak Comparison - Neuse River Mainstem HEC-HMS Model Computed vs. Observed for Hurricane Matthew Calibration Event

Gage ID	Gage Location	Time of Peak		Diff (hr)
		Computed	Observed	
2087500	NEUSE RIVER NEAR CLAYTON, NC	10/9/2016 11:00	10/9/2016 15:45	-4.75
2089000	NEUSE RIVER NEAR GOLDSBORO, NC	10/12/2016 19:15	10/12/2016 3:45	15.5
2089500	NEUSE RIVER AT KINSTON, NC	10/15/2016 3:15	10/14/2016 8:45	18.5
2091814	NEUSE RIVER NEAR FORT BARNWELL, NC	10/15/2016 8:00	10/15/2016 15:00	-7
2087324	CRABTREE CREEK AT US 1 AT RALEIGH, NC	10/8/2016 20:30	10/8/2016 21:00	-0.5
2087359	WALNUT CREEK AT SUNNYBROOK DRIVE NR RALEIGH, NC	10/8/2016 19:30	10/8/2016 19:45	-0.25
208758850	SWIFT CREEK NEAR MCCULLARS CROSSROADS, NC	10/8/2016 21:00	10/8/2016 21:30	-0.5
2088000	MIDDLE CREEK NEAR CLAYTON, NC	10/9/2016 2:15	10/9/2016 4:30	-2.25
2088500	LITTLE RIVER NEAR PRINCETON, NC	10/10/2016 12:45	10/10/2016 15:45	-3
2091500	CONTENTNEA CREEK AT HOOKERTON, NC	10/11/2016 9:15	10/11/2016 1:00	8.25
2085000	ENO RIVER AT HILLSBOROUGH, NC	10/8/2016 20:15	10/8/2016 20:45	-0.5
2085070	ENO RIVER NEAR DURHAM, NC	10/8/2016 22:30	10/8/2016 21:15	1.25
208521324	LITTLE RIVER AT SR1461 NEAR ORANGE FACTORY, NC	10/8/2016 20:30	10/8/2016 20:00	0.5
2085500	FLAT RIVER AT BAHAMA, NC	10/8/2016 22:30	10/9/2016 1:00	-2.5

Table 41. Time of Peak Comparison - Neuse River Mainstem HEC-HMS Model Computed vs. Observed for Hurricane Florence Calibration Event

Gage ID	Gage Location	Time of Peak		Diff (hr)
		Computed	Observed	
2087500	NEUSE RIVER NEAR CLAYTON, NC	9/16/2018 7:30	9/16/2018 0:45	6.75
2089000	NEUSE RIVER NEAR GOLDSBORO, NC	9/18/2018 13:45	9/18/2018 15:30	-1.75
2089500	NEUSE RIVER AT KINSTON, NC	9/21/2018 16:45	9/21/2018 14:15	2.5
2091814	NEUSE RIVER NEAR FORT BARNWELL, NC	9/20/2018 20:45	9/22/2018 23:30	-50.75
208758850	SWIFT CREEK NEAR MCCULLARS CROSSROADS, NC	9/17/2018 12:45	9/17/2018 11:15	1.5
2088000	MIDDLE CREEK NEAR CLAYTON, NC	9/17/2018 12:30	9/17/2018 9:45	2.75
2088500	LITTLE RIVER NEAR PRINCETON, NC	9/18/2018 4:15	9/18/2018 8:30	-4.25
2091500	CONTENTNEA CREEK AT HOOKERTON, NC	9/19/2018 2:15	9/19/2018 10:30	-8.25
2085000	ENO RIVER AT HILLSBOROUGH, NC	9/17/2018 13:45	9/17/2018 14:45	-1
2085070	ENO RIVER NEAR DURHAM, NC	9/17/2018 10:45	9/17/2018 10:15	0.5
208521324	LITTLE RIVER AT SR1461 NEAR ORANGE FACTORY, NC	9/17/2018 13:15	9/17/2018 10:00	3.25
2085500	FLAT RIVER AT BAHAMA, NC	9/17/2018 16:00	9/17/2018 18:45	-2.75

Table 42. Time of Peak Comparison - Neuse River Mainstem HEC-HMS Model Computed vs. Observed for September 2019 Calibration Event

Gage ID	Gage Location	Time of Peak		Diff (hr)
		Computed	Observed	
2087500	NEUSE RIVER NEAR CLAYTON, NC	9/6/2019 12:15	9/6/2019 8:45	3.5
2089000	NEUSE RIVER NEAR GOLDSBORO, NC	9/8/2019 0:45	9/8/2019 9:30	-8.75
2089500	NEUSE RIVER AT KINSTON, NC	9/10/2019 8:15	9/11/2019 4:15	-20
2091814	NEUSE RIVER NEAR FORT BARNWELL, NC	9/11/2019 12:45	9/11/2019 17:45	-5
2087324	CRABTREE CREEK AT US 1 AT RALEIGH, NC	9/6/2019 4:45	9/5/2019 23:15	5.5
2087359	WALNUT CREEK AT SUNNYBROOK DRIVE NR RALEIGH, NC	9/6/2019 1:00	9/6/2019 2:45	-1.75
208758850	SWIFT CREEK NEAR MCCULLARS CROSSROADS, NC	9/6/2019 10:00	9/6/2019 12:00	-2
2088000	MIDDLE CREEK NEAR CLAYTON, NC	9/6/2019 15:00	9/6/2019 15:30	-0.5
2088500	LITTLE RIVER NEAR PRINCETON, NC	9/6/2019 12:15	9/6/2019 8:45	3.5
2091500	CONTENTNEA CREEK AT HOOKERTON, NC	9/10/2019 0:30	9/9/2019 16:00	8.5

5.1.5 Design Rainfall

A design storm was used in the Neuse 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 was 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.

Due to the large size of the Neuse 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. Typical ARF as outlined in TP-40 and in HEC-HMS were not applicable due to the basin size, and a site-specific analysis was desired to accurately represent the design storms. There has been limited research related to Neuse River basin-specific aerial reduction factors that include large storm area centers, and new ARF development was not included in this basin study scope. Through coordination with Probable Maximum Precipitation development expertise at USACE NWP, a ASCE reference involving aerial reduction factors for two eastern regions, one within North Carolina (Allen and DeGaetano, 2005), was determined appropriate for use in this basin study. This reference specifically addressed the need for ARF in watershed areas larger than 1,000 square kilometers, as well as overall TP-40 updating as it was originally developed in the 1950s. Table 43 shows the representative basin-wide design rainfall values, before and after applying the ARF, used for a 48-hour design storm.

Table 43. Atlas 14 Before and After Aerial Reduction Factors

AEP	Atlas 14	Atlas 14 w/ ARF
0.5	4.55	3.50
0.2	6.20	4.59
0.1	7.48	5.42
0.04	9.34	6.63
0.02	10.90	7.52
0.01	12.70	8.57
0.005	14.60	9.78
0.002	17.60	11.79

Spatial distribution of the design storm was based on a realistic rainfall intensity across the basin. Due to the nature of the meteorology in the Neuse River basin, rain has generally occurred over much of the basin at once during historically significant events (Hurricanes Matthew & Florence), and not as isolated storm centers over one given headwater watershed. The Neuse River basin is influenced by strong areas of low pressure moving in from the Atlantic Ocean. These storms often bring with them high levels of moisture from subtropical sources and lead to widespread heavy rainfall that may last one or more days. Therefore, rainfall peak intensities were weighted by statistical normalization in order to avoid being overly conservative. Design storm precipitation values per subbasin were normalized to the recent historic flood events, Hurricane Matthew in 2016 and Hurricane Florence in 2018. NWS gridded precipitation for both event durations was processed in HEC-MetVue and subbasin-averaged rainfall totals were generated. Subbasin totals were then proportioned against the basin-wide total and a weighting factor was assigned to each subbasin to ensure adequate storm coverage. Due to the widespread flooding footprint of Hurricane Matthew throughout Neuse River basin and the stalled storm path of Hurricane Florence that predominately impacted the lower portions of the basin, normalized factors from Hurricane Matthew were chosen to best represent the spatial distribution of a design storm. Although outside the scope of this study, more research is likely warranted into development of a critical storm centering approach that includes varying storm duration, location, orientation, and temporal rainfall distribution.

The design storm temporal distribution was based on historic rainfall in the basin in order to be consistent with spatial design storm placement. Hourly subbasin hyetographs were developed in HEC-MetVue based on Hurricane Matthew gridded precipitation over a roughly 2-day duration. Each subbasin was then assigned a specific precipitation time-series gage in HEC-HMS. The total rainfall depth per subbasin for Hurricane Matthew was ratioed based on each design storm frequency total by using the Total Depth factor within the Specified Hyetograph, meteorologic model in HEC-HMS. An example of this subbasin-specific temporal distribution is shown in Figure 87.

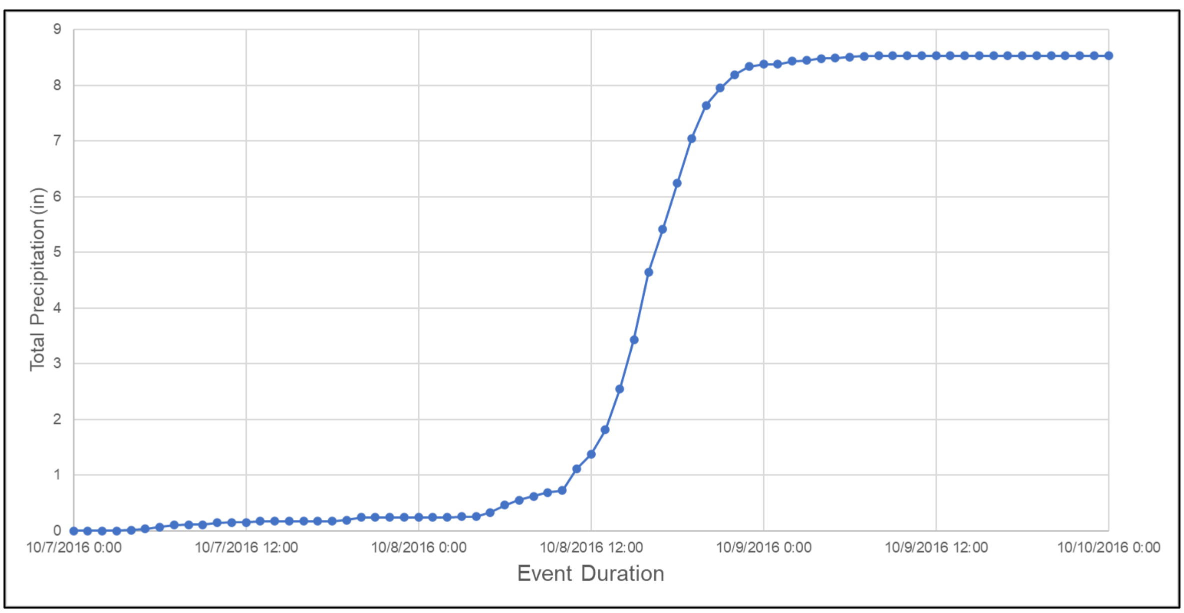

Figure 87. Example of Subbasin Temporal Distribution for Design Storms

The synthetic Frequency Storm meteorological method was used to present the suite of design storms for the Crabtree Creek, Hominy Swamp Creek, Big Ditch, and Adkins Branch basin HEC-HMS models. The nested precipitation depths involved in this method were determined applicable in assessing local flooding problems and opportunities. Furthermore, the small basin size and lack of calibration data for Hominy Swamp Creek, Big Ditch, and Adkins Branch made this method more appropriate than the user-specified hyetograph utilized for the Neuse River mainstem basin model. NOAA Atlas 14 Annual Maximum Series point precipitation values was used to develop design storms for the following annual exceedance probabilities: 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, 0.005, and 0.002. ARFs were not utilized for the Crabtree Creek, Hominy Swamp Creek, Big Ditch, or Adkins Branch HEC-HMS models. A 1-day storm duration was chosen for the four models, and for the Crabtree Creek, Hominy Swamp Creek, and Adkins Branch models, an intensity duration of 15 minutes was used. For the Big Ditch HEC-HMS model, an intensity duration of 5 minutes was chosen due to the small watershed size and highly urbanized landcover. Atlas 14 point precipitation frequency depths are listed in Table 44 through Table 47

Table 44. Crabtree Creek Basin HEC-HMS Atlas 14 AMS-Based Precipitation Frequency Estimates

AEP	Atlas 14
0.5	3.16
0.2	4.21
0.1	4.93
0.04	5.88
0.02	6.61
0.01	7.35
0.005	8.11
0.002	9.15

Table 45. Hominy Swamp Creek Basin HEC-HMS Atlas 14 AMS-Based Precipitation Frequency Estimates

AEP	Atlas 14
0.5	3.24
0.2	4.44
0.1	5.38
0.04	6.76
0.02	7.95
0.01	9.29
0.005	10.8
0.002	13.1

Table 46. Adkins Branch Basin HEC-HMS Atlas 14 AMS-Based Precipitation Frequency Estimates

AEP	Atlas 14
0.5	3.58
0.2	4.91
0.1	5.96
0.04	7.51
0.02	8.83
0.01	10.3
0.005	12
0.002	14.6

Table 47. Big Ditch Basin HEC-HMS Atlas 14 AMS-Based Precipitation Frequency Estimates

AEP	Atlas 14
0.5	3.42
0.2	4.69
0.1	5.69
0.04	7.16
0.02	8.41
0.01	9.83
0.005	11.4
0.002	13.9

5.1.6 Frequency Simulation Results

Design storms were applied to the five existing hydrologic conditions HEC-HMS models. The full suite of design storm frequencies was run, and flow estimates were produced for the 0.5-, 0.2-, 0.1-, 0.04-, 0.02-, 0.01-, 0.005-, and 0.002-AEP events. Peak computed flows were compared to other data sources including regional regression equations and site-specific gage frequency analyses as shown in Figure 88 through Figure 98 and Table 48 through Table 58. Regional data was derived from regression models to the study (USGS Scientific Investigations Report 2014-5030) and computed via excel spreadsheet. Following this comparative exercise all HEC-HMS computed design flows were carried toward into hydraulic modeling and analysis.

Hominy Swamp Creek basin HEC-HMS computed flows were compared to USGS regression equations based on the study area being in the Coastal Plain region, as shown in Figure 88 and Table 48. The computed flows are mostly contained within the 95% prediction intervals, with only the 0.002-AEP plotting above the upper interval. Overall, computed flows were greater than regression-based flows. Upon closer inspection and review of the FEMA effective hydrology, regression equations utilized were based on a location within the Piedmont region. Hominy Swamp Creek is near the fall line and can be associated with either region depending on the source delineation. Therefore, computed flows were also compared to regression equations based on the Piedmont region, as shown in Figure 89 and Table 49. Overall, computed flow better fit the discharge trend produced by assuming hydrologic characteristics of the Piedmont region. Computed flows are slightly lower for more frequent design storms and slightly greater for the more significant events when compared to the regression line.

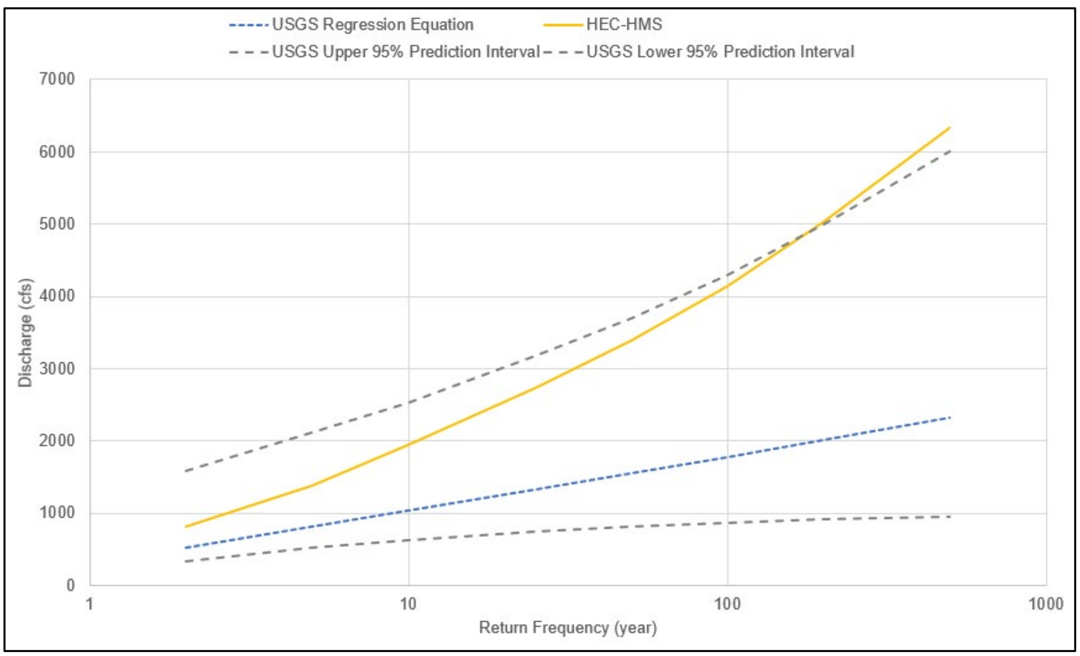

Figure 88. Hominy Swamp Creek Basin HEC-HMS Computed Flow vs. USGS Regression Equations (Coastal Plain Region) near Basin Outlet

Table 48. Hominy Swamp Creek Basin HEC-HMS Computed Flow vs. USGS Regression Equations (Coastal Plain Region) near Basin Outlet

AEP	HEC-HMS (cfs)	USGS Regression Equation (cfs)		
		Computed	Upper 95% PI	Lower 95% PI
0.5	815	516	1587	336
0.2	1374	813	2120	515
0.1	1937	1033	2527	633
0.04	2728	1324	3174	746
0.02	3391	1544	3695	814
0.01	4146	1773	4291	871
0.005	5033	2014	4997	912
0.002	6336	2323	5998	944

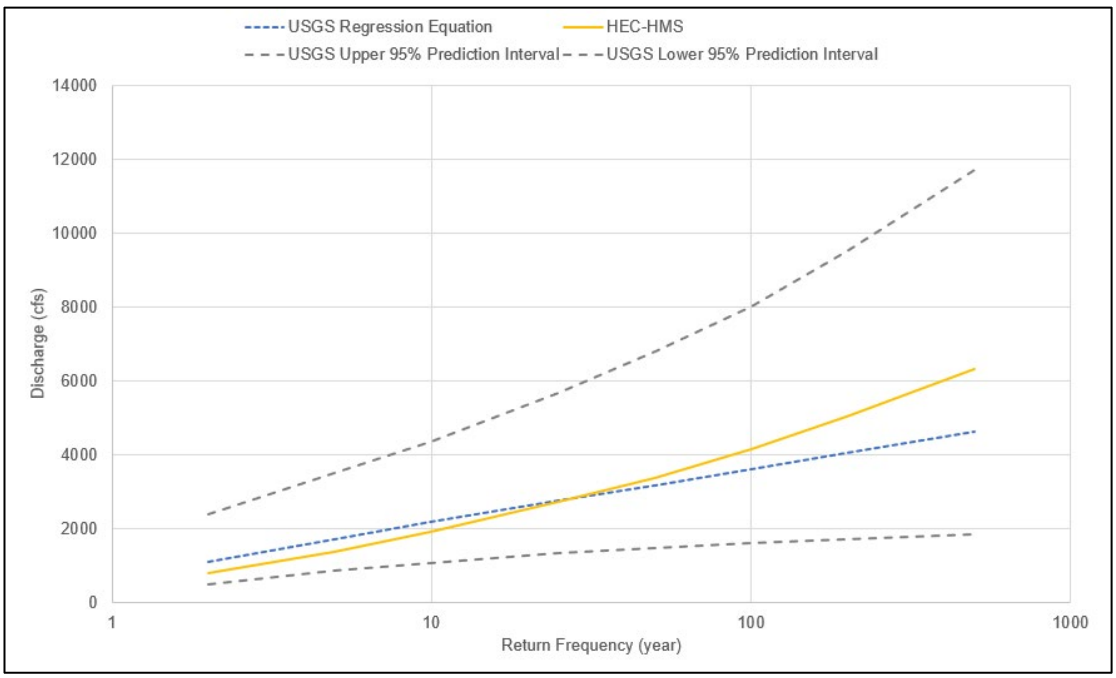

Figure 89. Hominy Swamp Creek Basin HEC-HMS Computed Flow vs. USGS Regression Equations (Piedmont Region) near Basin Outlet

Table 49. Hominy Swamp Creek Basin HEC-HMS Computed Flow vs. USGS Regression Equations (Piedmont Region) near Basin Outlet

AEP	HEC-HMS (cfs)	USGS Regression Equation (cfs)		
		Computed	Upper 95% PI	Lower 95% PI
0.5	815	1095	2381	504
0.2	1374	1733	3516	854
0.1	1937	2179	4353	1090
0.04	2728	2755	5685	1336
0.02	3391	3184	6783	1494
0.01	4146	3607	8008	1625
0.005	5033	4055	9493	1732
0.002	6336	4643	11702	1842

Adkins Branch basin HEC-HMS computed flows were compared to USGS regression equations based on the study area being in the Coastal Plain region, as shown in Figure 90 and Table 50. HEC-HMS computed flows were consistently larger than USGS regression equations throughout the full range of analyzed design events though were well within upper and lower prediction intervals. Review of the FEMA effective hydrology for Adkins Branch revealed the use of regression equations based on placement within the Piedmont region. Unlike Hominy Swamp Creek, Adkins Branch is well within the Coastal Plain region. It is uncertain if use of Piedmont region-based equations for the FEMA effective hydrology was a simple error or if there were other reasons. For comparison, the FEMA Effective 0.1-, 0.04-, 0.02-, 0.01-, 0.002-AEP flows using the Piedmont region regression equations have been included in the right-most column in Table 50.

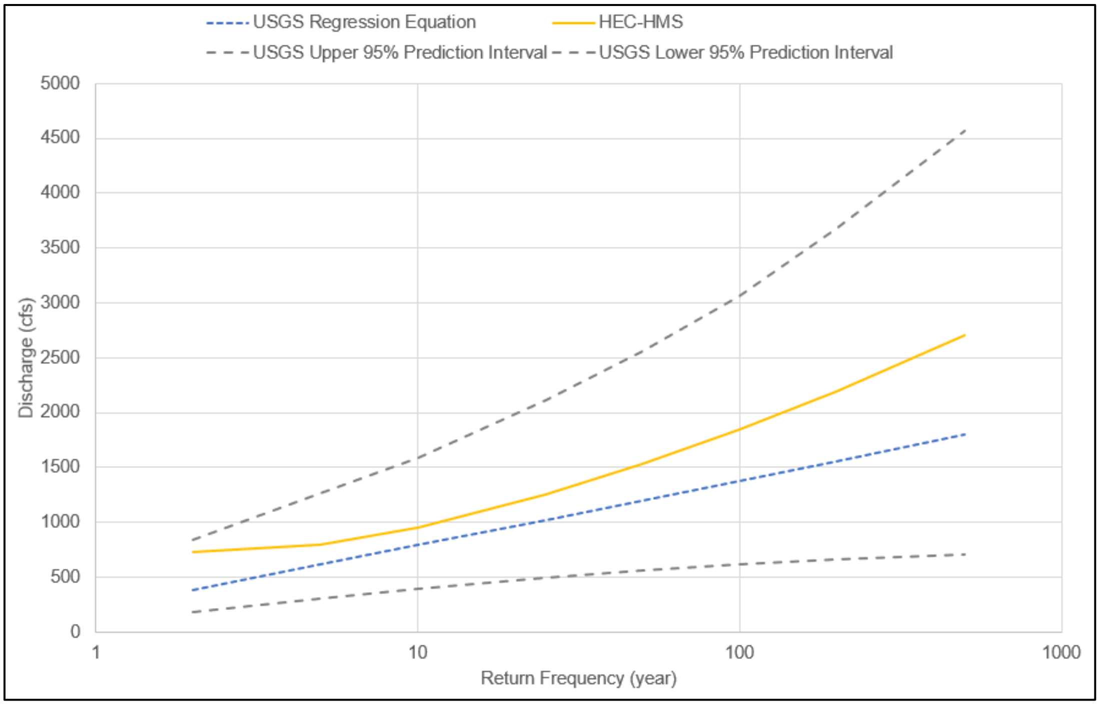

Figure 90. Adkins Branch Basin HEC-HMS Computed Flow vs. USGS Regression Equations near Basin Outlet

Table 50. Adkins Branch Basin HEC-HMS Computed Flow vs. USGS Regression Equations near Basin Outlet

AEP	HEC-HMS (cfs)	USGS Regression Equation (cfs) Computed (Coastal Region)	USGS Regression Equation (cfs) Upper 95% PI	USGS Regression Equation (cfs) Lower 95% PI	FEMA Effective (Piedmont Region)
0.5	725	387	845	336	
0.2	800	621	1265	515	
0.1	951	792	1590	633	1840
0.04	1257	1019	2113	746	2480
0.02	1531	1192	2553	814	2790
0.01	1845	1371	3062	871	3080
0.005	2189	1559	3672	912	
0.002	2707	1801	4568	944	3850

Big Ditch basin HEC-HMS computed flows were compared to USGS regression equations based on the study area being in the Coastal Plain region, as shown in Figure 91 and Table 51. Computed flows for all design storm AEPs were greater than regression-based flows but were contained with the upper and lower 95% prediction intervals. Review of the FEMA effective hydrology for Big Ditch revealed the use of regression equations that differed from the 2014 USGS versions. They were based on USGS Water-Resources Investigation Report 96-4084 (USGS, 1996). This older study did not provide regression equations that cover the suite of design storms and extrapolation was required beyond the 0.01-AEP event. An approximated ratio was calculated between computed flows and those produced by the USGS 96-4084 method. Result showed an average overestimation of regression-based flows by 1.06%.

A Bulletin 17C frequency analysis was conducted at the USGS Big Ditch at Retha St at Goldsboro, NC (02088682). This analysis was completed by standard methods, whereby a Pearson Type III distribution was fit to the logarithm of observed annual peak flow at the site. Although there had been considerable growth and land use change within the basin between the gage's period of record and existing conditions, a comparison was determined appropriate due to the overall lack of calibration data. A comparison of HEC-HMS computed flow and Bulletin 17C frequency analysis results are shown in Figure 92 and Table 52. HEC-HMS computed flows compared well to the frequency analysis, although there continues to be uncertainty related to changes in hydrologic conditions between the gage period of record and existing conditions in this study.

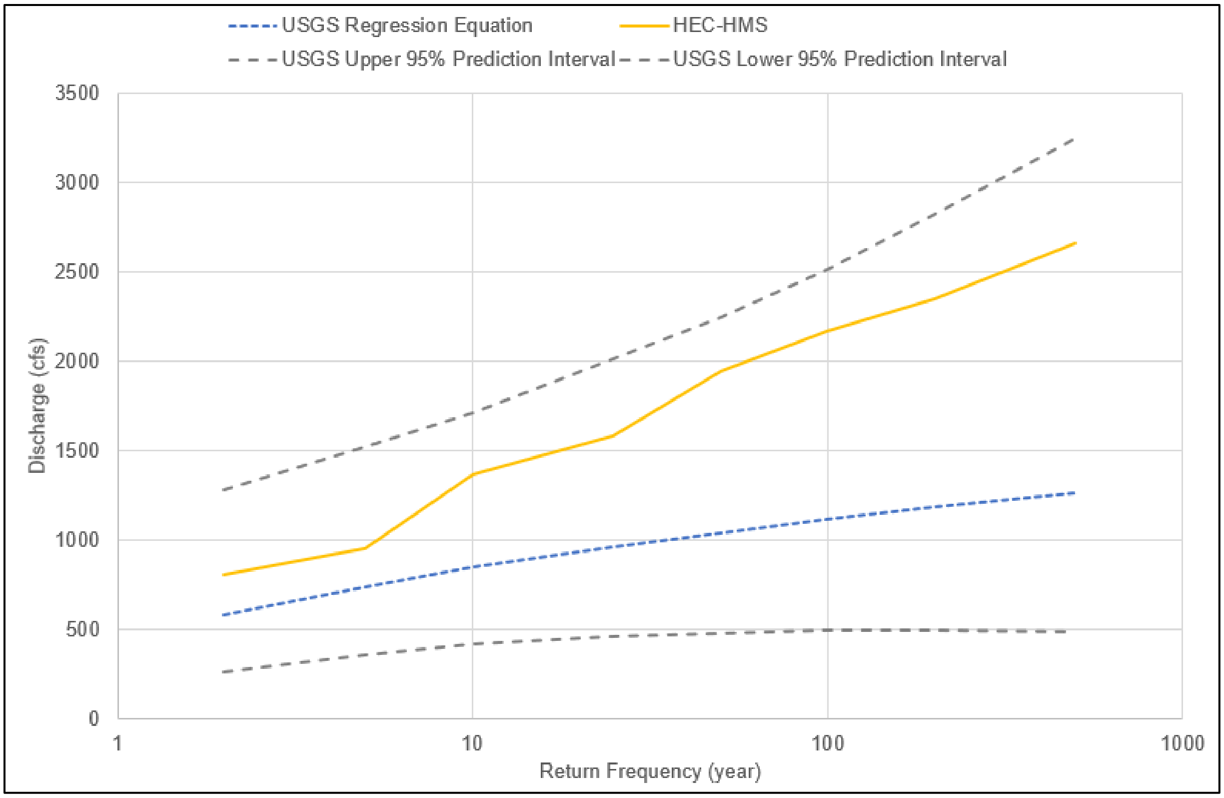

Figure 91. Big Ditch Basin HEC-HMS Computed Flow vs. USGS Regression Equations near Basin Outlet

Table 51. Big Ditch Basin HEC-HMS Computed Flow vs. USGS Regression Equations near Basin Outlet

AEP	HEC-HMS (cfs)	USGS Regression Equation (cfs)		
		Computed	Upper 95% PI	Lower 95% PI
0.5	813	581	1279	336
0.2	955	744	1527	515
0.1	1368	849	1716	633
0.04	1584	966	2019	746
0.02	1945	1042	2252	814
0.01	2169	1117	2518	871
0.005	2349	1188	2828	912
0.002	2665	1266	3247	944

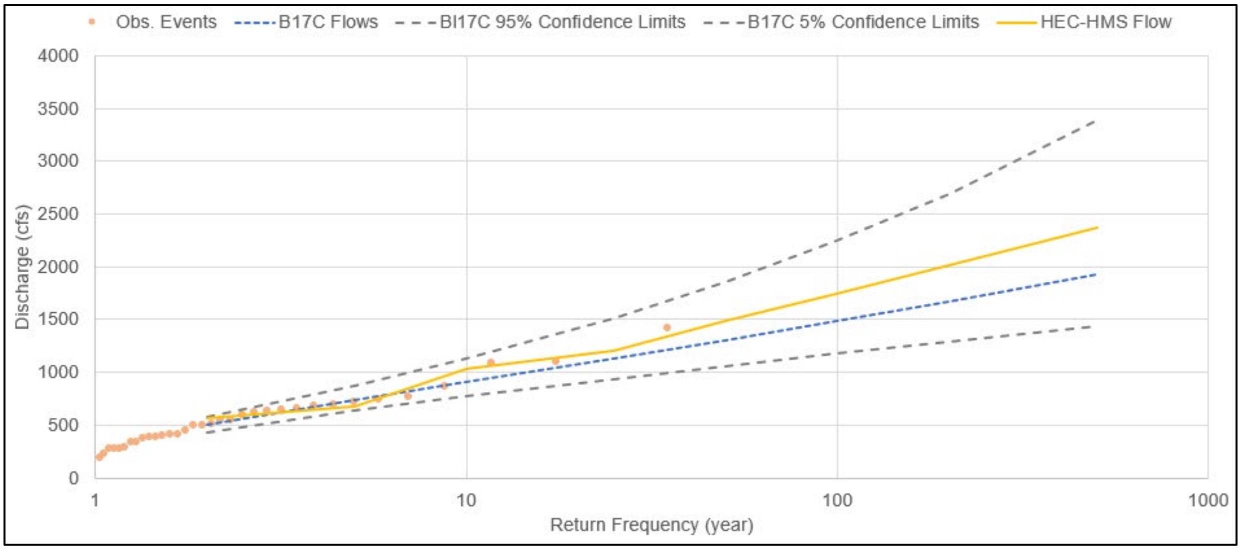

Figure 92. Big Ditch basin HEC-HMS flow comparison with historical Big Ditch at Retha Gage Bulletin 17C Frequency Analysis

Table 52. Big Ditch basin HEC-HMS flow comparison with historical Big Ditch at Retha Gage Bulletin 17C Frequency Analysis

AEP	HEC-HMS (cfs)	Bulletin 17C - USGS Retha Gage (cfs)		
		Computed	5% Confidence Limits	95% Confidence Limits
0.5	569	499	574	435
0.2	674	738	879	639
0.1	1028	907	1128	774
0.04	1199	1131	1511	940
0.02	1492	1306	1851	1060
0.01	1749	1486	2242	1176
0.005	2010	1674	2693	1288
0.002	2372	1934	3395	1433

A Bulletin 17C frequency analysis was conducted at the USGS Crabtree Creek at U.S Highway 1 at Raleigh, NC (02087324), for comparison to the Crabtree Creek basin HEC-HMS model design storms as shown in Figure 93 and Table 53. The plotted HEC-HMS flows closely match results from the frequency analysis. HEC-HMS-computed design storms more frequent than the 0.01-AEP were lower than the B17C curve and higher than for the less frequent 0.005- and 0.002-AEP events. Overall, HEC-HMS computed flow had an average variance of -5.8% compared to B17C results.

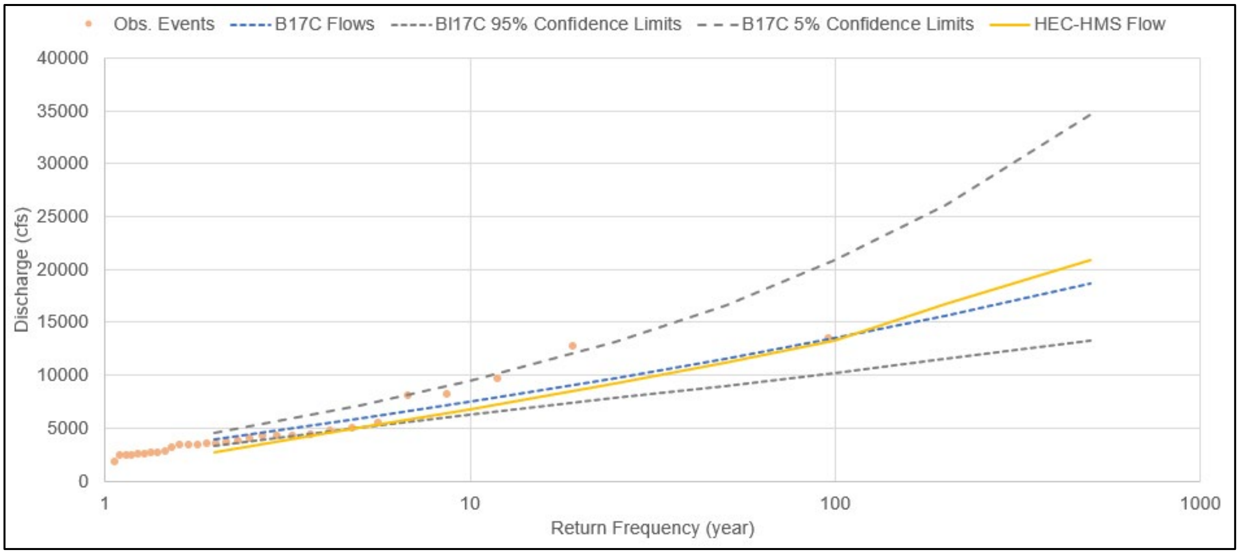

Figure 93. Crabtree Creek basin HEC-HMS flow comparison with USGS US-1 Gage Bulletin 17C Frequency Analysis

Table 53. Crabtree Creek basin HEC-HMS flow comparison with USGS US-1 Gage Bulletin 17C Frequency Analysis

AEP	HEC-HMS (cfs)	Bulletin 17C - USGS US-1 Gage		
		Computed	5% Confidence Limits	95% Confidence Limits
0.5	2760	3888	4547	3334
0.2	5046	5952	7194	5041
0.1	6809	7519	9460	6242
0.04	9200	9727	13143	7818
0.02	11144	11542	16629	9025
0.01	13235	13504	20875	10254
0.005	16656	15631	26042	11509
0.002	20865	18728	34622	13209

A series of Bulletin 17C frequency analyses were conducted for review of design storms in the Neuse River mainstem basin HEC-HMS model. Specific gage locations for this review were chosen that best represent the variety in design storm peak flow and volume throughout the large study area. A number of gages along the Neuse River mainstem are considered regulated by Fall Lake. Therefore, at these sites a period of record was established beginning in December 1983, when the volume of the reservoir reached elevation targets that allowed for normal operations. Additionally, station skew at the regulated sites was used for computing a generalized skew due to the alteration of natural flows by the Falls Lake flood risk management mission. Overall, the peak frequency flow rates simulated in the HEC-HMS model had a reasonable agreement with the Bulletin 17C frequency analyses. At the USGS Neuse River near Clayton gage, more frequent modeled AEP events were underestimated, and more severe events were slightly overestimated. Frequency results at the USGS Little River near Princeton gage showed a consistent overestimation of modeled AEP event, though fitting reasonably well within confidence limits. Inclusion of the recent historic events of Hurricane Matthew and Hurricane Florence in the frequency analysis appeared to impact the upper half of design storm AEPs. Modeled flows were in better agreement with frequency curves when one or both of these significant events were treated as high outliers, even though Bulletin 17C results did not explicitly identify these two events as such. Consequently, the peak flows associated with these significant events were included in all frequency analyses conducted as part of this study. Site-specific development of design storms would be better suited for a refined study area and would likely produce a closer match to gage frequency analyses. However, due to the Neuse River's large basin size and the intent in simulating a single basin-wide precipitation event, design storm frequency flows produced by the HEC-HMS model were considered acceptable. Comparison of HEC-HMS flow to Bulletin 17C gage frequency analysis at select sites is shown in Figure 94 through Figure 98 and Table 54 through Table 58.

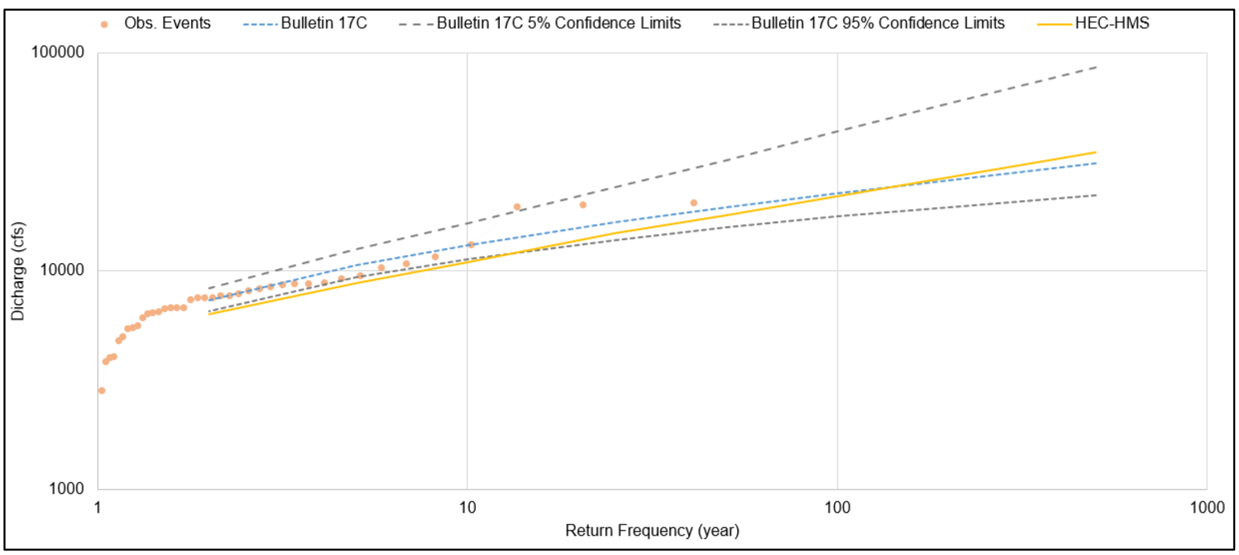

Figure 94. Neuse River Mainstem Basin HEC-HMS flow comparison with USGS Neuse River near Clayton Gage Bulletin 17C Frequency Analysis

Table 54. Neuse River Mainstem Basin HEC-HMS flow comparison with USGS Neuse River near Clayton Gage Bulletin 17C Frequency Analysis

AEP	HEC-HMS (cfs)	Bulletin 17C - USGS Clayton Gage		
		Computed	5% Confidence Limits	95% Confidence Limits
0.5	6333	7367	8327	6526
0.2	8781	10675	12615	9358
0.1	11010	13170	16625	11349
0.04	14930	16686	24125	13899
0.02	18083	19580	32276	15813
0.01	22026	22717	43465	17736
0.005	27002	26131	58795	19679
0.002	35162	31123	85413	22301

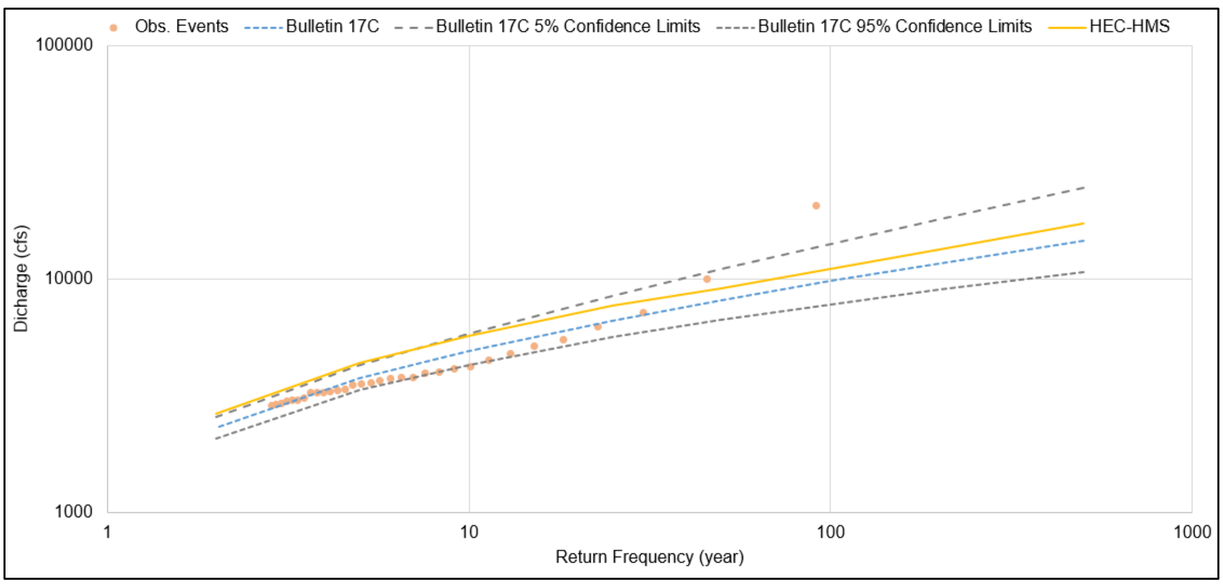

Figure 95. Neuse River Mainstem Basin HEC-HMS flow comparison with USGS Little River near Princeton Gage Bulletin 17C Frequency Analysis

Table 55. Neuse River Mainstem Basin HEC-HMS flow comparison with USGS Little River near Princeton Gage Bulletin 17C Frequency Analysis

AEP	HEC-HMS (cfs)	Bulletin 17C - USGS Princeton Gage		
		Computed	5% Confidence Limits	95% Confidence Limits
0.5	2659	2305	2558	2083
0.2	4354	3747	4284	3340
0.1	5693	4912	5841	4296
0.04	7683	6642	8462	5620
0.02	9136	8130	10998	6683
0.01	10995	9798	14136	7806
0.005	13358	11671	18009	8996
0.002	17264	14504	24542	10680

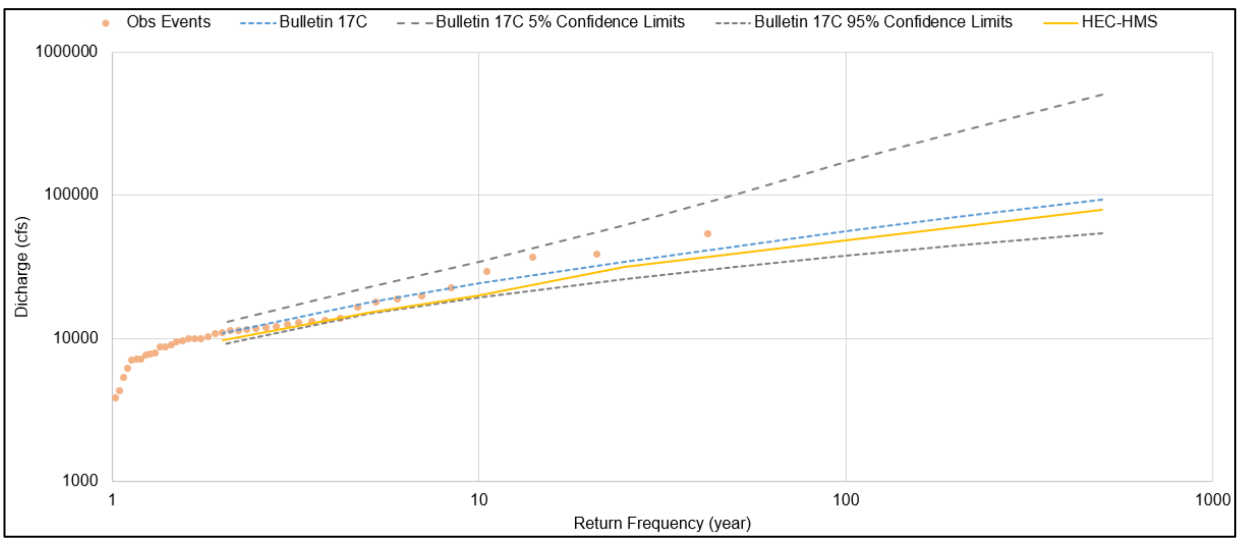

Figure 96. Neuse River Mainstem Basin HEC-HMS flow comparison with USGS Neuse River near Goldsboro Gage Bulletin 17C Frequency Analysis

Table 56. Neuse River Mainstem Basin HEC-HMS flow comparison with USGS Neuse River near Goldsboro Gage Bulletin 17C Frequency Analysis

AEP	HEC-HMS (cfs)	Bulletin 17C - USGS Goldsboro Gage		
		Computed	5% Confidence Limits	95% Confidence Limits
0.5	9706	10776	12700	9136
0.2	14991	17778	22689	14756
0.1	19808	24072	34282	19436
0.04	31472	34363	61851	26105
0.02	39316	44051	100913	31676
0.01	48427	55775	170584	37786
0.005	59950	69960	277260	44517
0.002	79014	93343	501471	54494

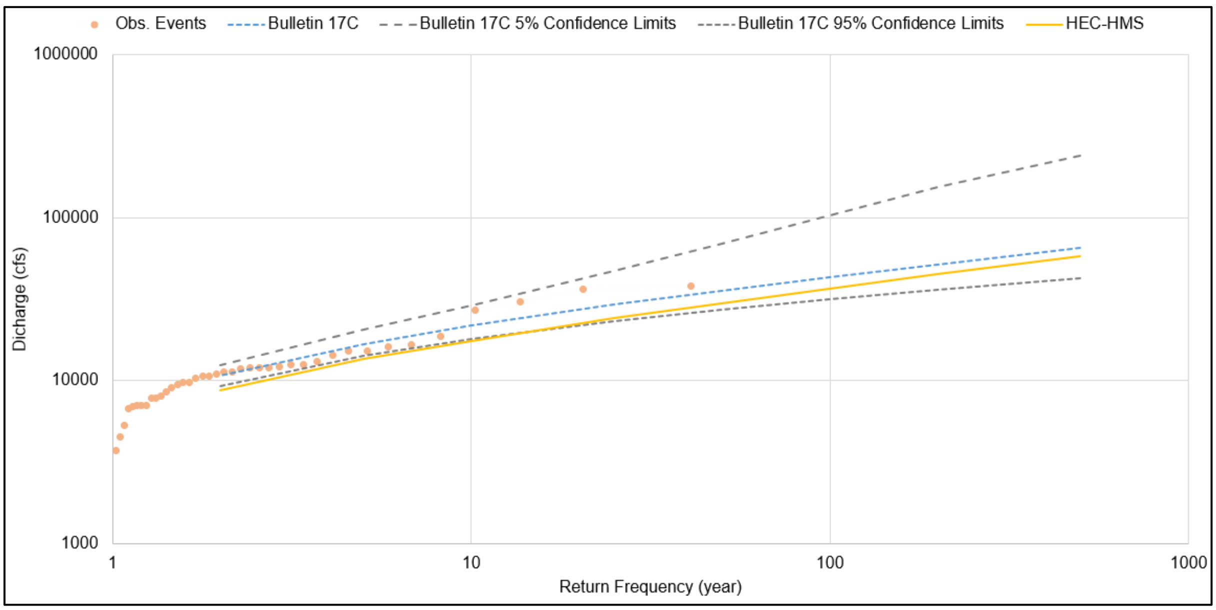

Figure 97. Neuse River Mainstem Basin HEC-HMS flow comparison with USGS Neuse River at Kinston Gage Bulletin 17C Frequency Analysis

Table 57. Neuse River Mainstem Basin HEC-HMS flow comparison with USGS Neuse River at Kinston Gage Bulletin 17C Frequency Analysis

AEP	HEC-HMS (cfs)	Bulletin 17C - USGS Kinston Gage		
		Computed	5% Confidence Limits	95% Confidence Limits
0.5	8773	10681	12387	9218
0.2	13515	16730	20584	14228
0.1	17592	21688	29062	18027
0.04	24331	29166	46893	23166
0.02	29734	35704	68828	27235
0.01	36394	43147	102367	31508
0.005	44744	51632	152928	35992
0.002	58152	64700	239783	42299

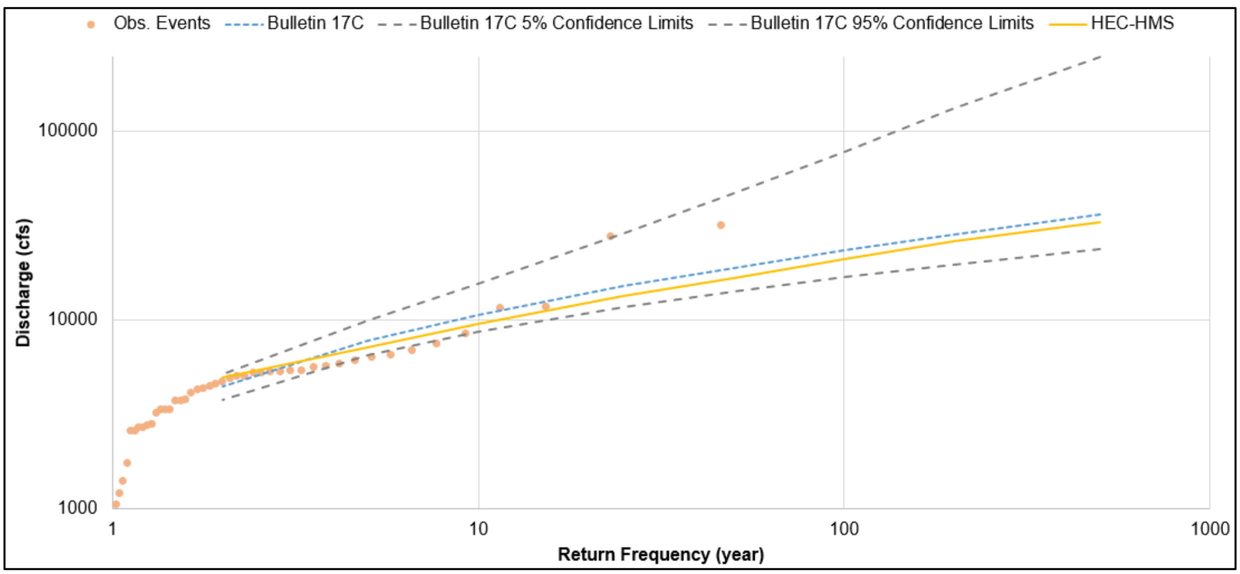

Figure 98. Neuse River Mainstem Basin HEC-HMS flow comparison with USGS Contentnea Creek at Hookerton Gage Bulletin 17C Frequency Analysis

Table 58. Neuse River Mainstem Basin HEC-HMS flow comparison with USGS Contentnea Creek at Hookerton Gage Bulletin 17C Frequency Analysis

AEP	HEC-HMS (cfs)	Bulletin 17C - USGS Hookerton Gage		
		Computed	5% Confidence Limits	95% Confidence Limits
0.5	11781	17347	20906	14536
0.2	17700	26897	34572	22206
0.1	23003	34188	47111	27574
0.04	31168	44513	68237	34596
0.02	38026	53024	88741	39936
0.01	46614	62251	114237	45331
0.005	57457	72278	145772	50812
0.002	75521	86903	199256	58199

5.2 Hydraulics

5.2.1 Hydraulic Model Background

Five separate HEC-RAS models were developed to simulate existing conditions throughout the study area. Each model footprint was associated with a corresponding HEC-HMS model as described in the previous section. All models were developed from existing FEMA-related studies or USACE efforts. Original FEMA model scope and quality were inconsistent in part due to the large study area and differing model update cycles. Several models required substantial modification that included new cross sections, reconfiguration of existing sections, addition of two-dimensional (2D) flow areas, addition of one-dimensional (1D) storage areas, geometry parameter adjustments, and georeferencing. Models obtained from existing USACE efforts included the Falls Lake Dam MMC Consequence Assessment and water management-related CWMS data. Between the difference sources, there was considerable modeling overlap, especially within the Neuse River mainstem. Refer to Sections 3.2 and 3.3 for sources of model topography, channel bathymetry, and structural data that were leveraged for the study HEC-RAS models.

The existing conditions hydraulic model associated with the Crabtree Creek basin was developed from an existing model previously constructed by a contractor for the City of Raleigh and the North Carolina Floodplain Mapping Program. This model was produced for updating the FEMA effective hydraulic model for Crabtree Creek from Lake Crabtree to the confluence with the Neuse River. It is currently associated with FEMA preliminary flood hazard data as depicted in NCFRIS (Clark, 2011; AECOM 2010).

5.2.2 Model Overview

The Neuse River mainstem HEC-RAS model was developed in the Hydrologic Engineering Center's River Analysis System (HEC-RAS), version 5.0.7. The model consists exclusively of 1D components. The Neuse River is modeled from its beginning at the downstream face of Falls Lake Dam (RS 204.193) to approximate 20 river miles past the confluence with the Trent River (RS -16.8). The total length of model along the Neuse River is approximately 221 miles. This length features over 70 storage areas and a total of 36 hydraulic structures, including 10 bridges with multiple openings. The Contentnea Creek, Little Contentnea Creek, Swift Creek (Lenoir Co.), and Trent River tributaries have been included in the model as well. These reaches were based on the USACE Falls Lake Dam consequence assessment that was associated with flood inundation resulting from dam breach. As such, the geometries of these tributaries are treated as points of potential backwater from mainstem flooding. There is a total of 341 cross sections along the Neuse River mainstem. A general location of cross sections along the Neuse River mainstem is shown in Figure 99.

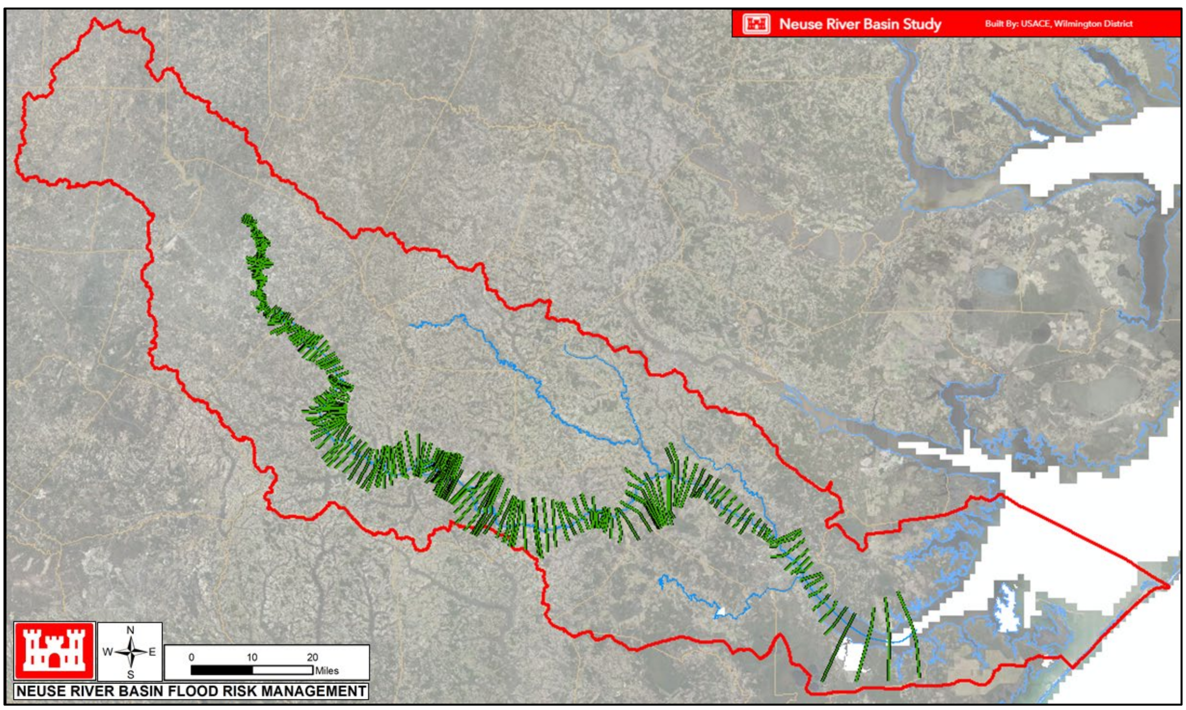

Figure 99. Neuse River Mainstem HEC-RAS General Over of Cross Sections

The Crabtree Creek HEC-RAS model was originally developed in HEC-RAS version 3.1.2 and was updated to version 5.0.7 as part of this study effort. The model consists of 1D components. The original creek extents were from approximately 1.2 miles downstream of Lake Crabtree (RS 106629) to the confluence with the Neuse River (RS 0). The study reach was shortened such that the beginning of the model was just downstream of Ebenezer Church Rd (RS 82898). The total length of model along Crabtree Creek is approximately 15.7 miles. This length features over 50 storage areas representing tributary mouths, and a total of 35 hydraulic structures, including one inline weir at Lassiter Mill Dam. Blocked obstructions are used to represent structures in the floodplain. There is a total of 285 cross sections.

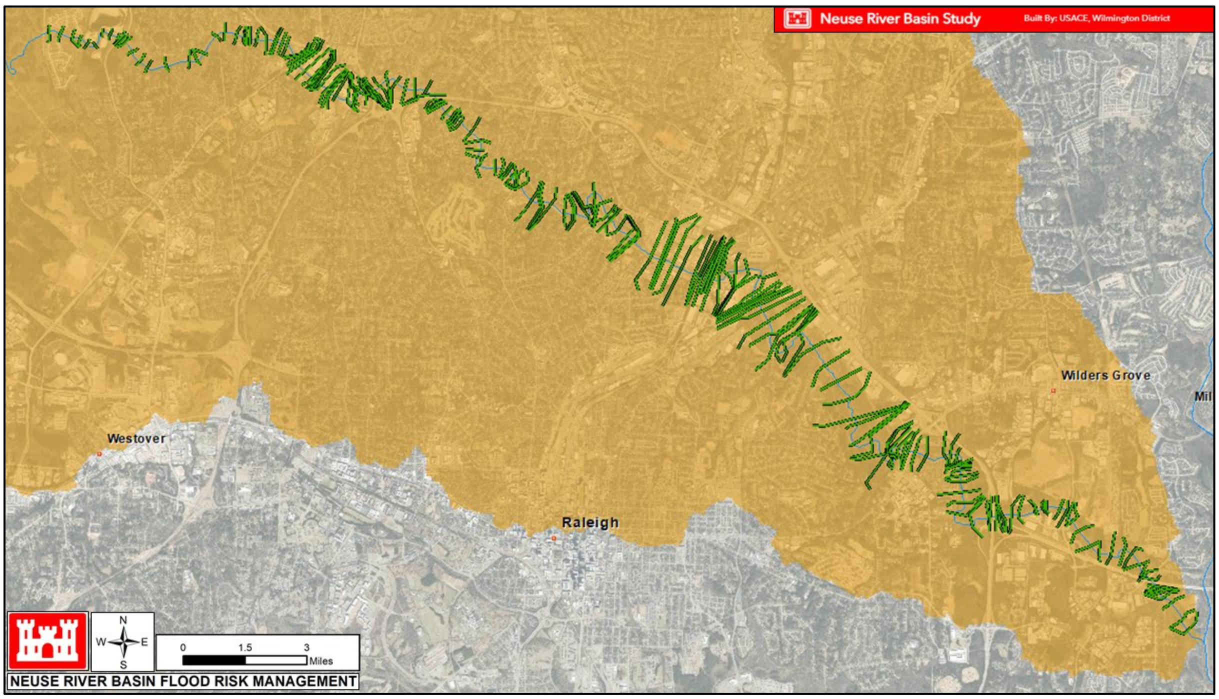

Figure 100. Crabtree Creek HEC-RAS General Overview of Cross Sections

The Hominy Swamp Creek HEC-RAS model was originally developed in HEC-RAS version 3.1 and was updated to version 5.0.7 as part of this study effort. The model consists of 1D components. Hominy Swamp Creek is modeled from its headwaters near the Wilson Industrial Air Center (RS 58310.43) to 0.5 miles upstream of the confluence with Contentnea Creek (RS 2765.41). The total length of model along Hominy Swamp Creek is approximately 10.5 miles. This length features 8 storage areas and a total of 21 hydraulic structures, including 11 bridges and 10 culverts. There is a total of 130 cross sections. A general location of cross section along Hominy Swamp Creek is shown in Figure 101.

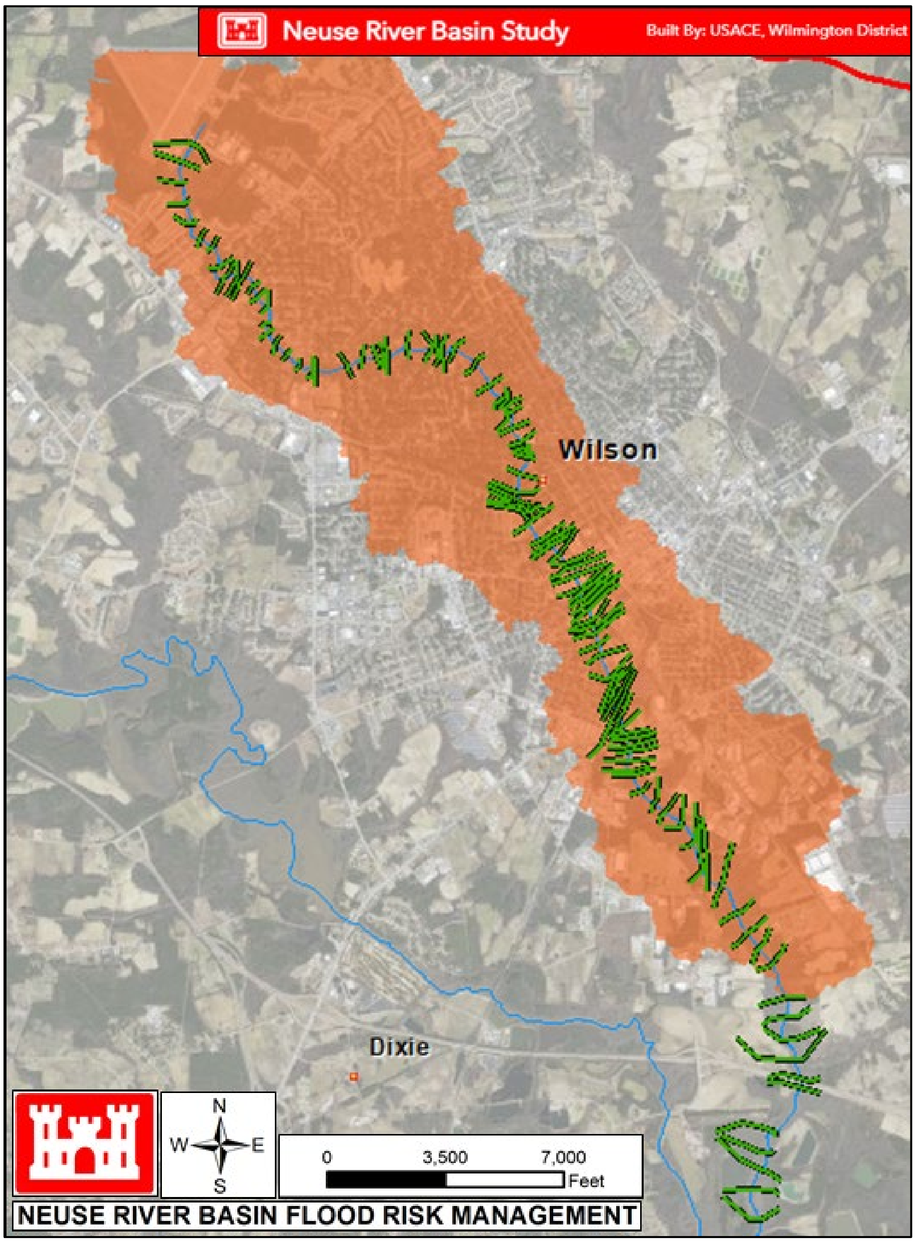

Figure 101. Hominy Swamp Creek HEC-RAS General Overview of Cross Sections

The Adkins Branch HEC-RAS model was originally developed in HEC-RAS version 3.1 and was updated to version 5.0.7 as part of this study effort. The model consists of 1D components. Adkins Branch is modeled from its headwaters situated between Sparre Dr to the north and Emerson Rd to the south (RS 28076.96) to the confluence with the Neuse River mainstem (RS 1052.762). The total length of model along Adkins Branch is approximately 5 miles. This length features a total of 12 hydraulic structures, including 2 bridges and 10 culverts. There is a total of 149 cross sections. A general location of cross section along Adkins Branch is shown in Figure 102.

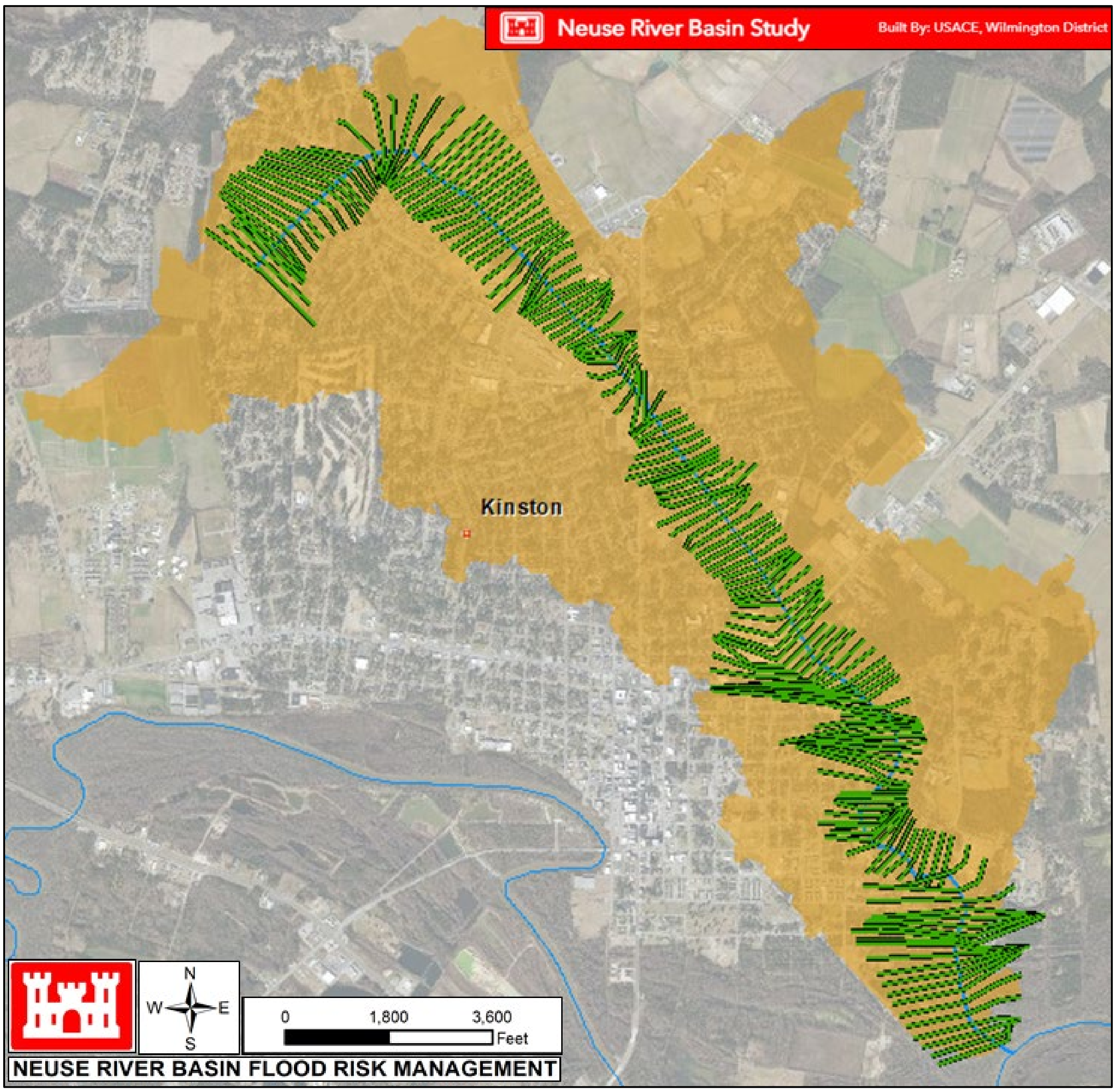

Figure 102. Adkins Branch HEC-RAS General Overview of Cross Sections

The Big Ditch HEC-RAS model was originally developed in HEC-RAS version 5.0 and was updated to version 5.0.7 as part of this study effort. The model consists of both 1D and 2D components. Big Ditch is modeled in 1D from its headwaters just downstream of Dr Martin Luther King Junior Expressway (RS 20233.13) to the confluence with the Neuse River mainstem (RS 1219.655). From just upstream of Retha St (RS 5186.766) to the confluence with the Neuse River mainstem (RS 1219.655) the left and right overbanks are modeled as 2D components. The total length of model along Big Ditch is approximately 3.6 miles. This length features a total of 21 hydraulic structures, including 5 bridges and 16 culverts. There is a total of 99 cross sections. A general location of cross section along Big Ditch is shown in Figure 103.

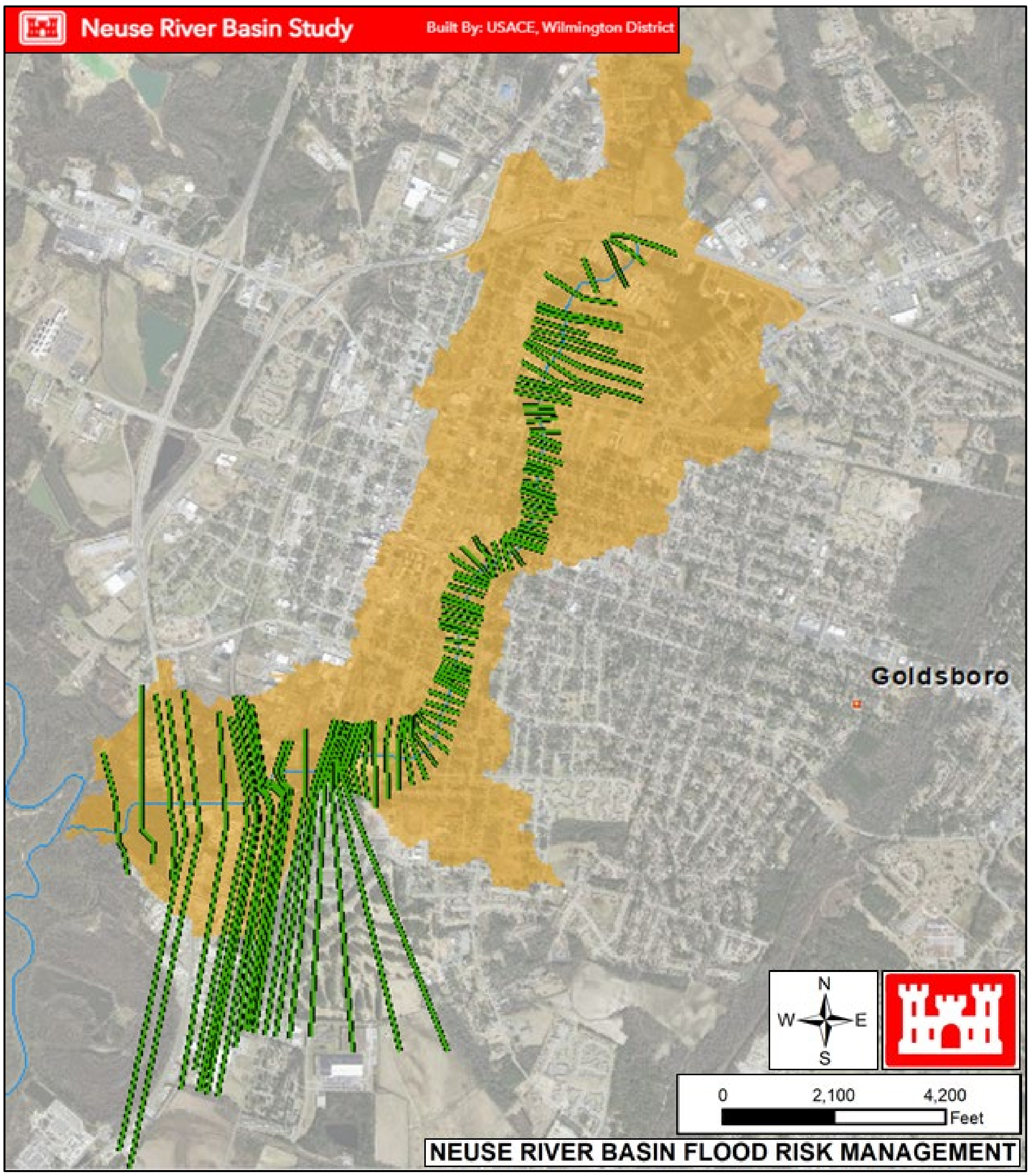

Figure 103. Big Ditch HEC-RAS General Overview of Cross Sections

5.2.3 Flow Data and Boundary Conditions

For all five HEC-RAS models, hydrologic records between the hydrologic and hydraulic models were manually transferred. Appropriate insertion of flow changes was made by applying combined flow records at all headwater cross sections. Storage areas at the headwaters of tributaries were fed a constant flowrate for initial model stabilization purposes. Local flow records were applied at cross sections that corresponded to subbasin outfall locations. Uniform lateral hydrographs were used in subbasin that were weren't significantly affected by tributary inflows. For the Crabtree Creek, Hominy Swamp Creek, Adkins Branch, and Big Ditch models, downstream boundary conditions were set to a normal depth equivalent to energy grade lines.

Development of downstream boundary conditions for the Neuse River mainstem HEC-RAS model was based on USACE Engineering Manual (EM) 1110-2-1416 (Engineering and Design River Hydraulics) which states that when the profile computation begins at the outlet of a stream influenced by tidal fluctuations, the maximum predicted high tide, including wind-wave set up, is taken as the starting elevation at a station usually located at the mouth of the stream. For the Neuse River mainstem, USGS Neuse River near Fort Barnwell station (02091814), roughly 25 river miles upstream from downtown New Bern, indicate that flow is affected by both astronomical and wind tides. These tidal fluctuations originate from the Pamlico Sound estuary, beyond the mouth of the Neuse River. The station of the downstream boundary is located near the intersection of Carteret, Craven, and Pamlico Counties. Due to its long period of record and a high degree of confidence in its established datums, the Beaufort, Duke Marine Lab, NC NOAA tide buoy (8656483) was selected to determine the maximum predicted high tide for downstream boundary condition. The NOAA site has a mean higher-high water datum of 3.54 feet or 1.46 feet, NAVD88 datum (https://tidesandcurrents.noaa.gov/datums.html?id=8656483). While there were several CO-OPS stage gages closer to the mouth of the Neuse River (ORLN7-Neuse River at Oriental, NC, HBKN7-Pamlico Sound near Hobucken, and CTIN7-Pamlico Sound at Cedar Island), these sites lacked a robust period of record, nor did they provide established datums similar to the Beaufort, NC NOAA site. Therefore, there would be a high degree of uncertainty related to stage/elevation datum conversions. The Beaufort, NC NOAA site is located in Carteret County and provided a conservative maximum predicted high tide value.

A review of two additional NOAA gauge sites was taken to verify the conservative choice of the Beaufort, NC NOAA site. The sites are shown in Figure 104. NOAA #8654467 USGS Station Hatteras NC and NOAA #8652587, Oregon Inlet Marina NC sites were located facing the estuary side (as opposed to directly facing the Atlantic Ocean). In this regard, they would potentially be more representative of the buffering effects from the barrier islands. Mean higher-high water datums (in NAVD88) for NOAA #8654467 and #8652587 were 0.26 feet and 0.48 feet, respectively.

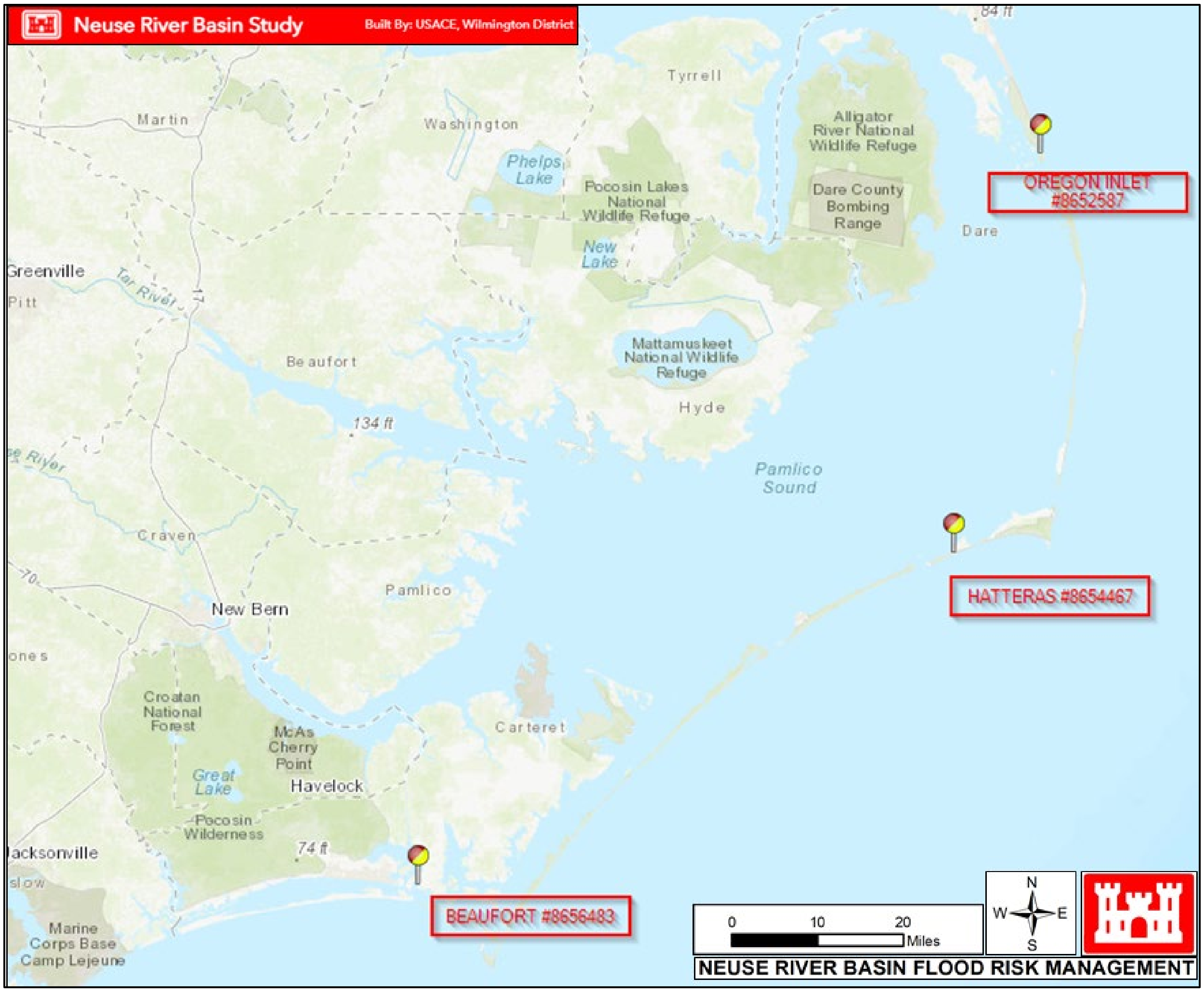

Figure 104. NOAA Tide Gauges near Study Area

Wind-wave setup was accounted for by leveraging the Engineering Research and Development Center (ERDC) Coastal and Hydraulics Laboratory (CHL) Coastal Hazard System (CHS). The CHS data is a probabilistic coastal hazard assessment providing results and statistics based on high-resolution numerical modeling of coastal storms. Select CHS virtual gauge nodes located within the Pamlico Sound, were chosen to represent the downstream boundary condition (https://chs.erdc.dren.mil/Study/Index). Specific node data included Significant Wave Height per return frequency, as developed for the South Atlantic Coastal Study (SACS). Values for the 0.5-, 0.2-, 0.1-, 0.05-, 0.02-, 0.01, 0.005-, and 0.002-AEP return frequencies were extracted from virtual gauge node #4425 and are listed in Table 59.

Table 59. CHS Frequency Wave Characteristics for Downstream Boundary

Return Frequency (AEP)	Significant Wave Height (ft)
0.5	2.14
0.2	2.99
0.1	3.71
0.05	4.34
0.02	5.03
0.01	5.46
0.005	5.85
0.002	6.32

A Significant Wave Height value for the 0.4-AEP was linearly interpolated for use in this study. A review of coastal analysis within the 2020 FEMA FIS showed a range of 0.01-AEP Significant Wave Heights between 5.3-ft to 5.8-ft, in close agreement with the CHS analysis. The final existing conditions downstream boundary condition was computed by adding a constant 1.46-foot value, representing maximum high tide, to the Significant Wave Heights shown above. The final values are listed in Table 60. A comparison of these final values minus averaged CHS Still Water Levels near the downstream boundary showed an averaged difference of -0.4 feet, with a range of +1.4 feet to -1.9 feet, increasing as AEP decreased.

Table 60. Neuse River Mainstem Downstream Boundary Conditions – Starting Water Surface Elevation

Return Frequency (AEP)	Starting Water Surface Elevation (ft, NAVD88)
0.5	3.60
0.2	4.45
0.1	5.17
0.4	5.64
0.2	6.49
0.01	6.92
0.005	7.31
0.002	7.78

The water surface elevation boundary condition time series was based on a ratio between the peak stage observed during the historic Hurricane Matthew event and the different return frequency values listed in the preceding table. Hydrograph ordinates of each 15-minute timestep were multiplied by this ratio to produce the final stage hydrograph. Sensitivity analysis demonstrated that using a constant water surface elevation as the downstream boundary condition produced similar inundation extents and depths but was determined to be overly conservative based on historical performance of the river and estuary.

5.2.4 Calibration

The Neuse River mainstem HEC-RAS model was calibrated to high-water marks for Hurricane Matthew in 2016 (USGS, 2017). Due to the lessened impact from the event to the upper portion of the basin, there were limited High Water Marks (HWM) collected above Smithfield, NC. The majority of HWMs were collected between Smithfield, Goldsboro, and Kinston. A comparison of computed water surface elevations and high-water marks is listed in Table 61. Overall, computed water surface elevations are within 1.0 foot at each of these locations, indicating successful calibration.

In addition to HWMs, stage-discharge rating curves were extracted from the HEC-RAS at cross sections representative of USGS streamflow gage locations. Published USGS rating curves were then plotted against the modeled curve for comparison. This comparison would provide insight on how well the model was able to replicate water levels over a range of flows, beyond the single HEC-RAS calibration event. It would also help supplement the somewhat inconsistent spacing of HWMs throughout the modeled reach. There were some limitations given that generally USGS ratings curve accuracy decreases as out-of-bank flow becomes more prominent as a portion of the total cross section flow area profile. Furthermore, the surveyed cross section data that USGS leverages to create their rating curve, that are continually assessed and shifted, were not part of the bathymetry and topology used in the HEC-RAS model. The stage-discharge rating curves for HEC-RAS cross section representative of the USGS Clayton (02087500), Goldsboro (02089000), and Kinston (02089500) gage locations are shown in Figure 105, Figure 106, and Figure 107 below. Overall, the HEC-RAS computed curves slightly overestimated stages for flows that were generally confined to the river channel, where differences in bathymetry data between the study model and USGS would have a greater effect. All three published USGS rating curves were not extended high enough to capture stages and flows anticipated during the most infrequent events as part of this study (i.e., 0.005- and 0.002-AEP events). Nevertheless, the comparison showed that both Goldsboro and Kinston stage-discharge rating curves produced by the HEC-RAS matched well with the USGS published curves for moderate-to-severe flow conditions.

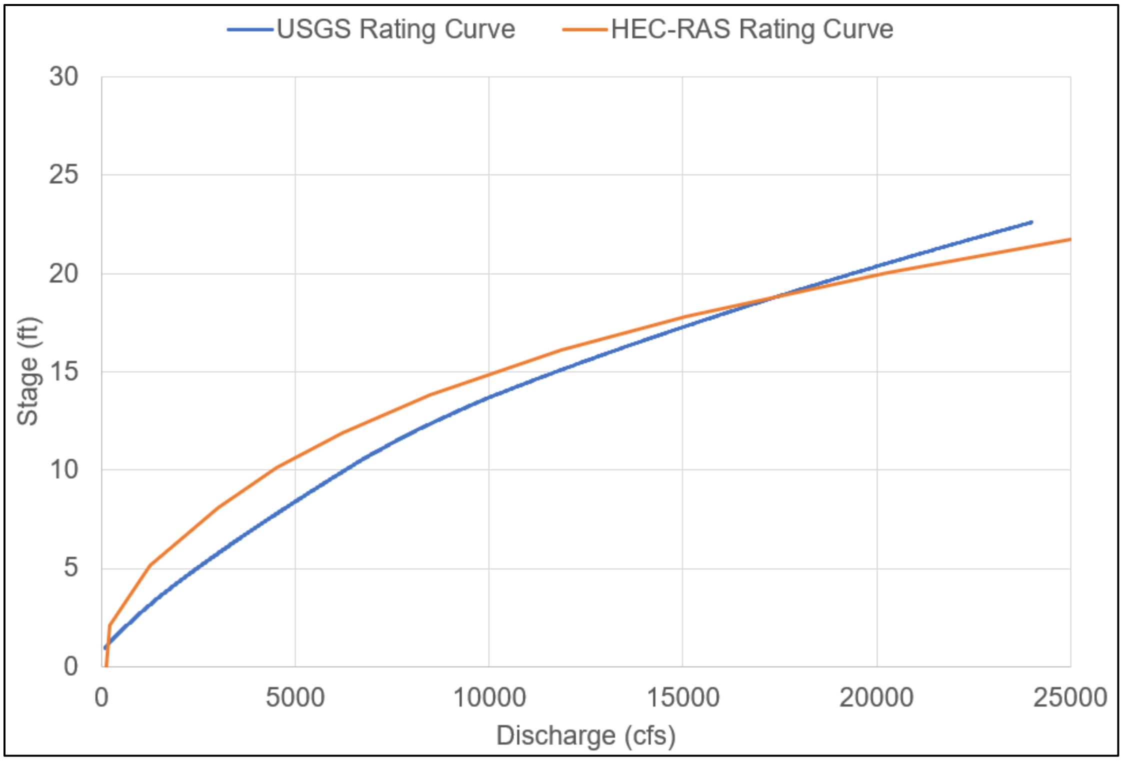

Figure 105. Neuse River Mainstem HEC-RAS Computed vs. USGS Stage-Discharge Rating Curve at 02087500 Clayton Gage

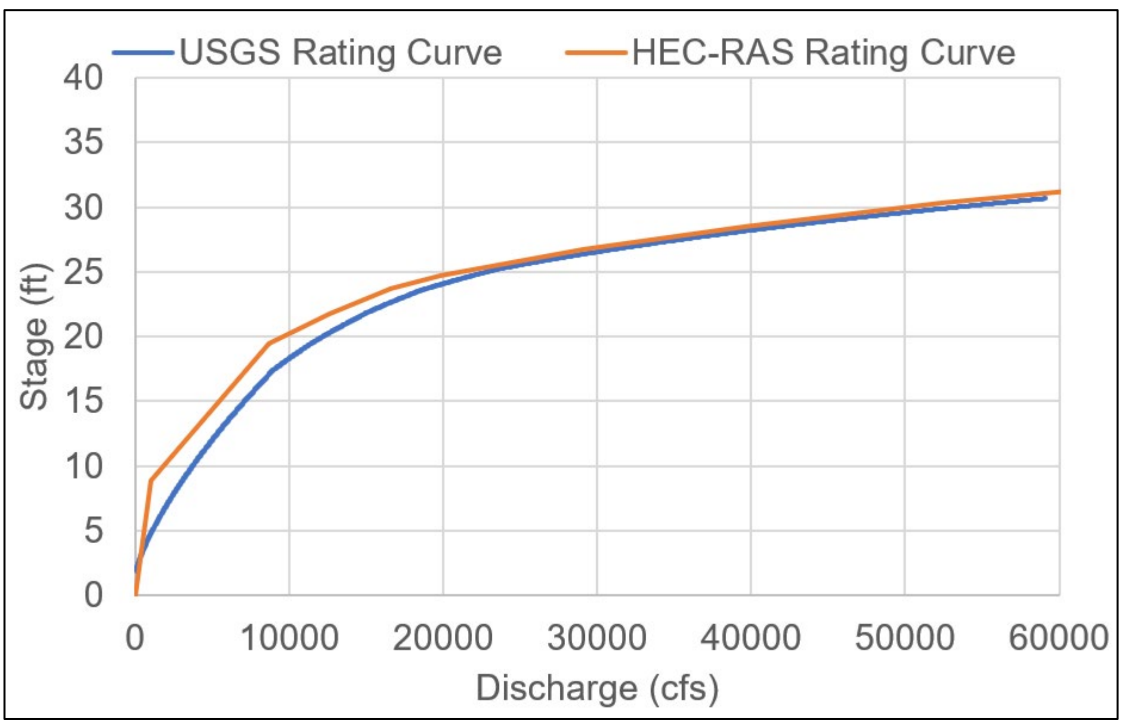

Figure 106. Neuse River Mainstem HEC-RAS Computed vs. USGS Stage-Discharge Rating Curve at 02088000 Goldsboro Gage

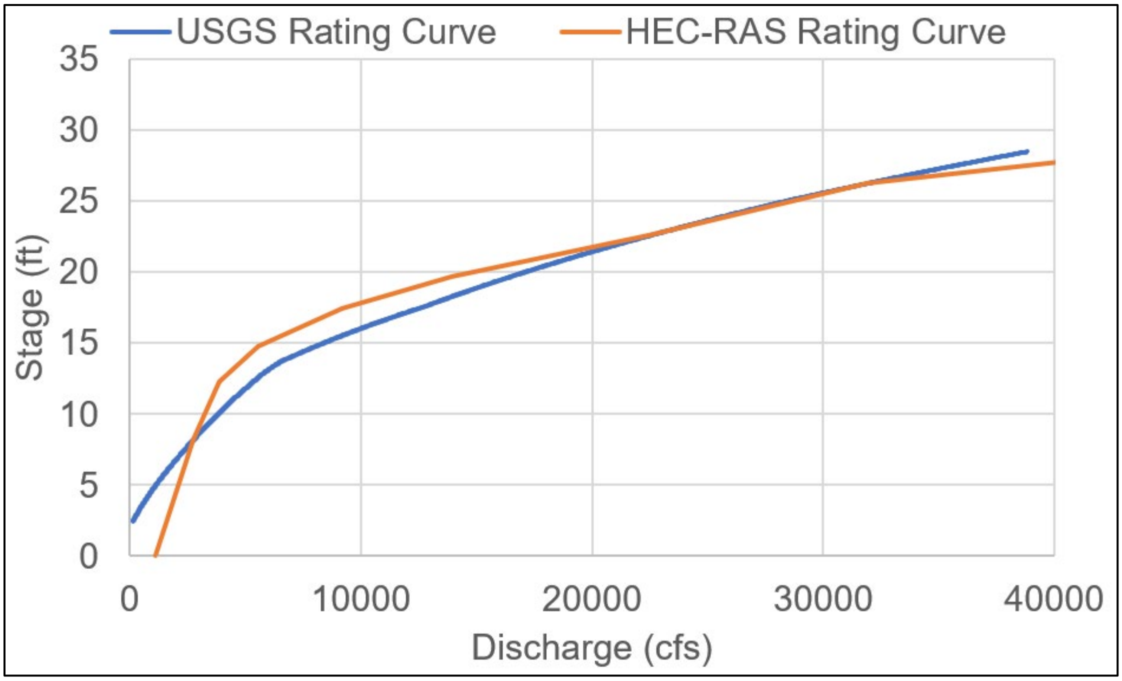

Figure 107. Neuse River Mainstem HEC-RAS Computed vs. USGS Stage-Discharge Rating Curve at 02089500 Kinston Gage

Table 61. Neuse River Mainstem HEC-RAS Calibration to Hurricane Matthew High-Water Marks

River Station	HWM Description	High-Water Mark (ft, NAVD88)	Computed WSEL (ft, NAVD88)	Difference (ft)
171.336	USGS 02087500 Clayton	147.9	148.0	0.1
158.045		128.5	128.3	-0.2
157.412	USGS 02087570 Smithfield	128.1	128.0	-0.1
157.077		127.4	127.2	-0.2
157.042		127.4	126.7	-0.7
153.551		122.9	122.4	-0.5
109.533		74.4	74.0	-0.4
100.552		72.6	72.7	0.1
99.505	USGS 02089000 Goldsboro	71.7	72.2	0.5
99.102		71.5	71.8	0.3
97.249		69.2	70.0	0.8
54.87	USGS 02089500 Kinston	38.1	37.7	-0.4

The calibration process for the Crabtree Creek HEC-RAS model was provided by AECOM. A number of high-water marks obtained for Tropical Storm Alberto were compared to modeled water surface elevations. Results of this calibration are listed in Table 62 through Table 64. Adjustments of manning's n values and ineffective flow areas were primarily used to calibrate the model. Overall, model results were able to replicate observed stages produced by the event. For HWMs along Crabtree Creek, average differences between computed values and observed were near 0.3-ft with a standard deviation of ~0.95-ft. Calibrated values beyond that range were seen along tributaries to Crabtree Creek, however, they were located above Lake Crabtree and did not have a significant impact to the Crabtree Creek study area, located downstream of Lake Crabtree.

Table 62. Crabtree Creek HEC-RAS Tropical Storm Alberto HWM Comparison – Part 1

Stream	Location	TS Alberto HWM WS EL (ft)	Modeled WS EL (ft)	Δ WS EL (ft)
Crabtree Creek	USGS Gage at Wake Forest Rd	205.5	205.7	0.2
Crabtree Creek	USGS Gage at Anderson Dr	207.6	207.1	-0.6
Crabtree Creek	USGS Gage at US 70	226.6	227.6	1.0
Crabtree Creek	D/S side of US 264	172.5	173.6	1.2
Crabtree Creek	1100 ft D/S of New Bern Ave	187.6	188.0	0.4
Crabtree Creek	300 ft U/S of New Bern Ave	188.9	189.5	0.6
Crabtree Creek	D/S side of Capital Blvd	200.0	199.0	-1.0
Crabtree Creek	D/S side of Capital Blvd	198.7	199.0	0.3
Crabtree Creek	CTC_HWM_20	202.0	201.6	-0.4
Crabtree Creek	CTC_HWM_22	204.0	205.6	1.6
Crabtree Creek	800 ft D/S of Wake Forest Rd	204.4	205.6	1.2
Crabtree Creek	350 ft U/S of Wake Forest Rd	205.9	206.1	0.2
Crabtree Creek	700 ft U/S of Wake Forest Rd	207.0	206.1	-0.9
Crabtree Creek	100 ft U/S of Anderson Dr	207.6	208.8	1.2
Crabtree Creek	U/S side of Lassiter Mill Rd	211.1	212.9	1.8
Crabtree Creek	Yadkin Rd	218.5	219.9	0.6
Crabtree Creek	1800 ft D/S of Creedmoor Rd	231.7	229.9	-1.8

Table 63. Crabtree Creek HEC-RAS Tropical Storm Alberto HWM Comparison – Part 2

Stream	Location	TS Alberto HWM WS EL (ft)	Modeled WS EL (ft)	Δ WS EL (ft)
Crabtree Creek	3000 ft U/S of Creedmoor Rd	234.0	233.7	-0.3
Basin 15, Stream 25	Hooper St	216.6	217.1	0.5
Basin 15, Stream 25	Forestville Rd	247.2	248	0.8
Basin 18, Stream 16	1200 ft d/s of Glenwood Ave	340.6	339.3	-1.3
Basin 18, Stream 4	Country Tr.	376.8	377.2	0.6
Basin 18, Stream 8	Unnamed Road	389	388.5	-0.5
Beaverdam Creek	Scotland St	214.7	216.1	1.4
Beaverdam Creek (B12, S1)	Old Milburnie Rd	185	186.4	1.4
Beaverdam Creek (B15, S21)	N Beaver Ln	181.2	180.5	-0.7
Big Branch	Hardimont Dr	227.8	227.7	-0.1
Big Branch	Cheswick Dr	211.1	211.3	0.2
Bridges Branch	Barksdale Dr	195.1	193.5	-1.6
East Fork Mine Creek	Newton Rd	326.5	323.2	-3.3
Haresnipe Creek	Millbrook Rd	294.1	292.5	-1.6
Haresnipe Creek	Rembert Dr	264.4	264.3	-0.1
House Creek	639 ft u/s Horton St	305.2	305.6	0.4
House Creek	Blue Ridge Rd	227.9	229.3	1.4
Little Brier Creek	1600 ft u/s Brier Creek Pkwy	328.8	328.9	0.1
Little Brier Creek	US-70	338.9	343.3	4.4
Lynn Road Tributary	Lead Mine Rd	306.1	305.5	-0.6
Marsh Creek	Capital Blvd	214.2	210.9	-3.3
Marsh Creek	New Hope Church Rd	231.8	230.1	-1.7
Marsh Creek	Quail Ridge Rd	290.2	288.6	-1.6
Mine Creek	Millbrook Rd	234.8	235.2	0.4
Mine Creek	Lynn Rd	272.6	275.3	2.7
New Hope Tributary	New Hope Church Rd	250.5	251.1	0.6

Table 64. Crabtree Creek HEC-RAS Tropical Storm Alberto HWM Comparison – Part 3

Stream	Location	TS Alberto HWM WS EL (ft)	Modeled WS EL (ft)	Δ WS EL (ft)
Pigeon House Branch	Capital Blvd	206	204.8	-1.2
SW Prong Beaverdam Creek	Just DS of Market Bridge	243.4	244.1	0.7
Sycamore Creek	250 ft downstream of Glenwood Ave	365.9	364.3	-1.6
Turkey Creek	W Lake Anne Rd	335.1	334.3	-0.8

Calibration for Hominy Swamp Creek, Adkins Branch and Big Ditch HEC-RAS models was not possible due to lack of observed event data in the form of streamflow gage or collected high-water marks. The USGS Big Ditch at Retha St at Goldsboro, NC gage (02088682) was investigated for use in calibration. The latest available rating curve at the historical gage location dated back to the mid-1980s. Furthermore, the gage was only able to capture low flow conditions. Due to the number of uncertainties related to this historical gage, it was not utilized for calibration. Professional judgment was used to select channel and overbank manning's n values that were consistent with calibrated models elsewhere in the study area.

5.2.5 Validation

In order to gage the accuracy of model calibrations and performance, The Neuse River mainstem HEC-RAS model was validated to the Hurricane Florence event in 2018 (USGS, 2019). High-water marks were used to assess the accuracy of modeled water surface elevation of this event simulation. A comparison of computed water surface elevations and high-water marks is listed in Table 65. In general, there was agreement between the two sources. Computed water surface elevations were within 2 feet of observed data at various locations in the basin along the mainstem. Validation to HWMs between the NC-11, Queen St, and railroad bridges near Kinston slightly underestimated water surface elevations (WSEL). However, validation to HWMs a short distance both upstream and downstream of this segment were within 0.5-ft.

Table 65. Neuse River Mainstem HEC-RAS Validation to Hurricane Florence High-Water Marks

River Station	HWM Description	High-Water Mark (ft, NAVD88)	Computed WSEL (ft, NAVD88)	Difference (ft)
171.336	USGS 02088000 Clayton	138.1	139.0	0.9
112.289		73.9	74.0	0.1
101.166		70.53	70.6	0.1
99.505	USGS 02089000 Goldsboro	69.5	69.8	0.3
99.102		69.2	69.37	0.2
57.492		36.79	36.22	-0.6
55.959		35.37	35.21	-0.2
54.894		35.41	34.28	-1.1
54.87	USGS 02089500 Kinston	35.5	33.8	-1.7
53.744		34.74	33.33	-1.4
51.98		31.36	31.83	0.5

5.2.6 Frequency Simulation Results

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.

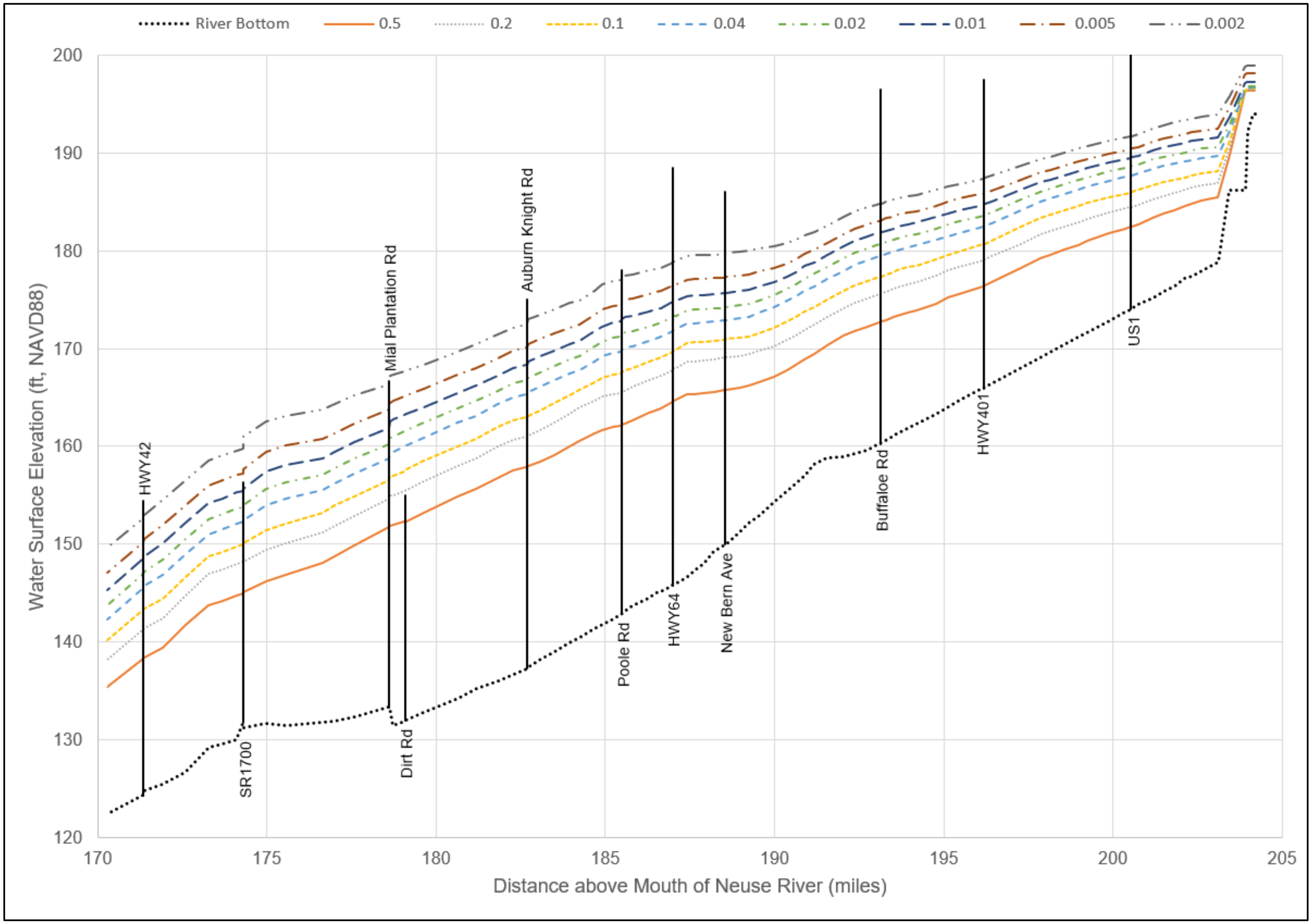

Figure 108. Neuse River Existing Conditions Modeled Water Surface Profiles for Select Design Events from Falls Lake Dam to HWY-42 in Johnston County

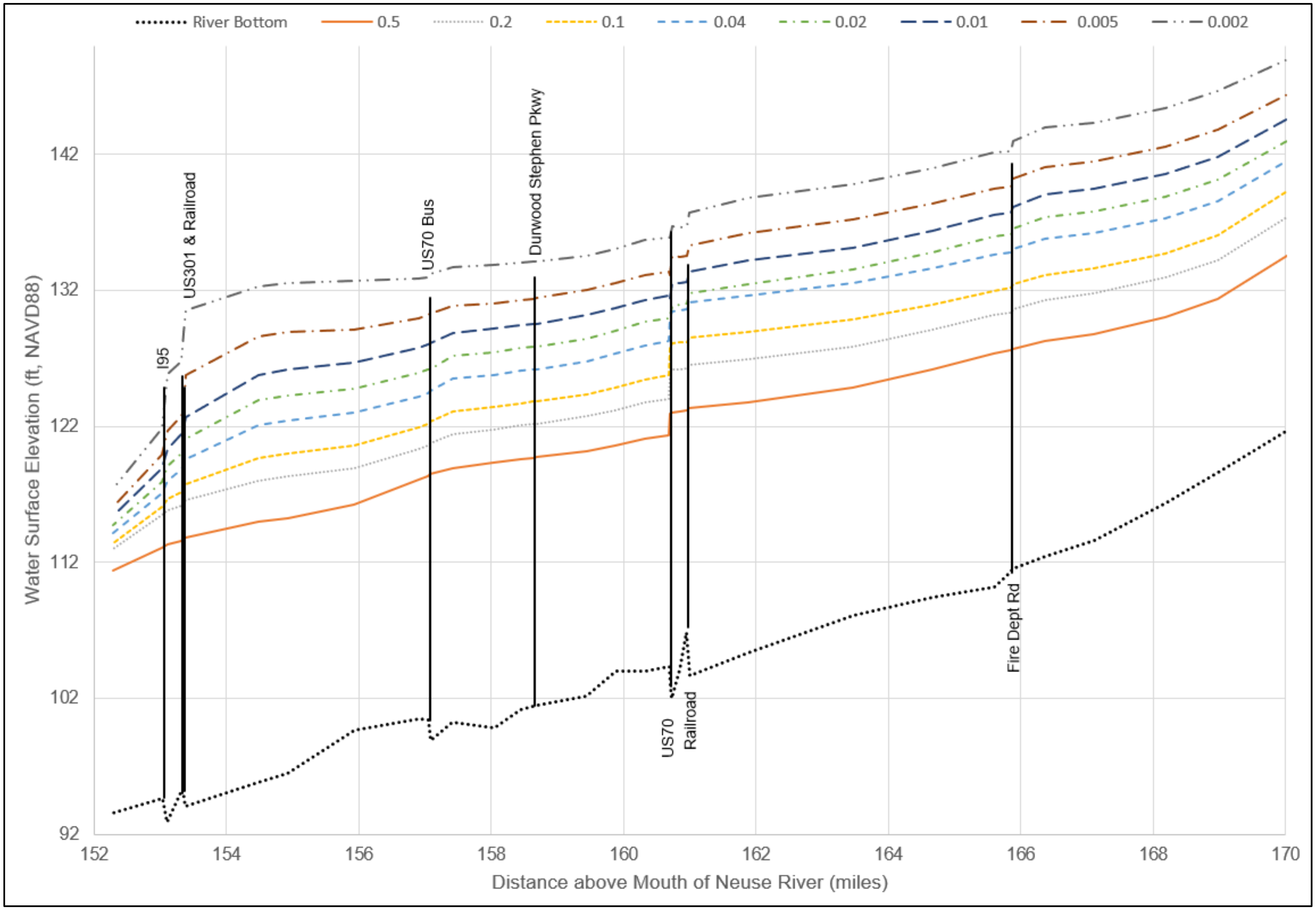

Figure 109. Neuse River Existing Conditions Modeled Water Surface Profiles for Select Design Events from HWY-42 to I-95 in Johnston County

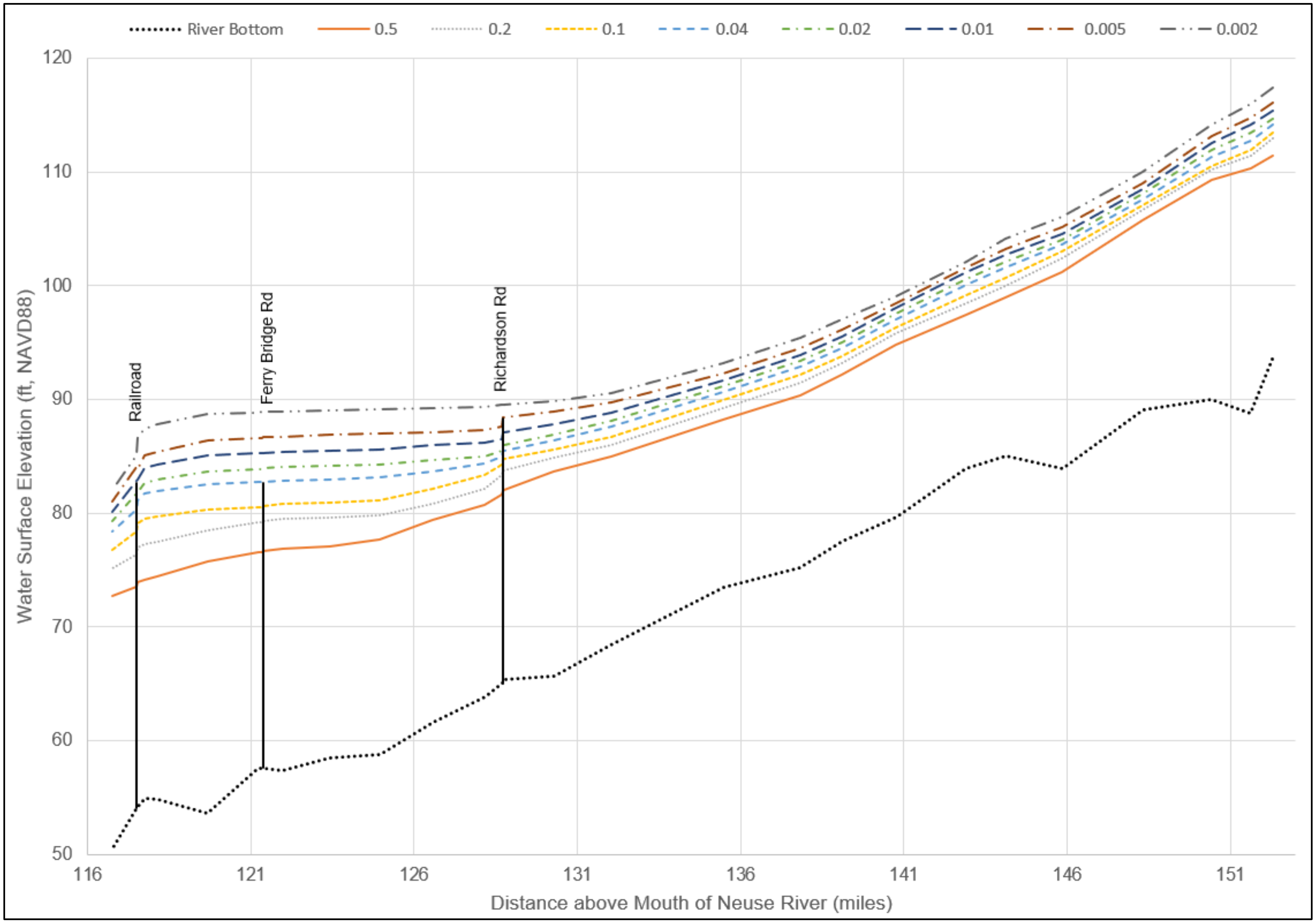

Figure 110. Neuse River Existing Conditions Modeled Water Surface Profiles for Select Design Events from I-95 to Ferry Bridge Rd in Wayne County

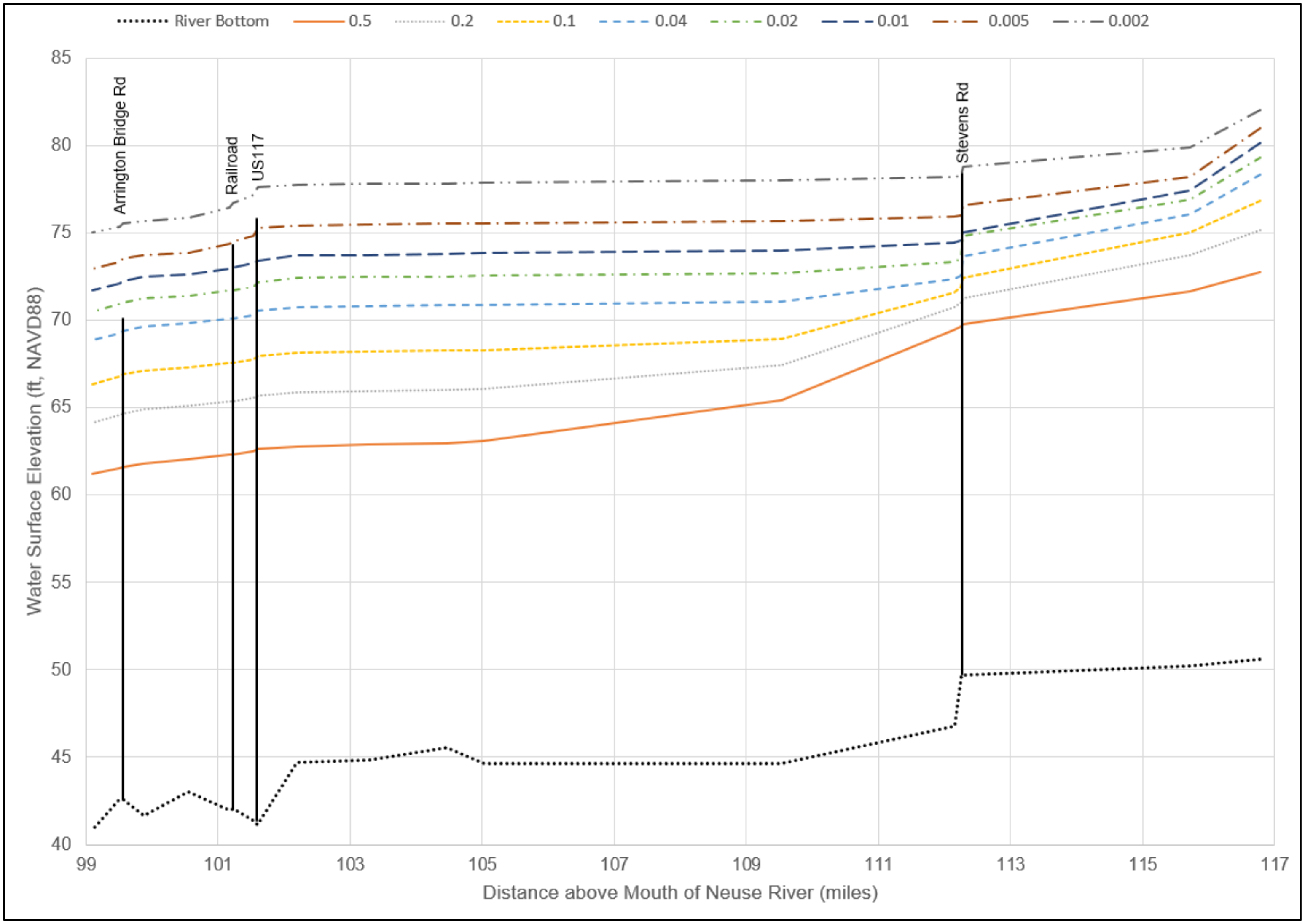

Figure 111. Neuse River Existing Conditions Modeled Water Surface Profiles for Select Design Events from Ferry Bridge Rd to Arrington Bridge Rd in Wayne County

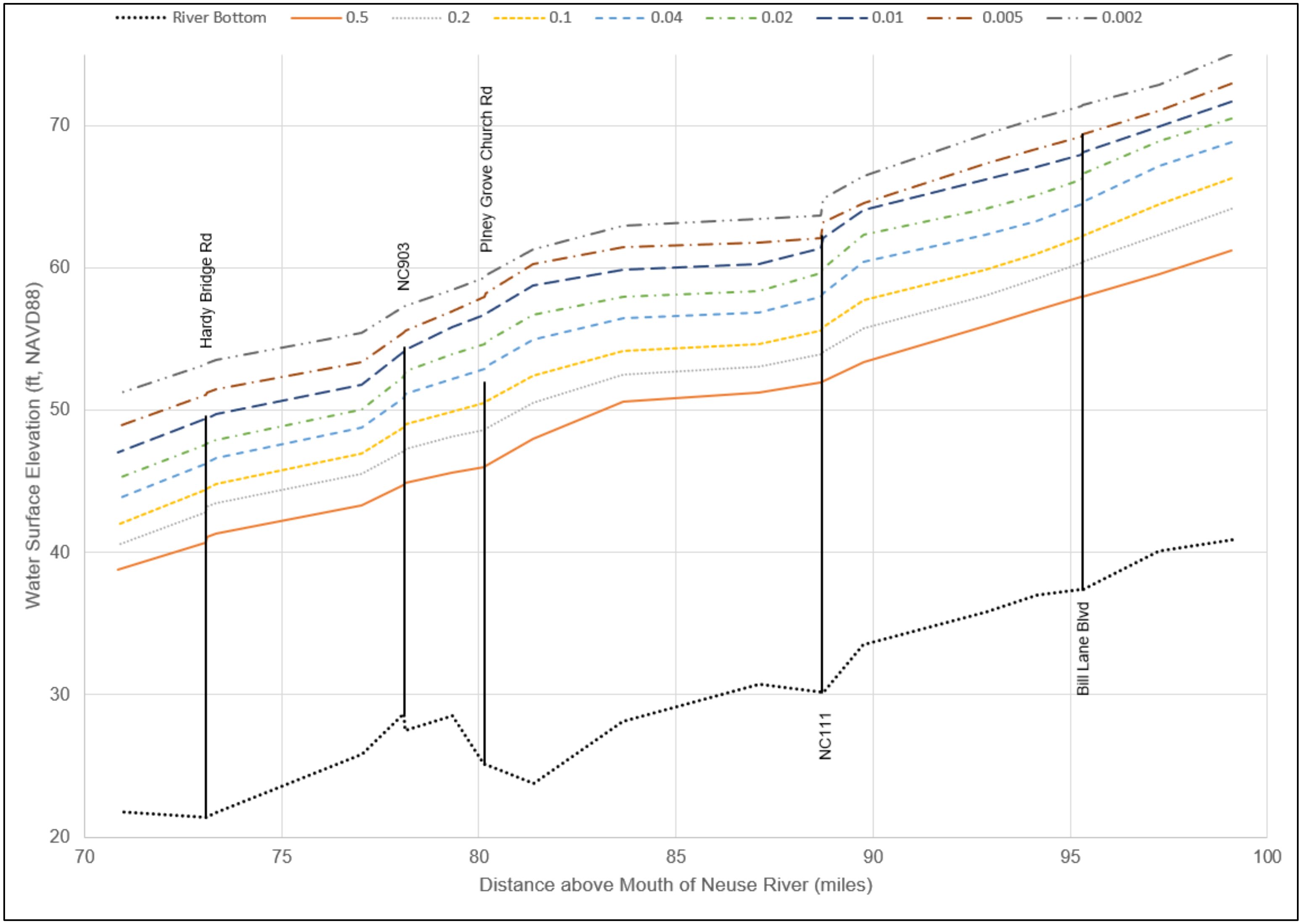

Figure 112. Neuse River Existing Conditions Modeled Water Surface Profiles for Select Design Events from Arrington Bridge Rd to Hardy Bridge Rd in Lenoir County

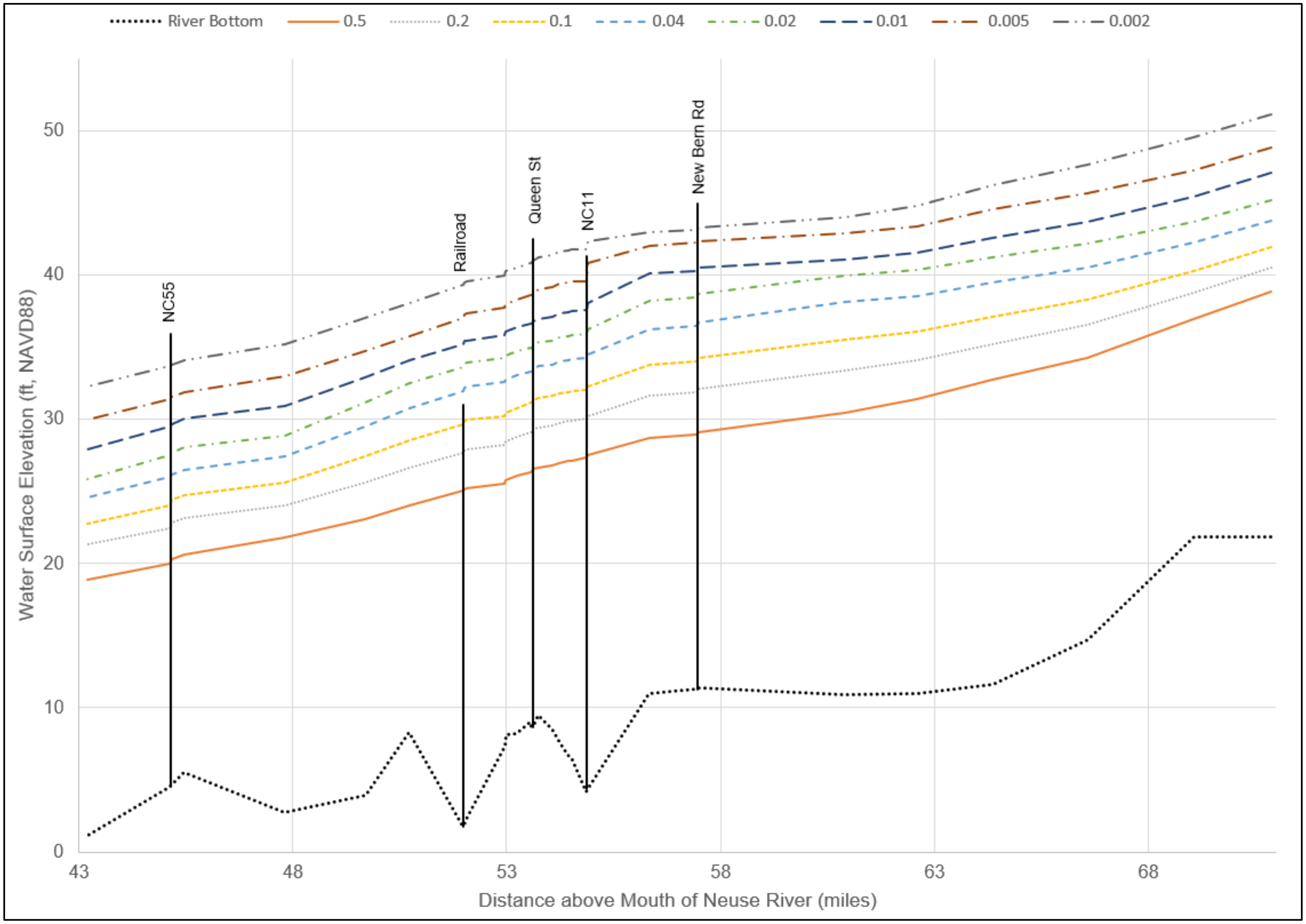

Figure 113. Neuse River Existing Conditions Modeled Water Surface Profiles for Select Design Events from Hardy Bridge Rd to NC-55 in Lenoir County

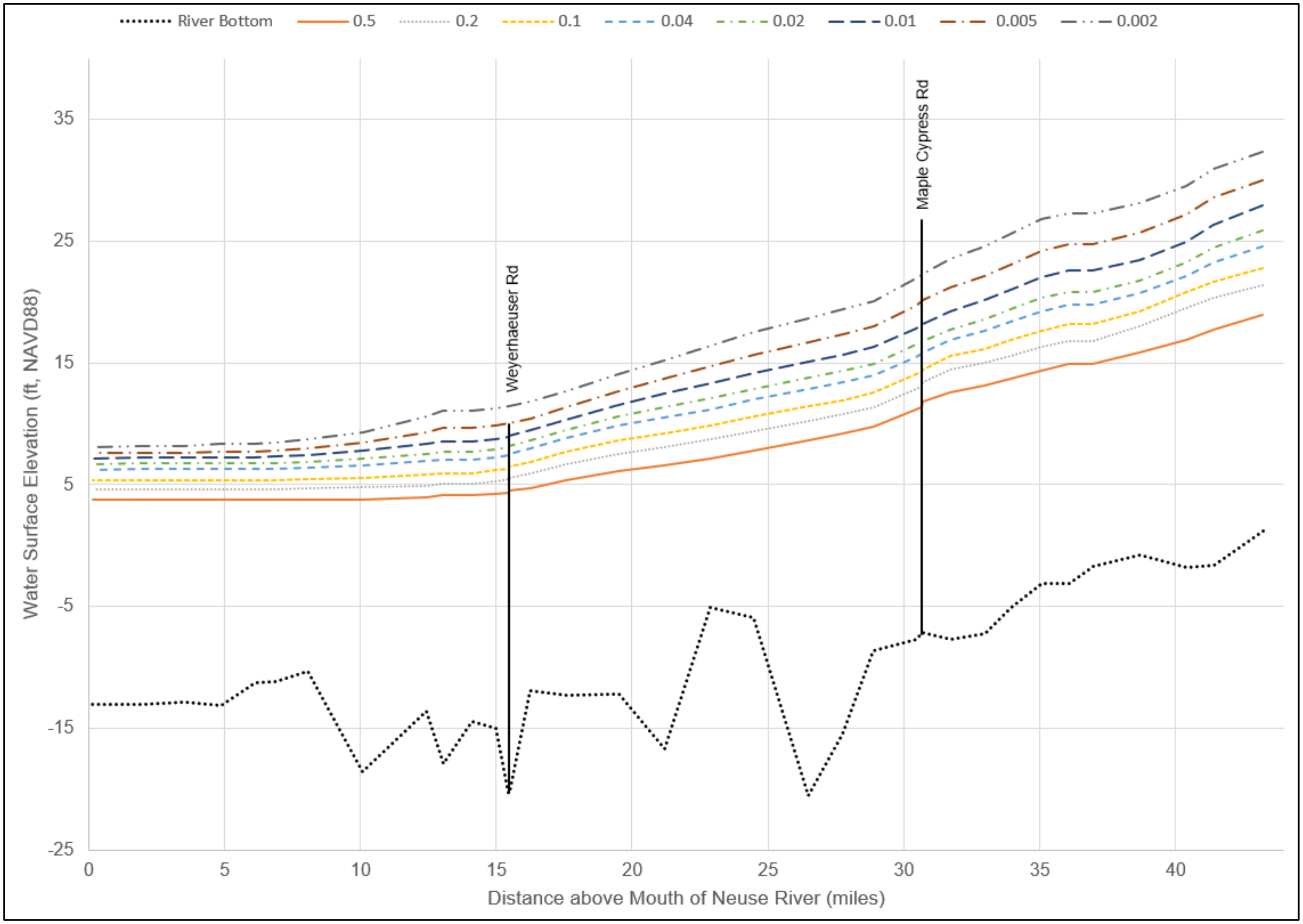

Figure 114. Neuse River Existing Conditions Modeled Water Surface Profiles for Select Design Events from NC-55 to Mouth in Craven County

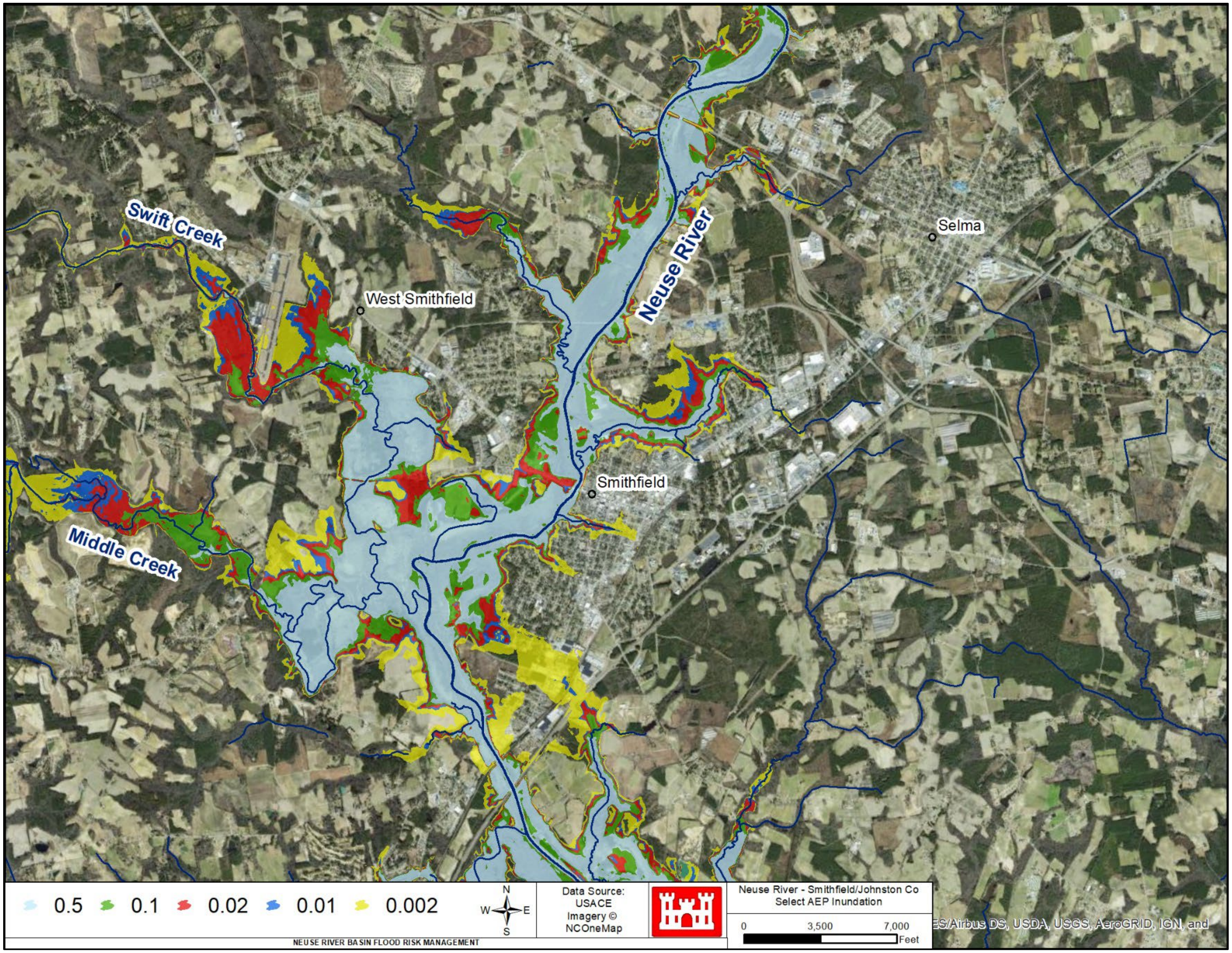

Figure 115. Neuse River Existing Conditions Modeled Inundation for Select Design Events near Smithfield, Johnston County

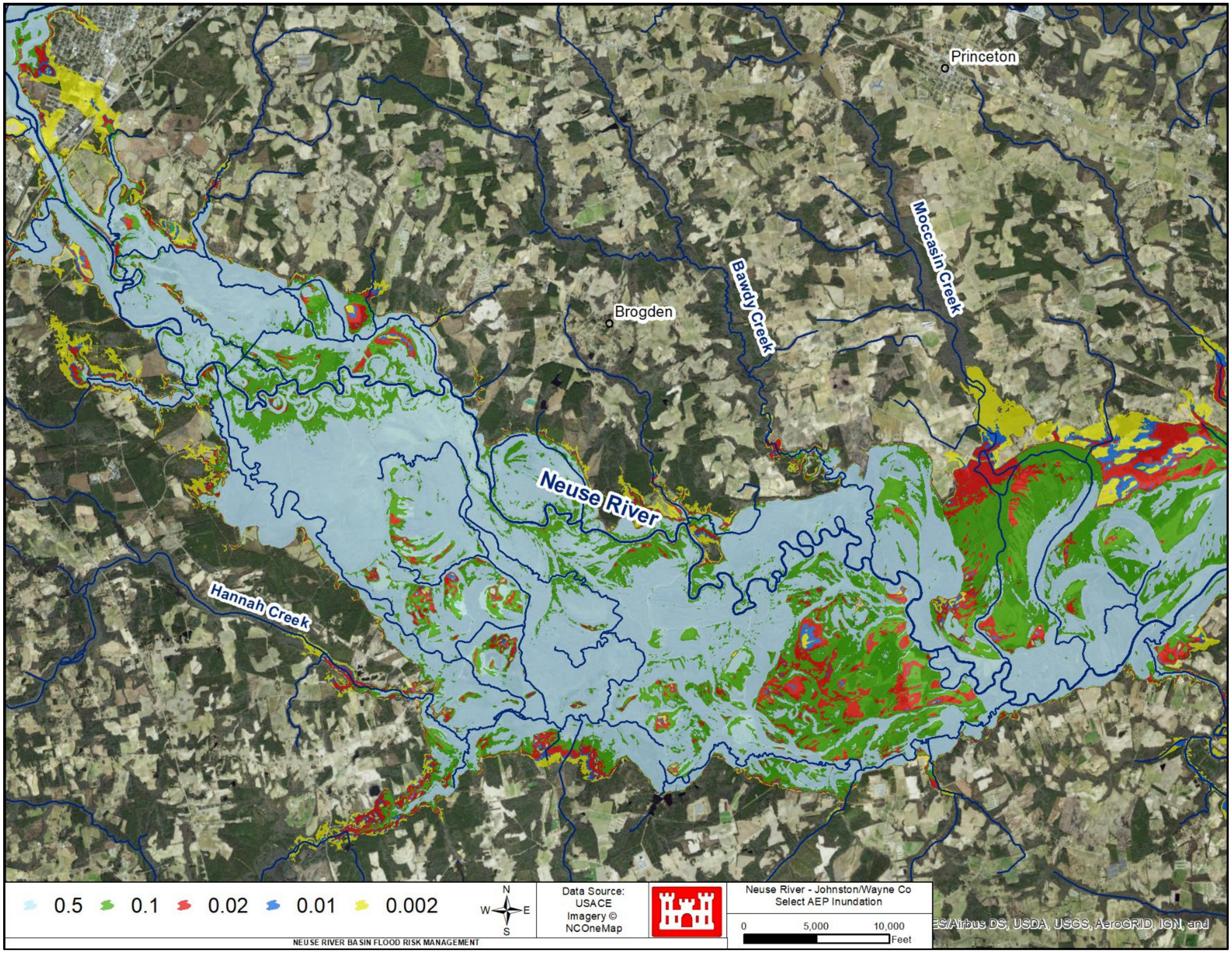

Figure 116. Neuse River Existing Conditions Modeled Inundation for Select Design Events in Rural Johnston and Wayne Counties

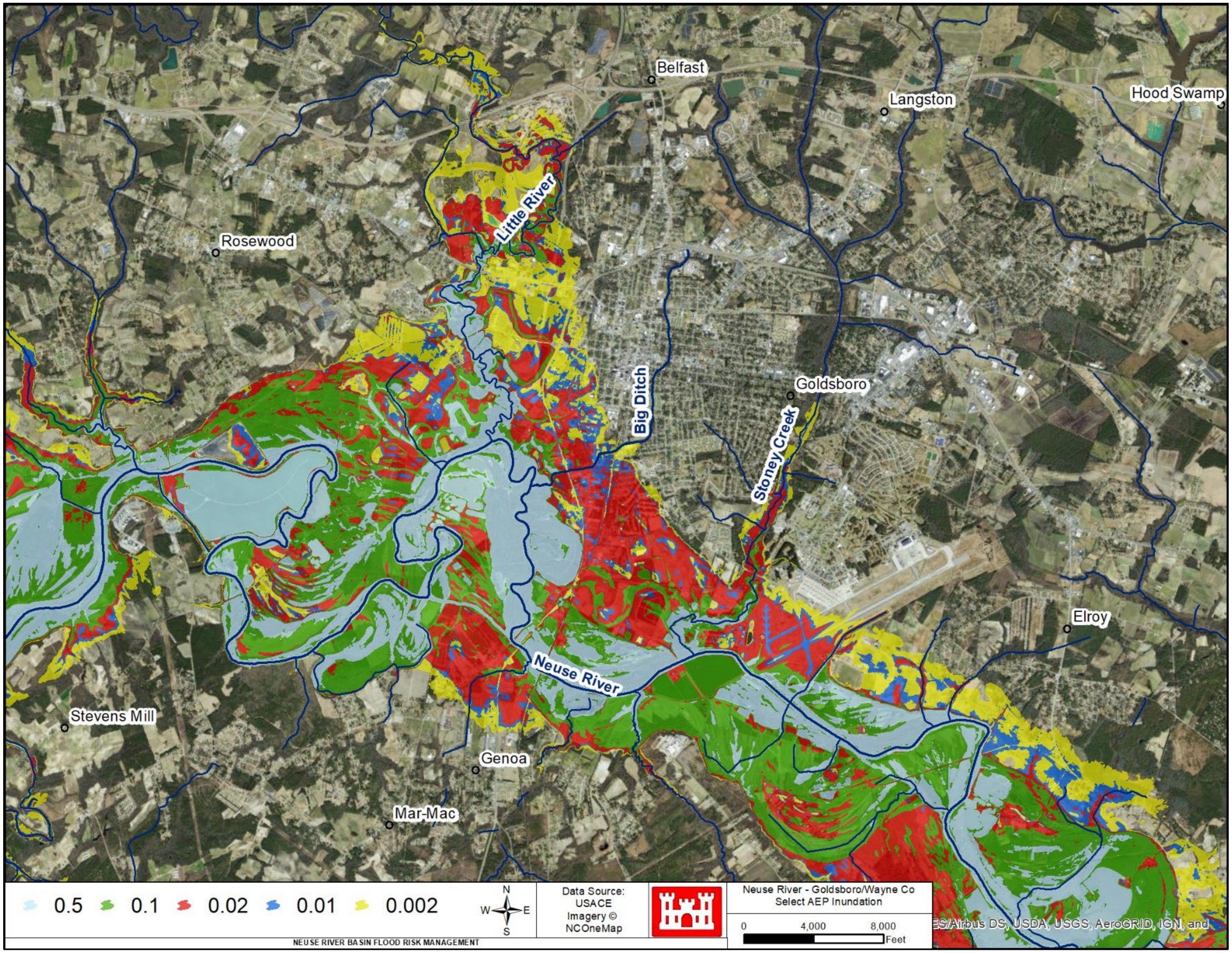

Figure 117. Neuse River Existing Conditions Modeled Inundation for Select Design Events near Goldsboro, Wayne County

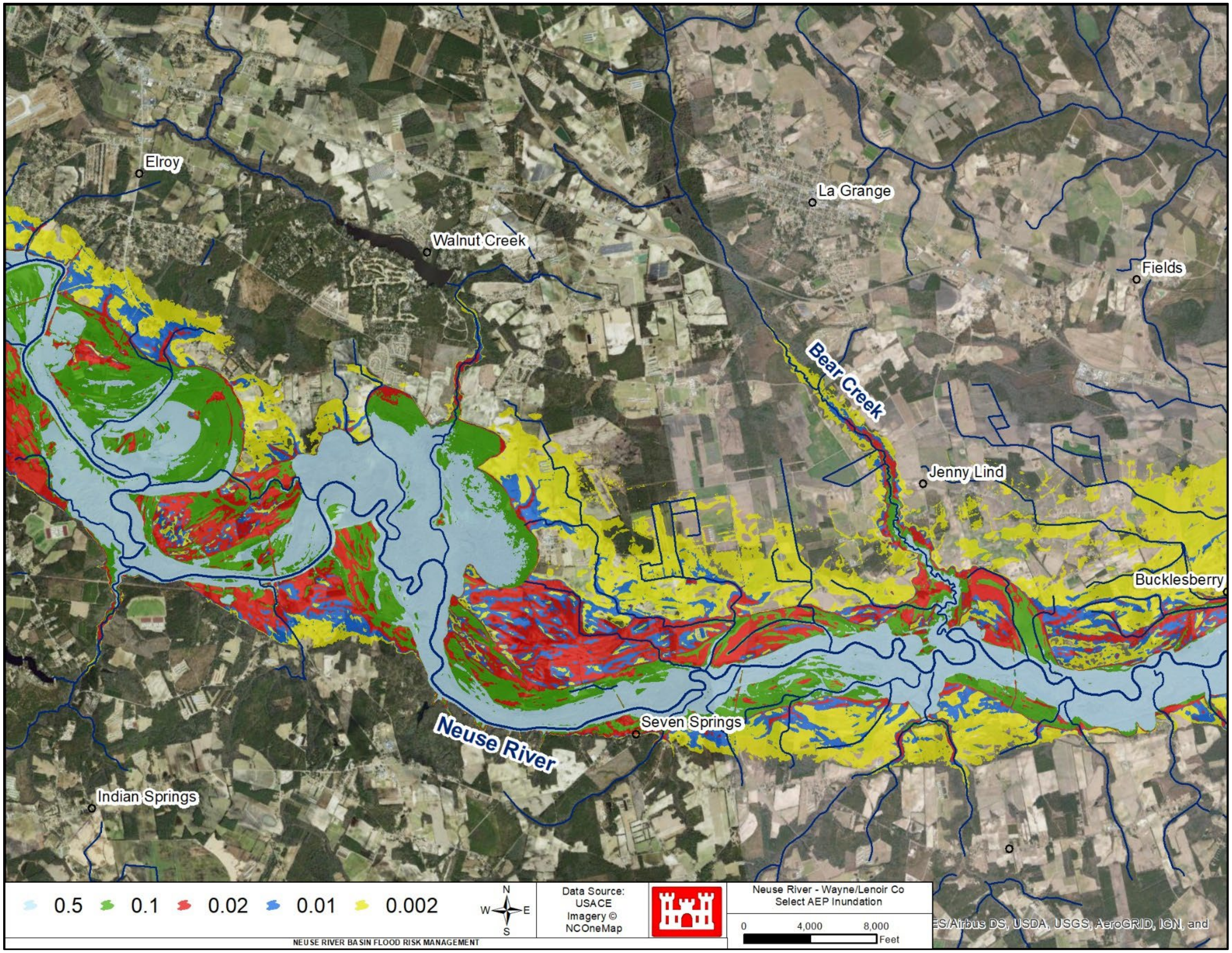

Figure 118. Neuse River Existing Conditions Modeled Inundation for Select Design Events in Rural Wayne and Lenoir Counties

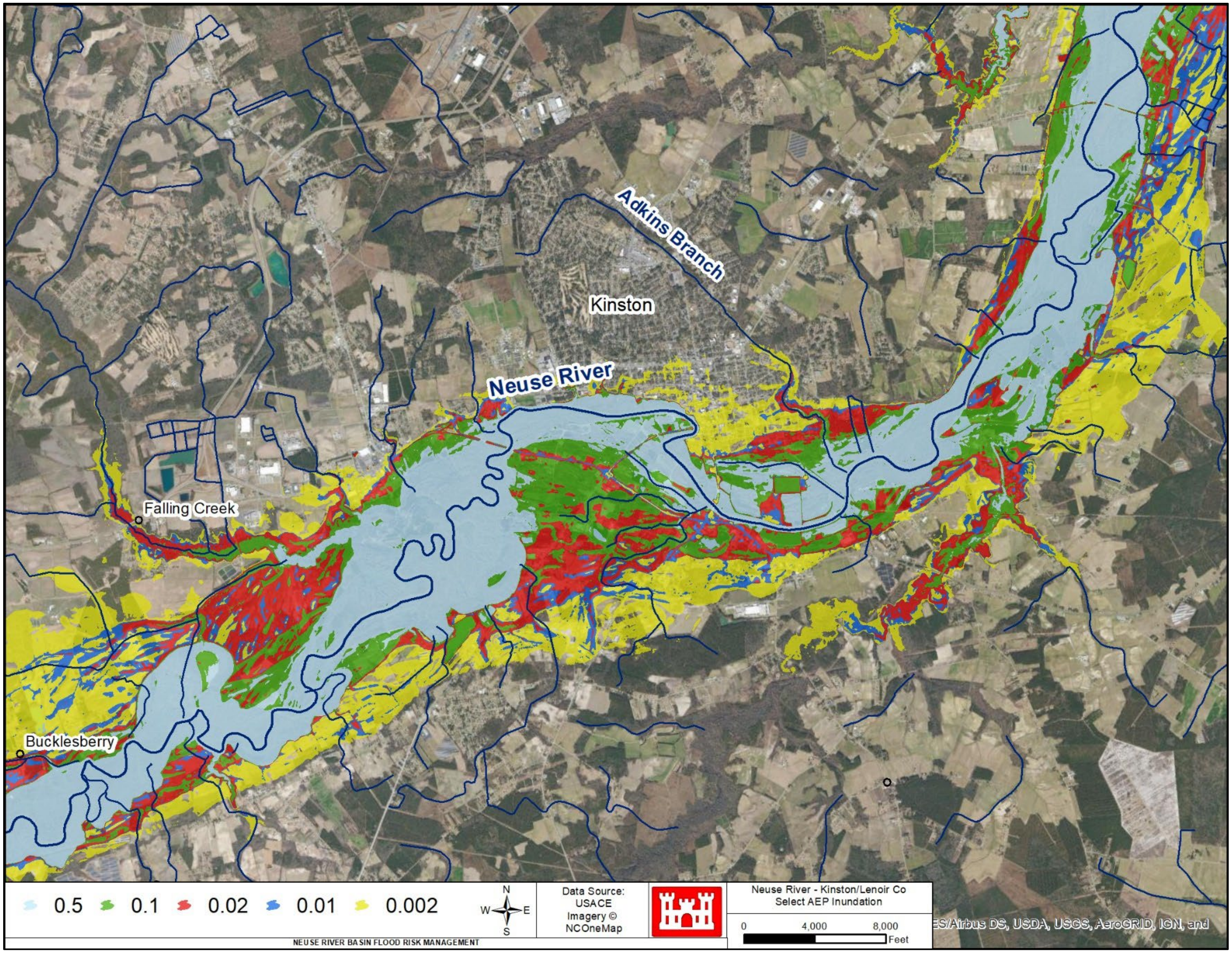

Figure 119. Neuse River Existing Conditions Modeled Inundation for Select Design Events near Kinston, Lenoir County

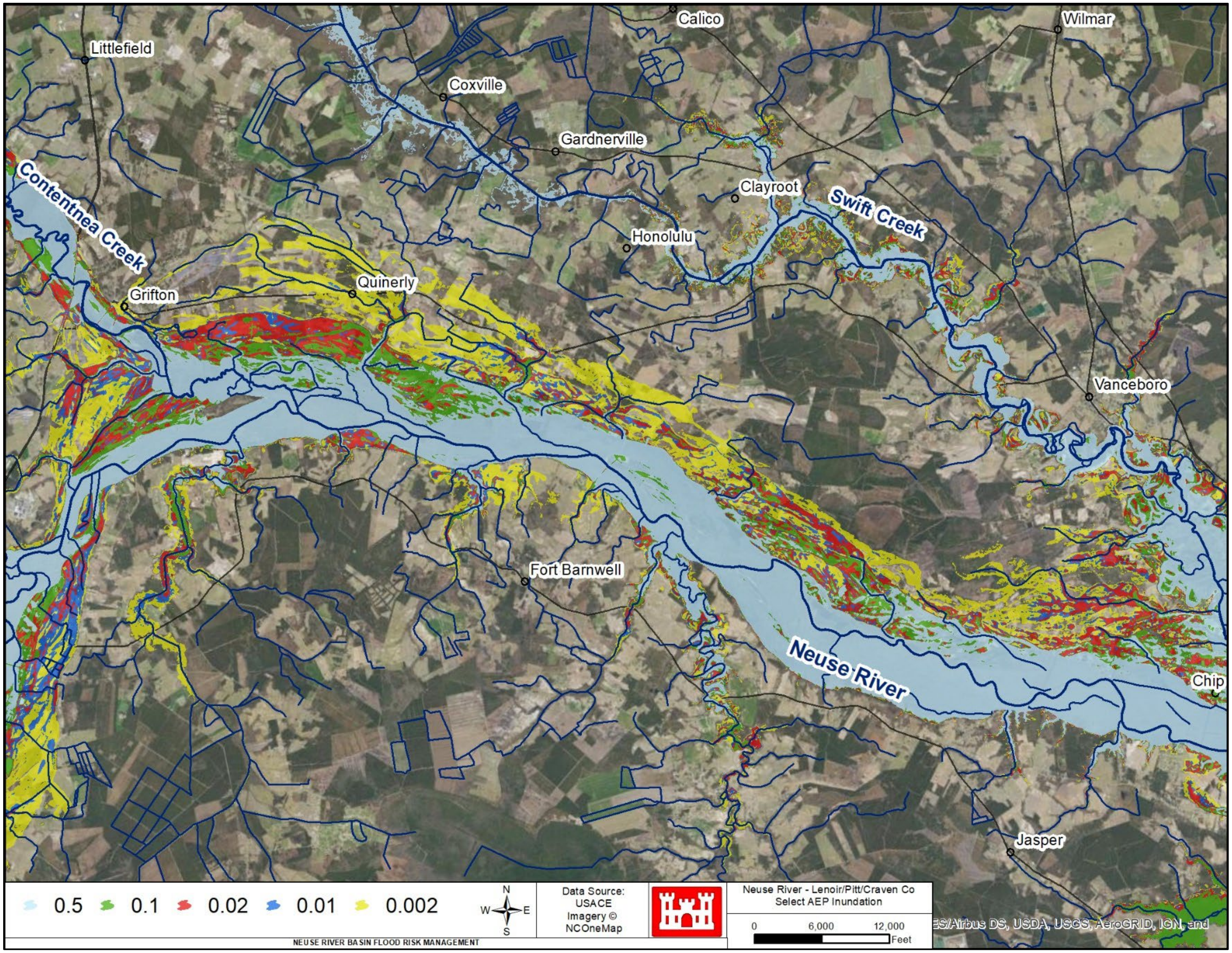

Figure 120. Neuse River Existing Conditions Modeled Inundation for Select Design Events in Rural Lenoir and Craven Counties

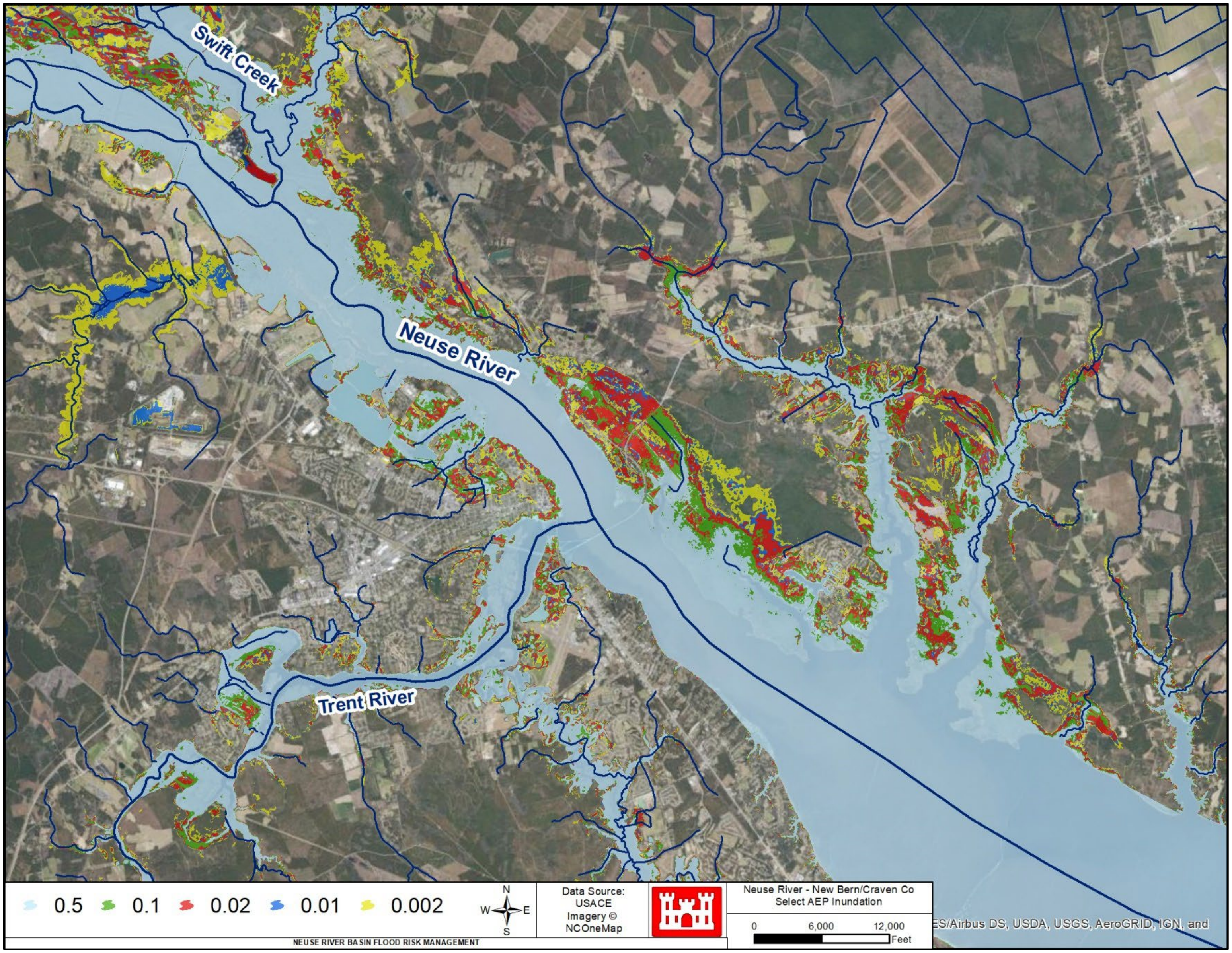

Figure 121. Neuse River Existing Conditions Modeled Inundation for Select Design Events near New Bern, Craven County

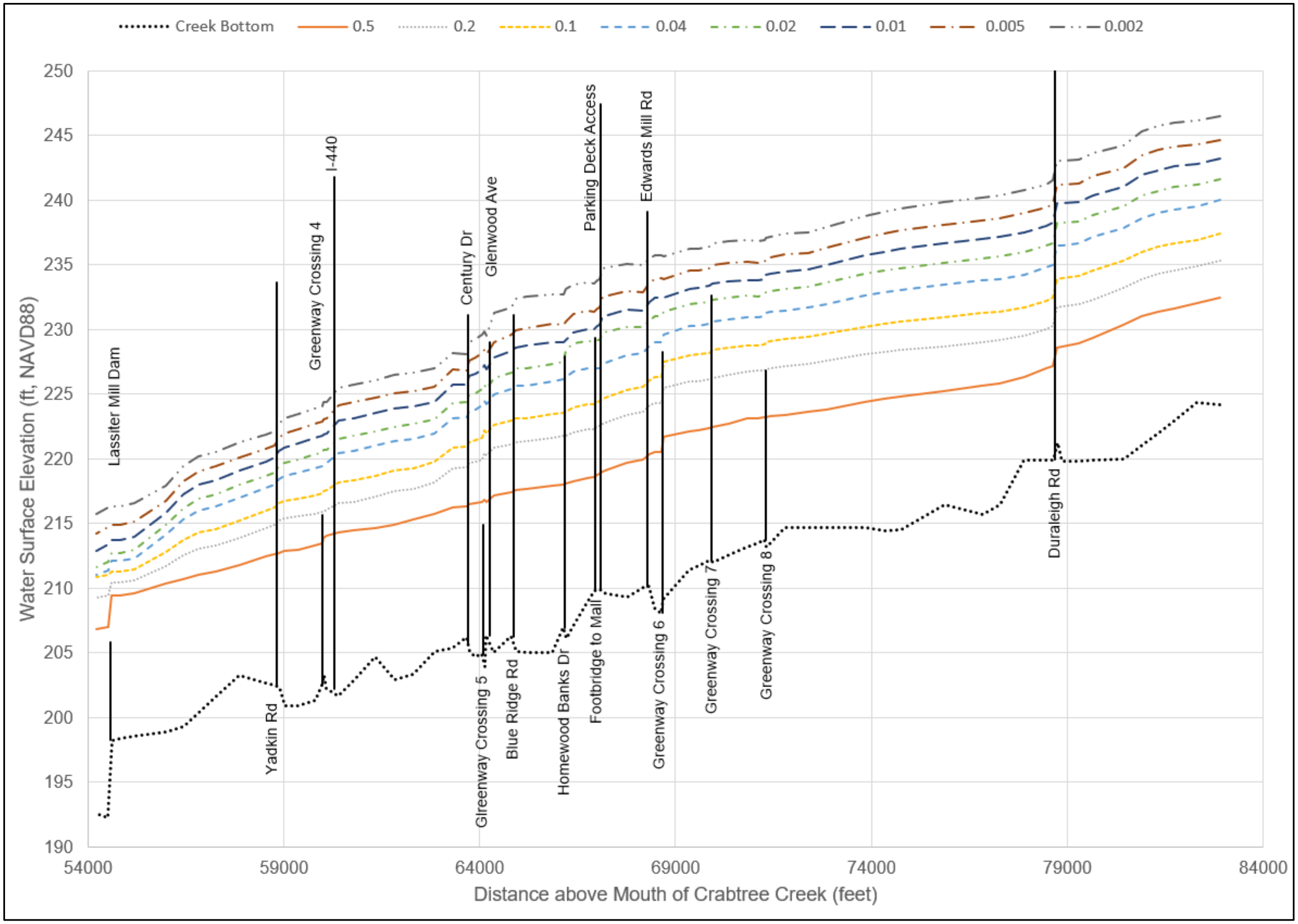

Figure 122. Crabtree Creek Existing Conditions Modeled Water Surface Profiles for Select Design Events from Ebenezer Church Rd to Lassiter Mill Rd

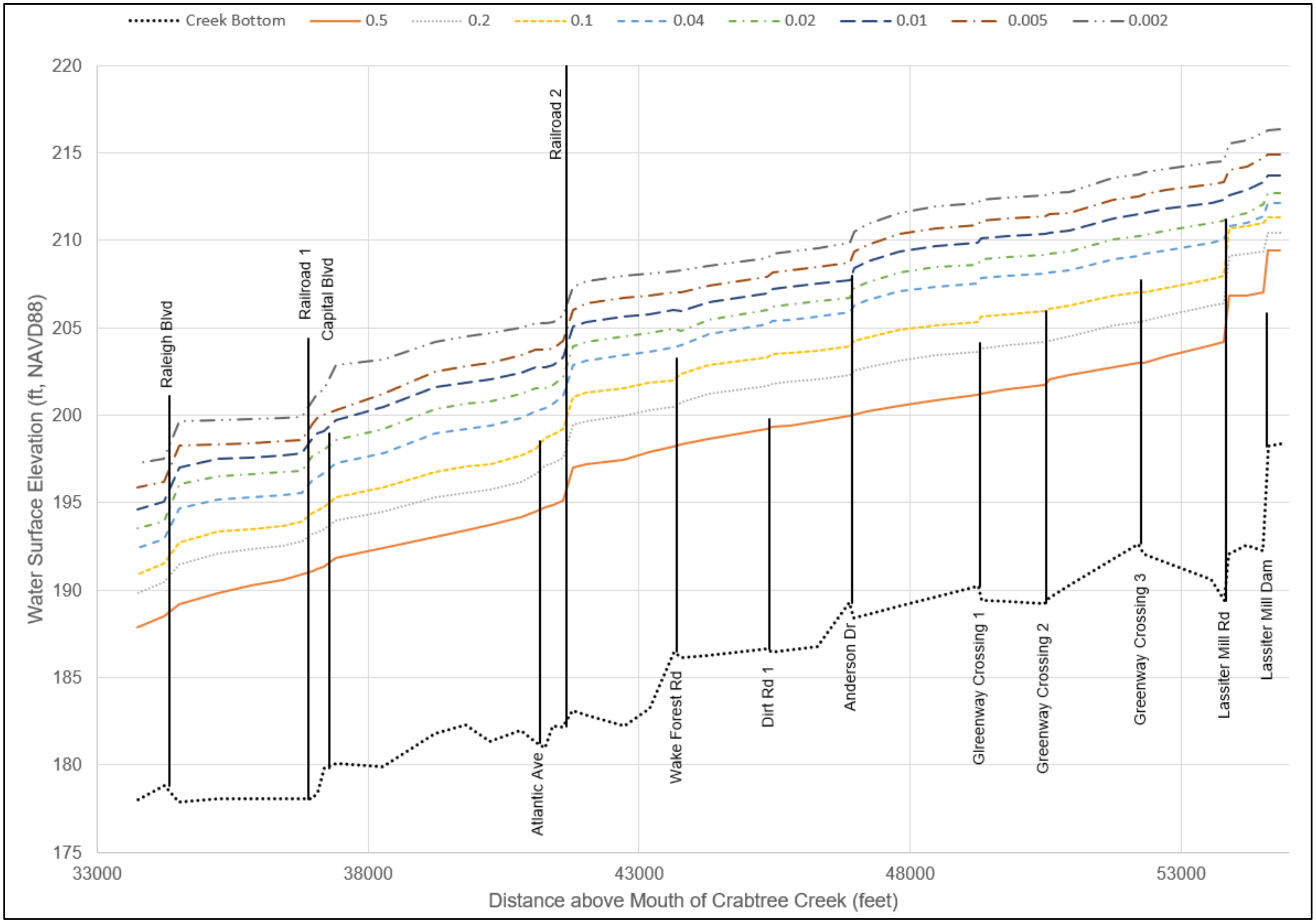

Figure 123. Crabtree Creek Existing Conditions Modeled Water Surface Profiles for Select Design Events from Lassiter Mill Rd to Raleigh Blvd

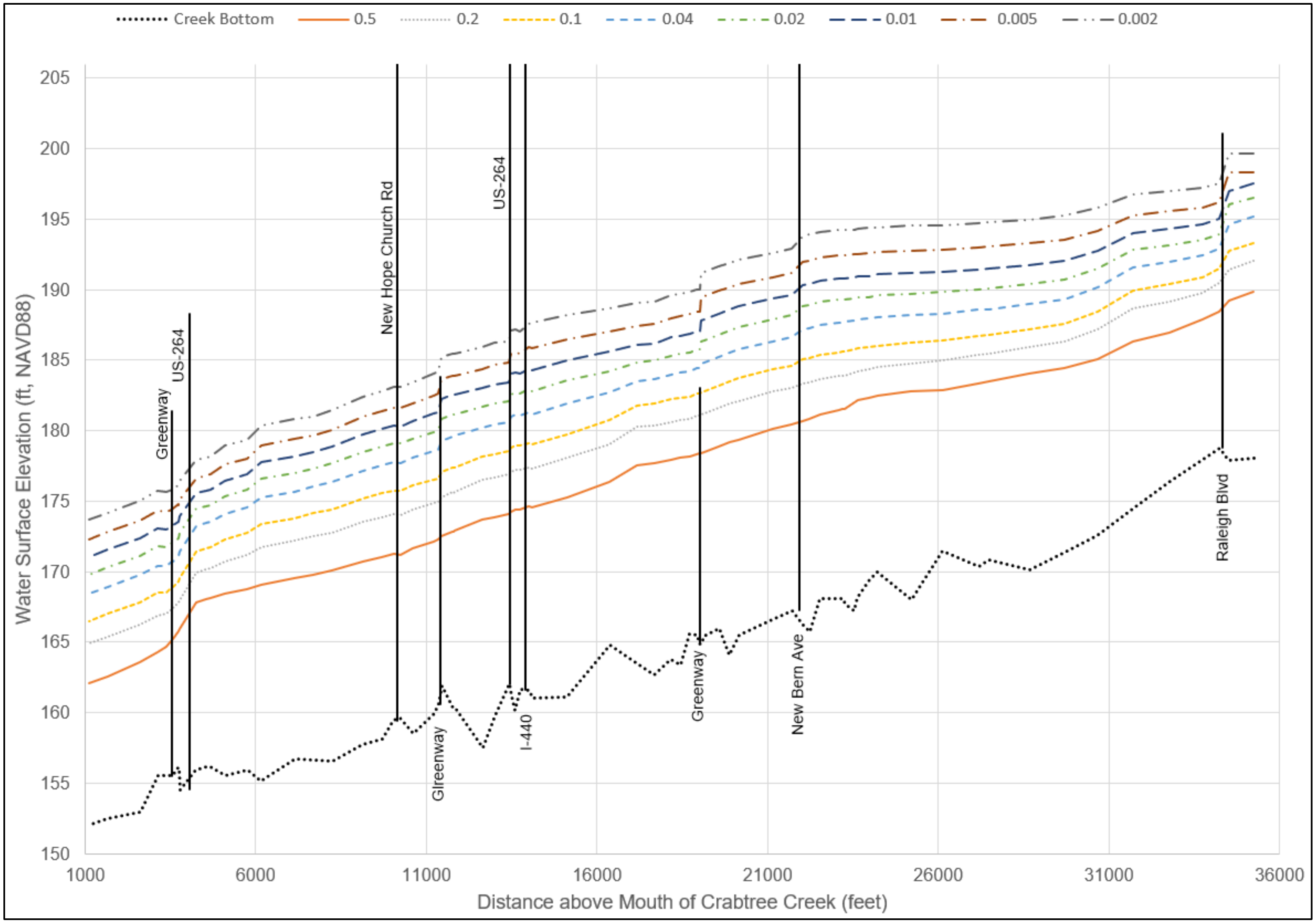

Figure 124. Crabtree Creek Existing Conditions Modeled Water Surface Profiles for Select Design Events from Raleigh Blvd to Mouth

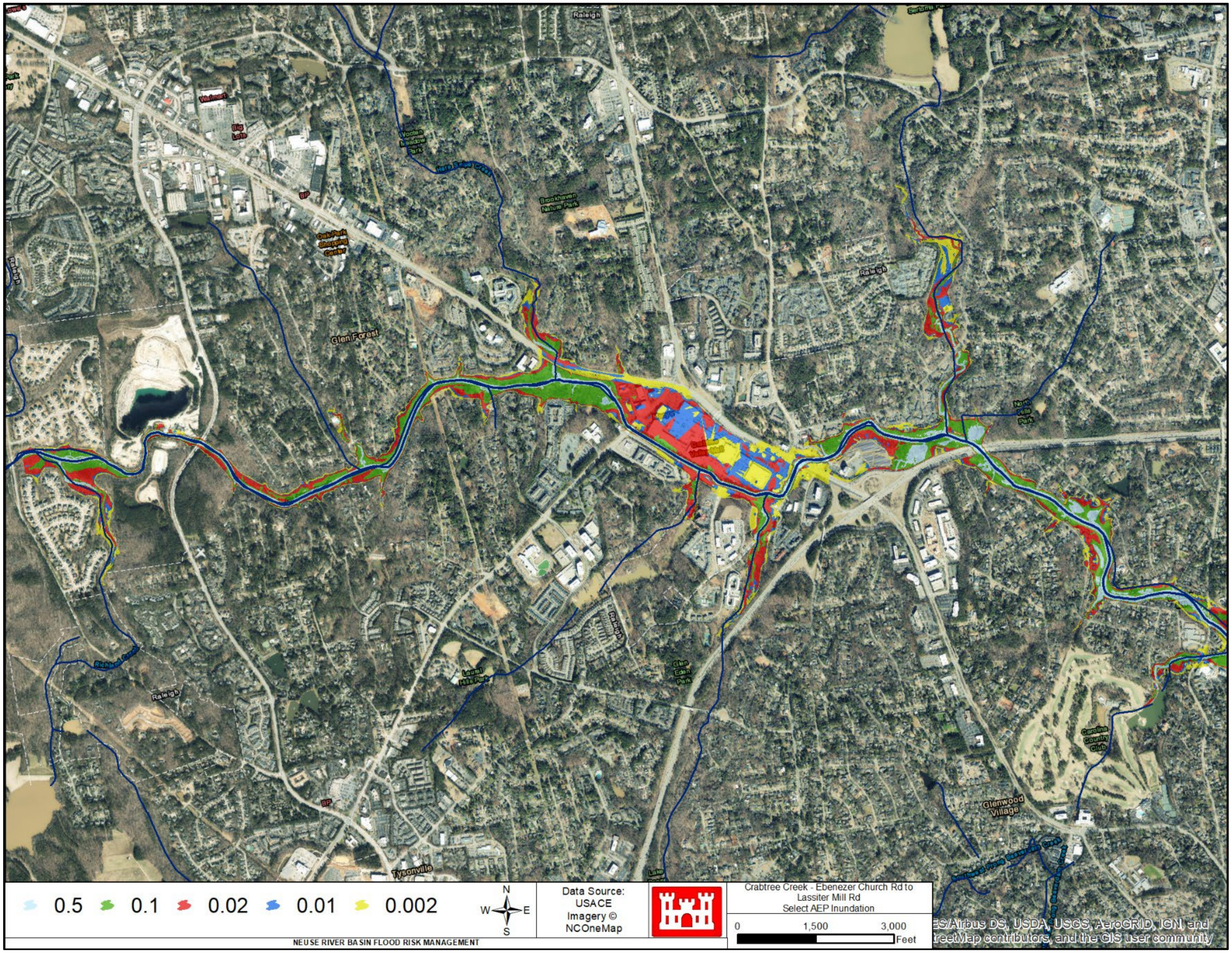

Figure 125. Crabtree Creek Existing Conditions Modeled Inundation for Select Design Events from Ebenezer Church Rd to Lassiter Mill Rd

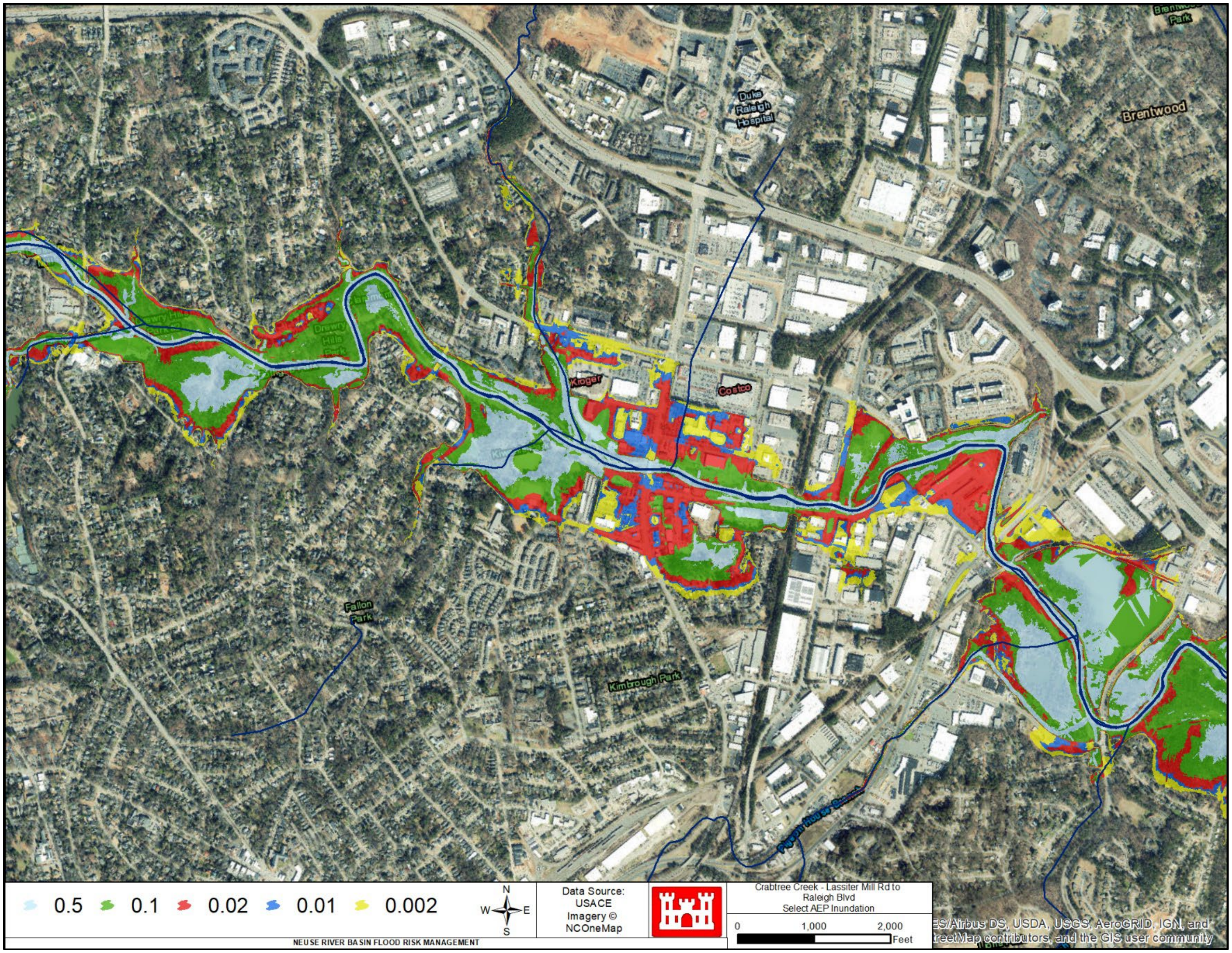

Figure 126. Crabtree Creek Existing Conditions Modeled Inundation for Select Design Events from Lassiter Mill Rd to Raleigh Blvd

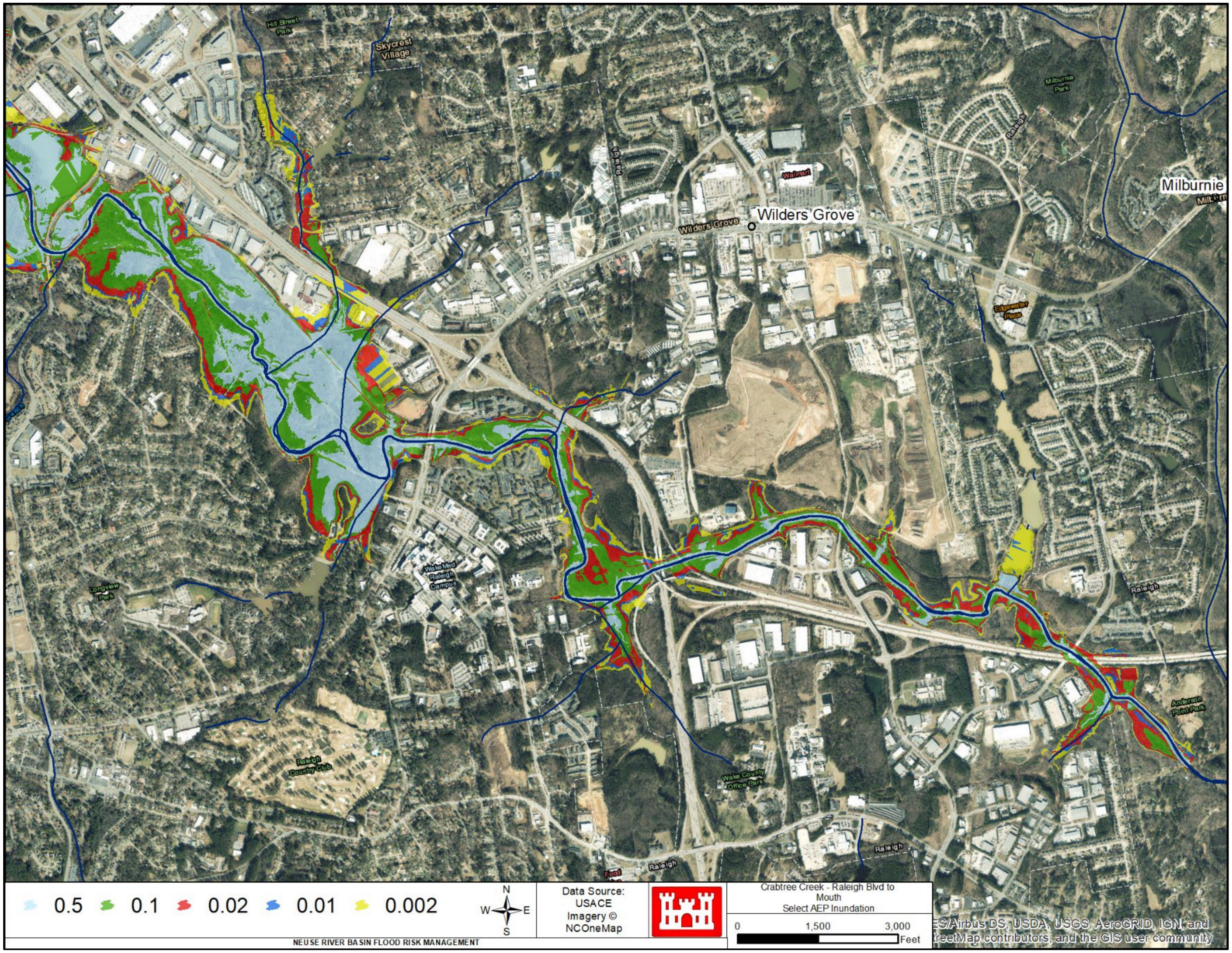

Figure 127. Crabtree Creek Existing Conditions Modeled Inundation for Select Design Events from Raleigh Blvd to Mouth

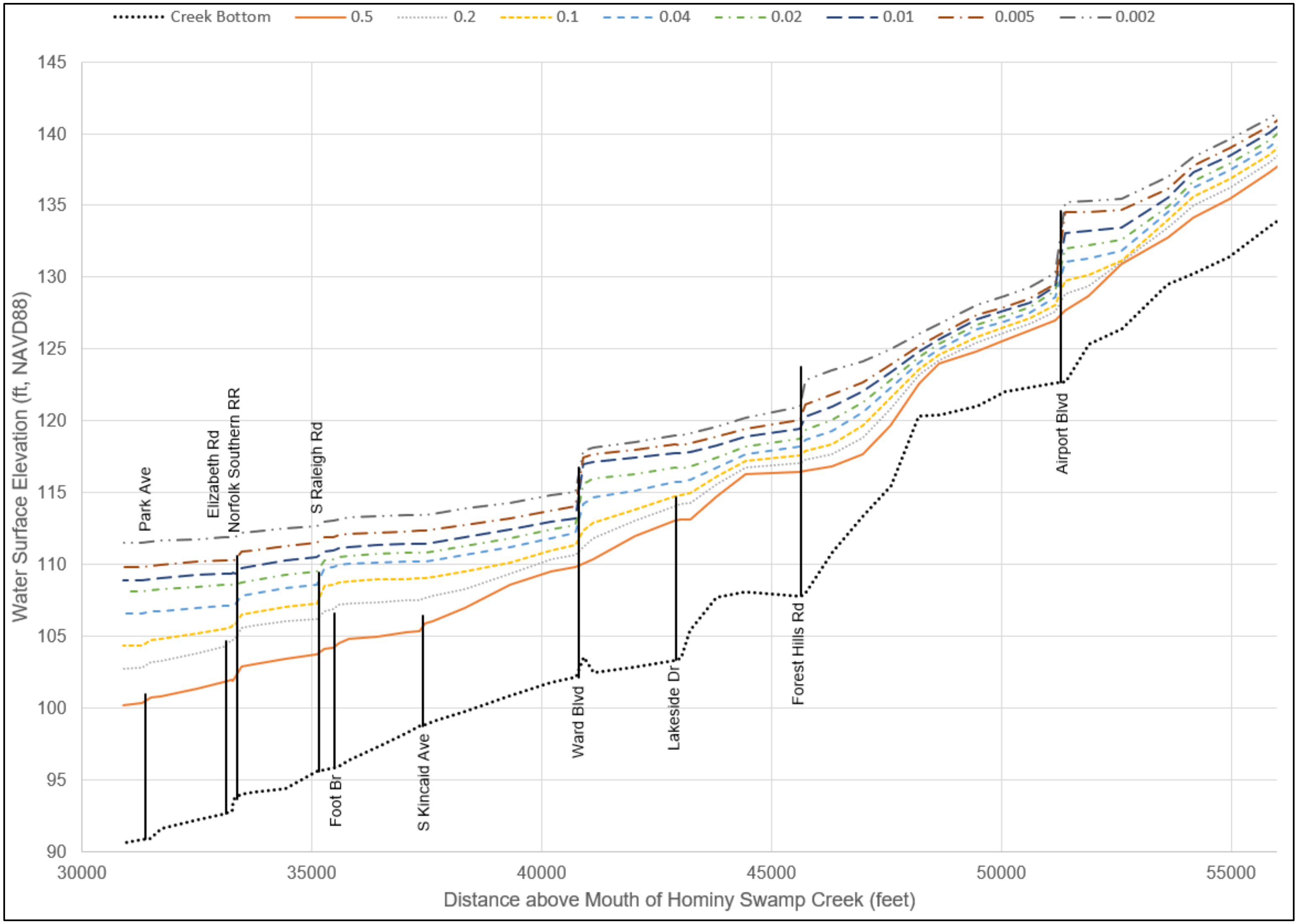

Figure 128. Hominy Swamp Creek Existing Conditions Modeled Water Surface Profiles for Select Design Events from Airport Blvd to Park Ave

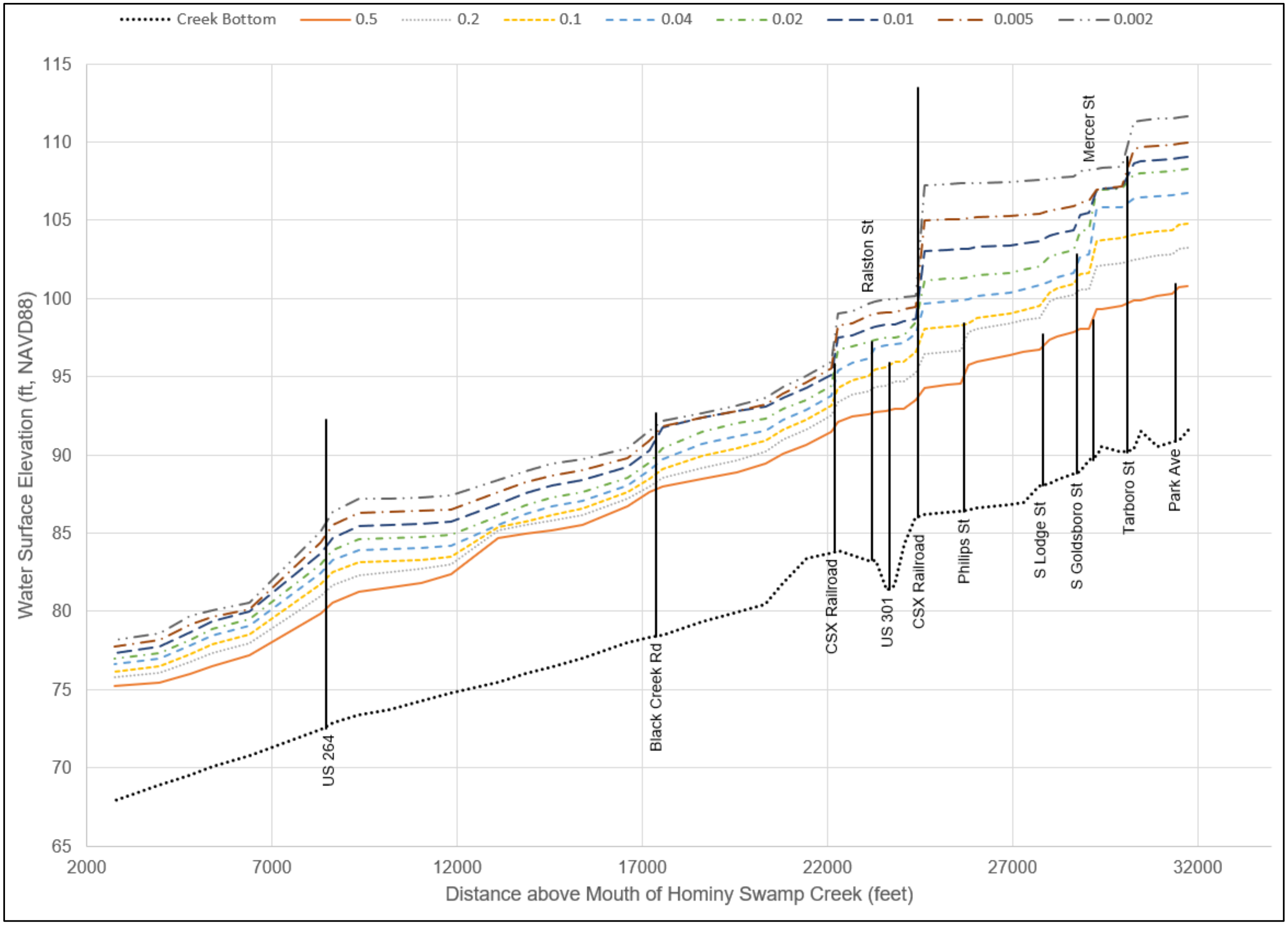

Figure 129. Hominy Swamp Creek Existing Conditions Modeled Water Surface Profiles for Select Design Events from Park Ave to Mouth

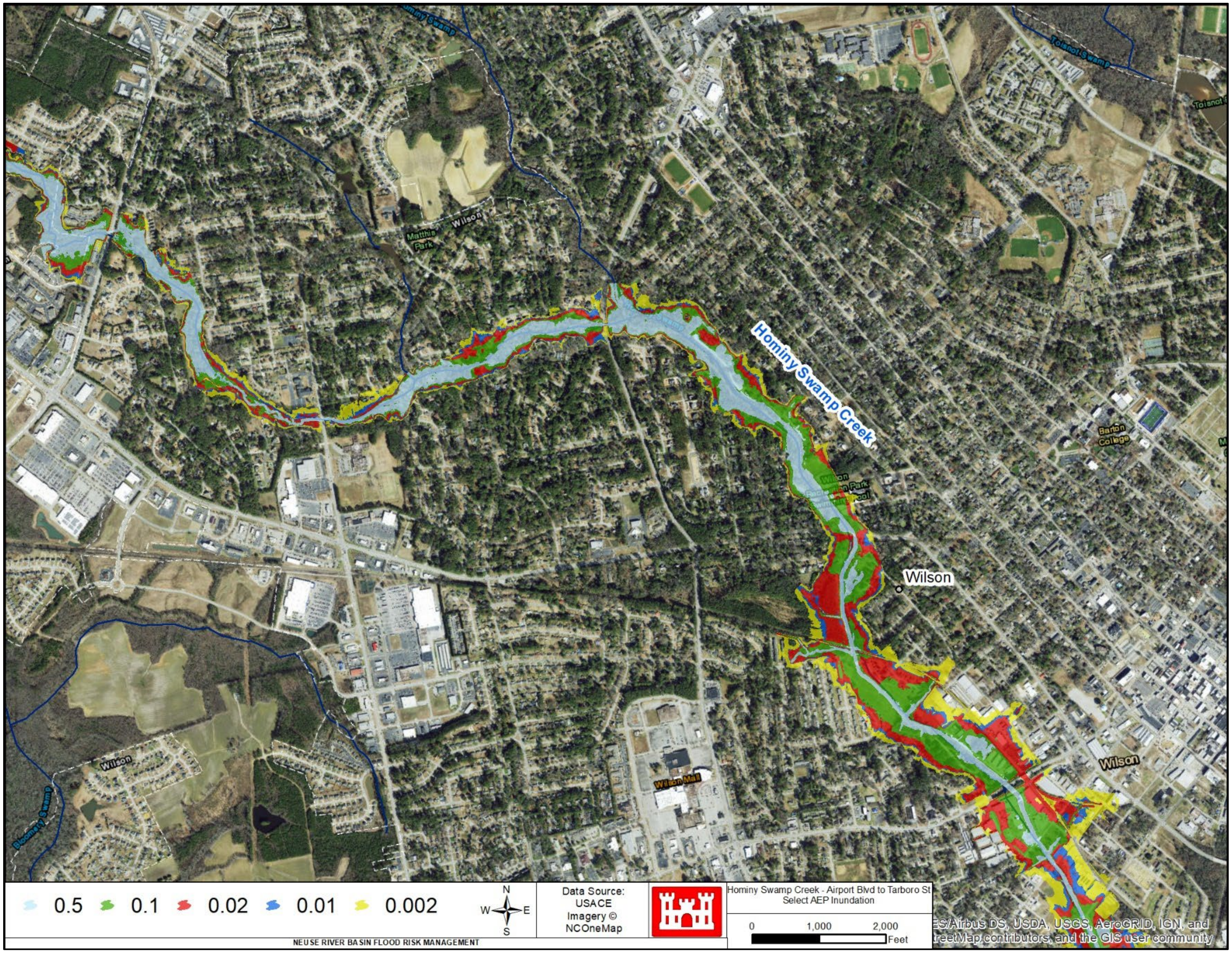

Figure 130. Hominy Swamp Creek Existing Conditions Modeled Inundation for Select Design Events from Airport Blvd to Tarboro St

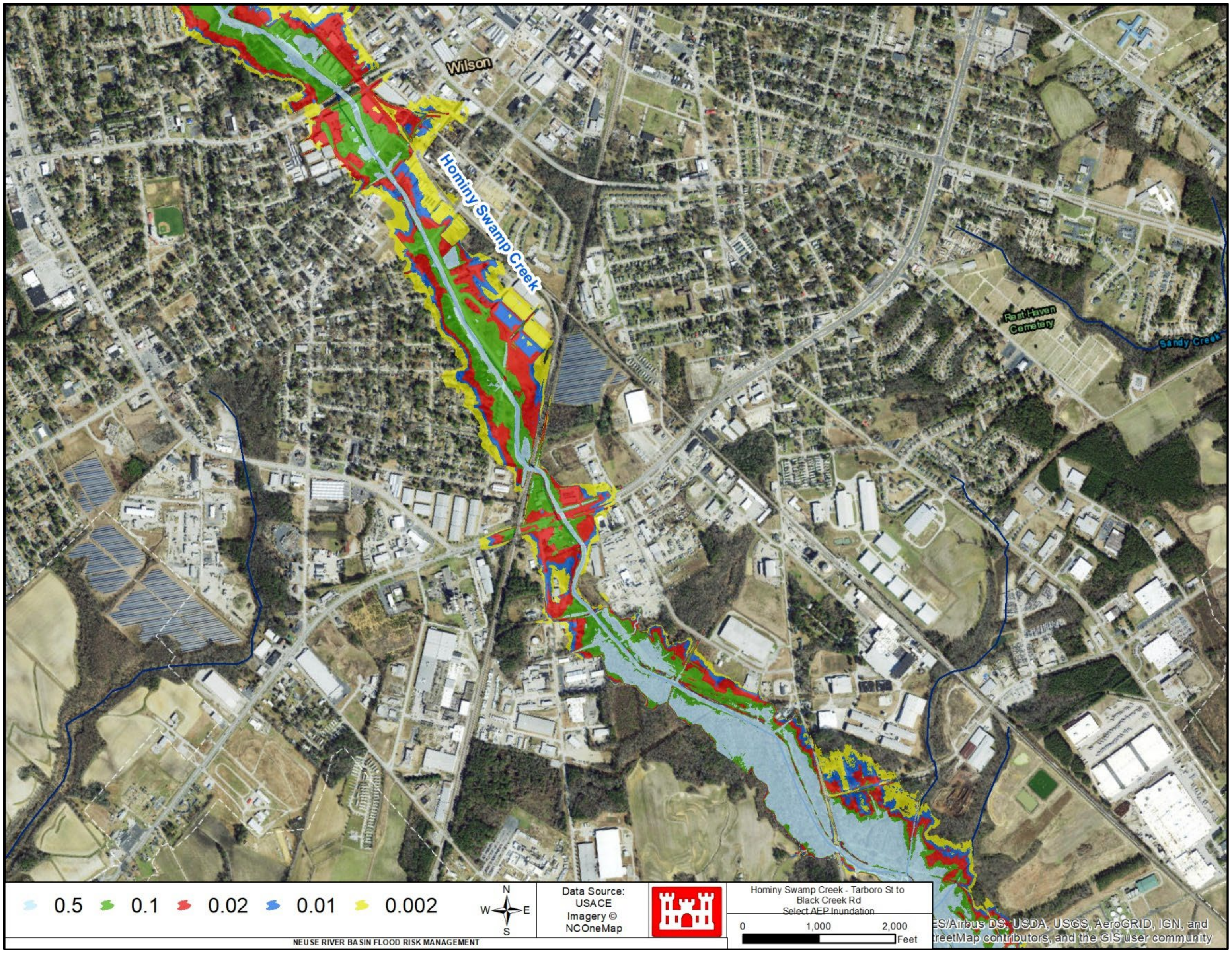

Figure 131. Hominy Swamp Creek Existing Conditions Modeled Inundation for Select Design Events from Tarboro St to Black Creek Rd

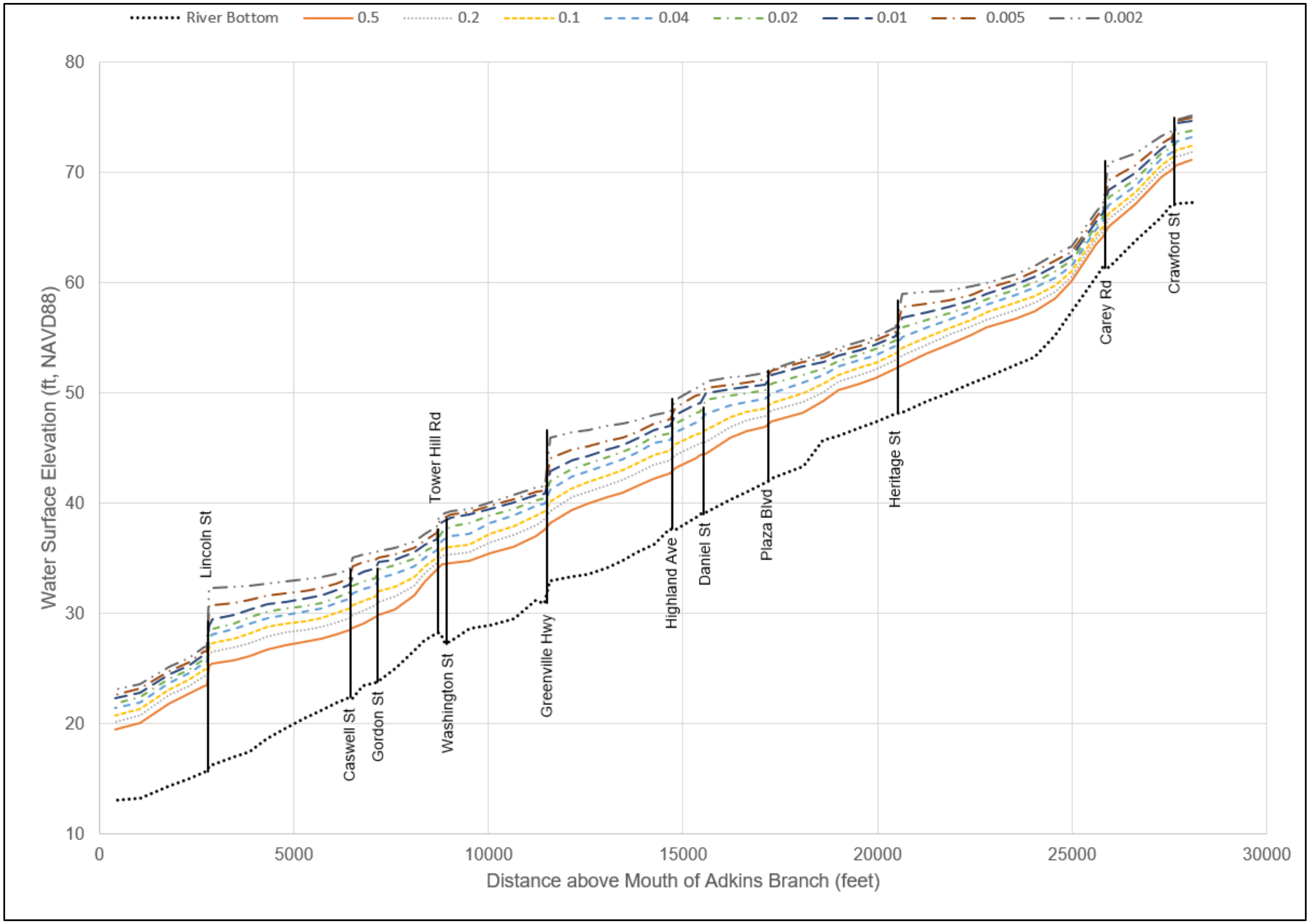

Figure 132. Adkins Branch Existing Conditions Modeled Water Surface Profiles for Select Design Events from Crawford St to Mouth

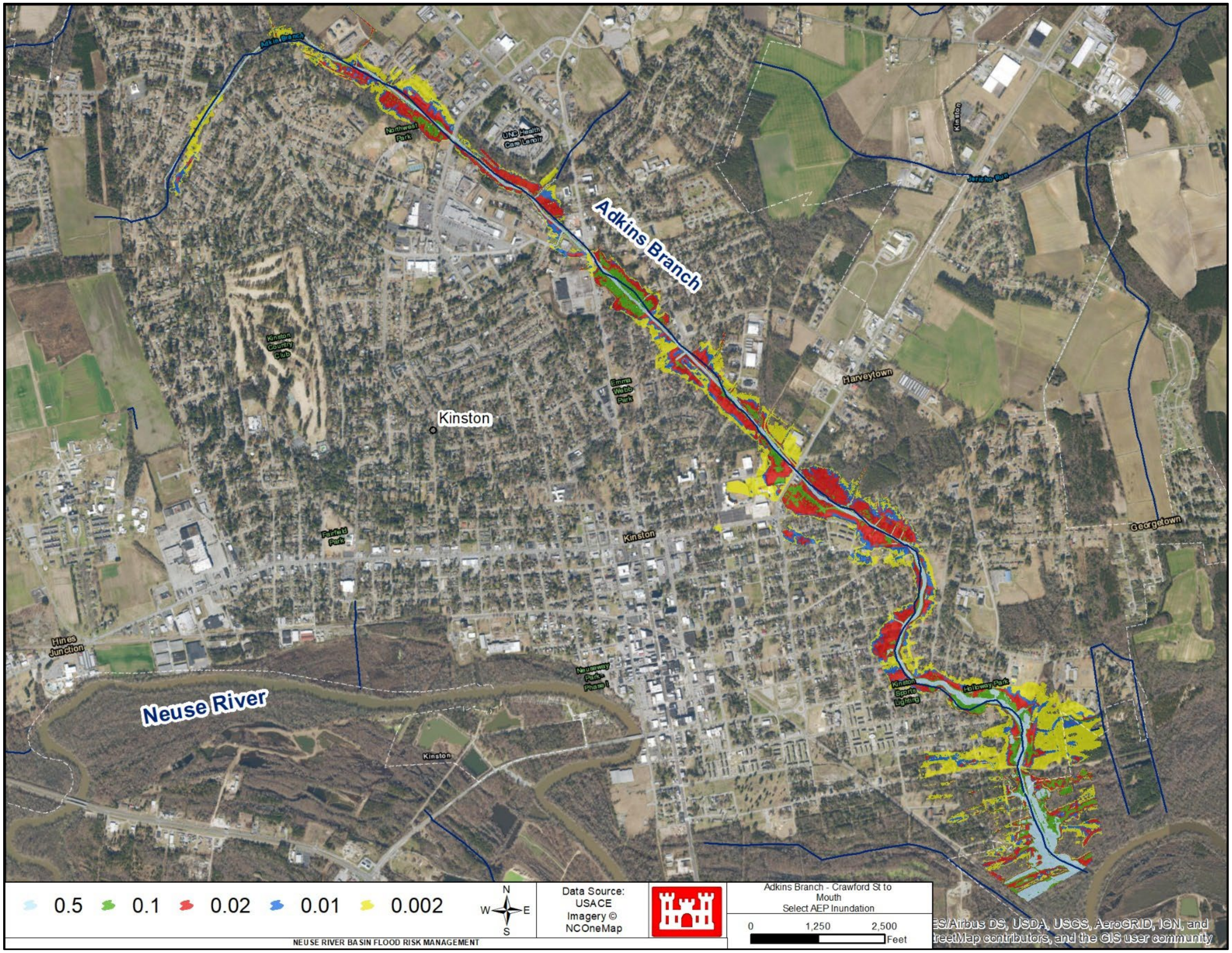

Figure 133. Adkins Branch Existing Conditions Modeled Inundation for Select Design Events from Crawford St to Mouth

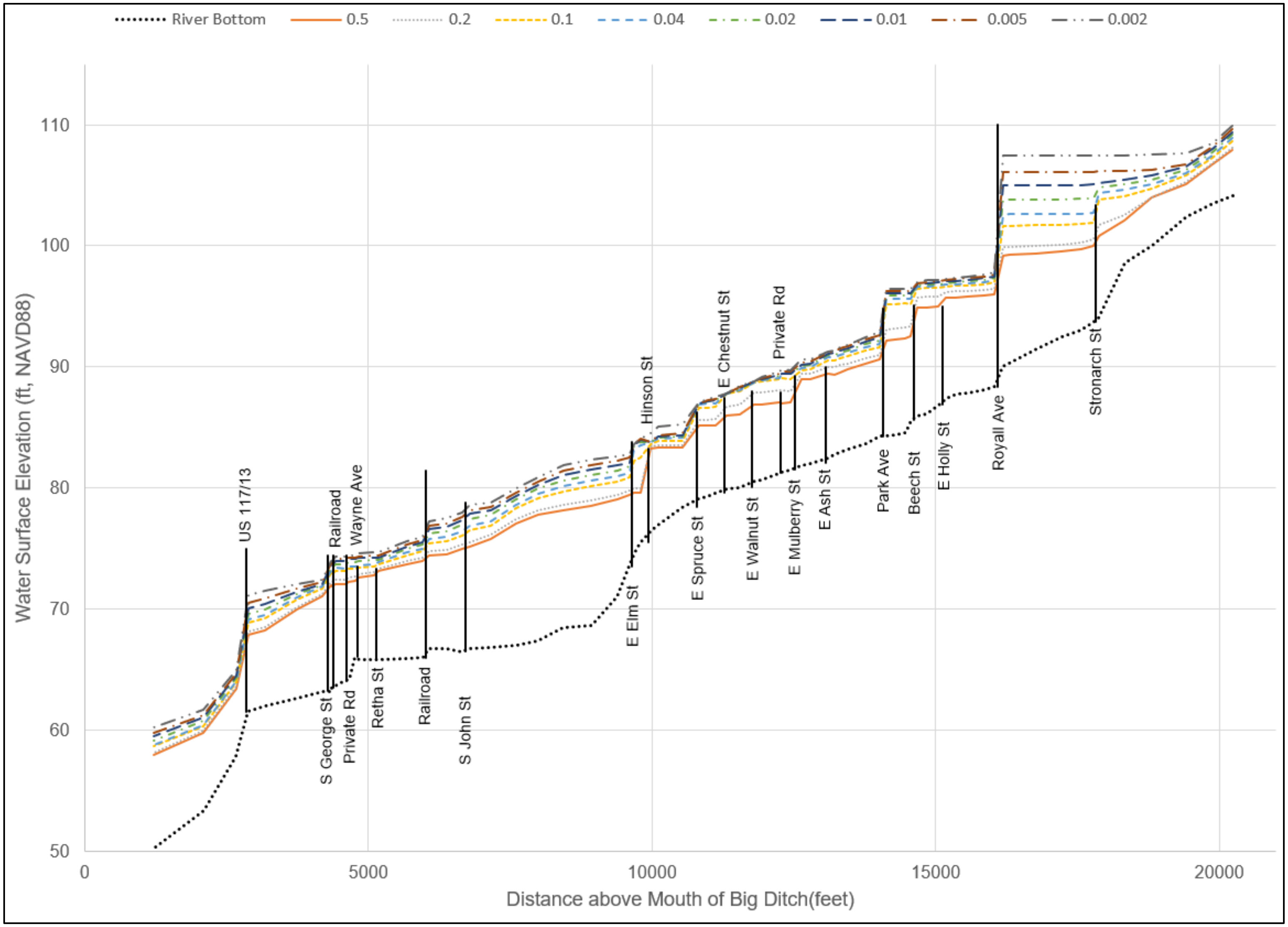

Figure 134. Big Ditch Existing Conditions Modeled Water Surface Profiles for Select Design Events from Stronarch St to Mouth

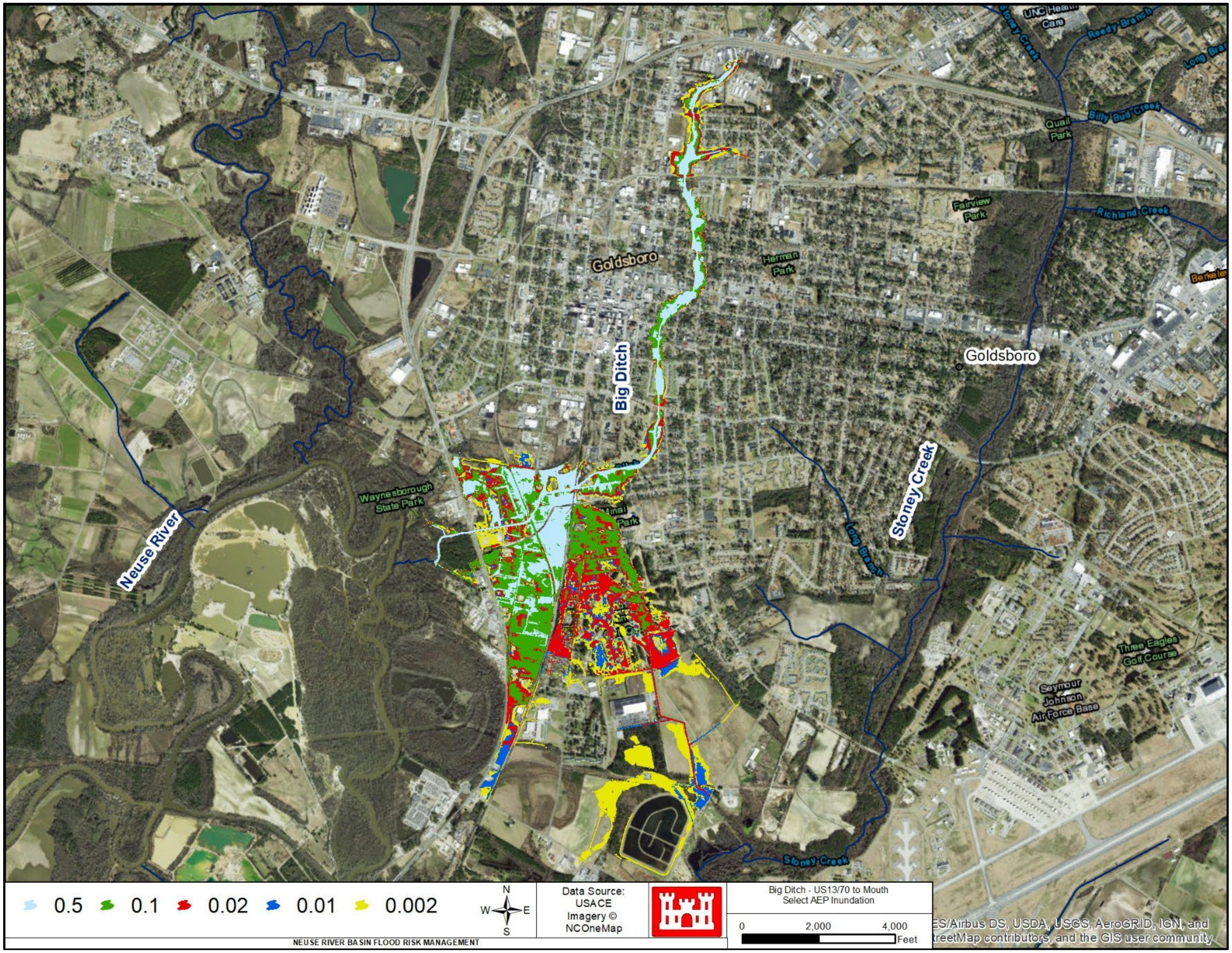

Figure 135. Big Ditch Existing Conditions Modeled Inundation for Select Design Events from Stronarch St to Mouth

5.3 Compound Flooding Considerations

Downstream boundary condition data described in Section 5.2.3 above assumed some dependency in water surface elevations between riverine flows (Neuse River) and estuary stage (Pamlico Sound). Fundamentally, the possibility exists for both estuarine and riverine flooding to occur at the same time for the most downstream portions of the Neuse River basin study. Extreme winds and elevated tides that originate from coastal storms can propagate across the Pamlico Sound estuary and impede the Neuse 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 mouth of the Neuse 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 (NCHRP, 2010). The Neuse River and Pamlico Sound riverine/estuary interaction shares some similarities with a riverine-only scenario, but those engineering methodologies should be used with caution and acknowledgement of uncertainties.

Several 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. Due to study limitations, these analyses were conducted under existing conditions and may not fully capture the effects of compound flooding under future conditions.

5.3.1 Correlation Analysis

A correlation analysis, coincident in time, was conducted to investigate the relationship between gaged data within the Neuse river and Pamlico Sound using the Hydrologic Engineering Center’s Statistical Software Package (HEC-SSP). This analysis would provide a degree of linear correlation between data sources with a value of 0.0 indicating no correlation and 1.0 indicating 100-percent correlation. The primary and secondary sources for riverine flows and estuary conditions was based on the USGS Neuse River near Fort Barnwell station (02091814) and the Beaufort, Duke Marine Lab, NC NOAA tide buoy (8656483), respectively. Annual peak flows at the Fort Barnwell USGS station are listed in Table 66 and water surface elevations were extracted from the Beaufort, NC NOAA station period of record (1977-2022), shown in Figure 136.

Table 66. USGS Neuse River near Fort Barnwell (02091814) Annual Peak Flows

Day	Month	Year	Peak Discharge (cfs)
11	Oct	1996	14,500
6	Feb	1998	24,300
20	Sep	1999	57,200
23	Oct	1999	24,300
10	Apr	2001	11,500
1	Feb	2002	10,100
17	Apr	2003	18,600
21	Dec	2003	12,200
27	Mar	2005	9,110
4	Sep	2006	16,200
24	Nov	2006	24,400
13	Apr	2008	9,660
11	Mar	2009	11,600
13	Feb	2010	17,400
30	Aug	2011	16,600
31	Mar	2012	7,470
13	Jul	2013	18,700
6	May	2014	15,200
21	Jan	2015	18,500
12	Feb	2016	21,700
15	Oct	2016	49,400
23	Sep	2018	40,100
18	Dec	2018	17,300
15	Feb	2020	19,500
19	Nov	2020	29,300

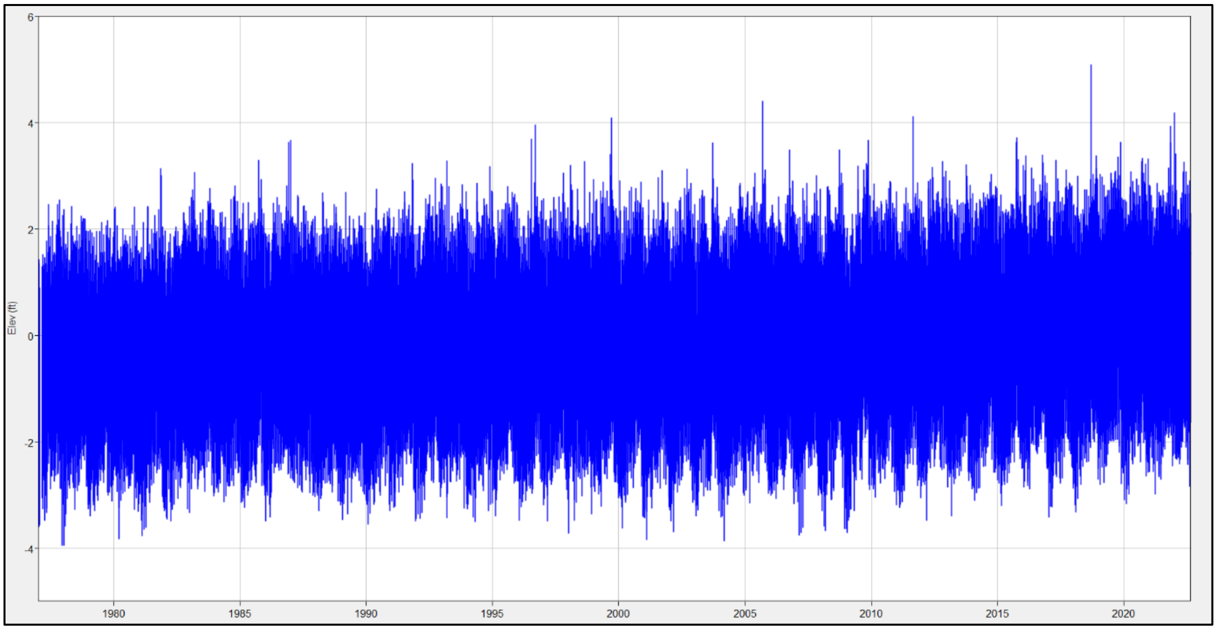

Figure 136. Beaufort, Duke Marine Lab, NC NOAA tide buoy (8656483) Hourly Water Surface Elevations

The Beaufort, NC NOAA water surface elevation data was extracted assuming two different timesteps for which correlation analyses would be conducted, hourly and daily. A total of 22 years (1998-2020) of overlapping data was shared across the two sources. A correlation analysis of annual peak flows at Fort Barnwell and the hourly-derived duration curve at Beaufort resulted in a correlation coefficient of -0.063. A correlation analysis based on a daily duration curve at Beaufort resulted in a correlation coefficient of 0.392. Per EM 1110-2-1415, datasets can be considered effectively independent when the absolute value of the correlation coefficient is less than 0.4. Notably, a related correlation analysis was provided by HEC for the nearby Tar-Pamlico Watershed that shared similar hydraulic characteristics with the Neuse River watershed. The HEC correlation analysis was based on annual peak flows from the USGS Tar River at Greenville, NC station (02084000) and hourly Beaufort, NC NOAA data. The correlation coefficient calculated for the Tar-Pamlico watershed was -0.032. The following explanation was provided by HEC:

> In this case, it appears the annual maximum peak discharge for the Tar River at Greenville, NC has no relationship to the water level within the Pamlico Sound. This makes physical sense since the Pamlico Sound is a much larger body of water, compared to the Tar River, that can drain/fill through many connections to the Atlantic Ocean. This is very valuable information for a flood risk management study, as the assumption of independence greatly simplifies a coincident frequency analysis that is likely the next step in the project (https://www.hec.usace.army.mil/confluence/sspdocs/sspexamples/latest/correlation-analysis-examples/coincident-in-time-tar-river).

Although a correlation coefficient of 0.392, based on Beaufort, NC NOAA daily data was technically less than the EM-suggested threshold of 0.4, correlation this high between two variables is too high to ignore, so potential correlation was investigated.

5.3.2 Coincident Frequency Analysis

Based on the different degrees of correlation between Neuse River flows and Pamlico Sound estuary water surface elevations described in the preceding section, a series of coincident frequency analyses were conducted. The weak correlation (-0.063 coefficient) between Neuse River flows and hourly Pamlico Sound water surface elevations was investigated by performing eight coincident frequency analyses in HEC-SSP. The analyses define three variables to represent flows along the Neuse River and water levels within the Pamlico Sound (variables A and B), and stage near the mouth of the Neuse River (variable C). No one particular location of variable C would fully capture effects of compound flooding, therefore, multiple analyses with different variable C locations were investigated. The main assumptions for these analyses were that variables A and B were independent, and only one frequency curve for variable A was established. Another critical choice in defining the analysis was determination of which dataset be relatively more influential in creation of peak water surface elevations. Due to general uncertainties and limited scope of this compound flooding assessment, both riverine and estuary datasets were treated as more influential in separate analyses to provide a range of coincident frequency effects. Lastly, Variable A data type was based on annual peak frequency analysis and Variable B was based on a duration analysis. Variable combinations for the HEC-SSP coincident frequency analyses are listed in Table 67.

Table 67. HEC-SSP Coincident Frequency Analyses Variable Combinations

HEC-SSP Coincident Frequency Analysis	Variable A (more Influential)	Variable B (Less Influential	Variable C Location (HEC-RAS XS)
Configuration #1	Ft Barnwell Flow	Beaufort Stage	6.87
Configuration #2	Beaufort Stage	Ft Barnwell Flow	6.87
Configuration #3	Ft Barnwell Flow	Beaufort Stage	10.08
Configuration #4	Beaufort Stage	Ft Barnwell Flow	10.08
Configuration #5	Ft Barnwell Flow	Beaufort Stage	13.053
Configuration #6	Beaufort Stage	Ft Barnwell Flow	13.053
Configuration #7	Ft Barnwell Flow	Beaufort Stage	16.248
Configuration #8	Beaufort Stage	Ft Barnwell Flow	16.248

Each configuration required a response table that corresponds each unique combination of Fort Barnwell flow and Beaufort stage to a water surface elevation (Variable C). Response tables were developed by extracting a portion of the study HEC-RAS Neuse River mainstem model that began just upstream of the USGS Fort Barnwell station and ended approximately 24 river miles downstream of the City of New Bern, NC. The steady HEC-RAS model used flow data based on either a general frequency analysis or duration analysis of the USGS Fort Barnwell station. The downstream boundary condition was set to a known water surface elevation based on either a general frequency analysis or duration analysis of the Beaufort, NC NOAA tide buoy hourly dataset. HEC-RAS results at the cross sections listed in Table 67 were used to populate the eight unique response tables. Cross section locations are shown in Figure 137.

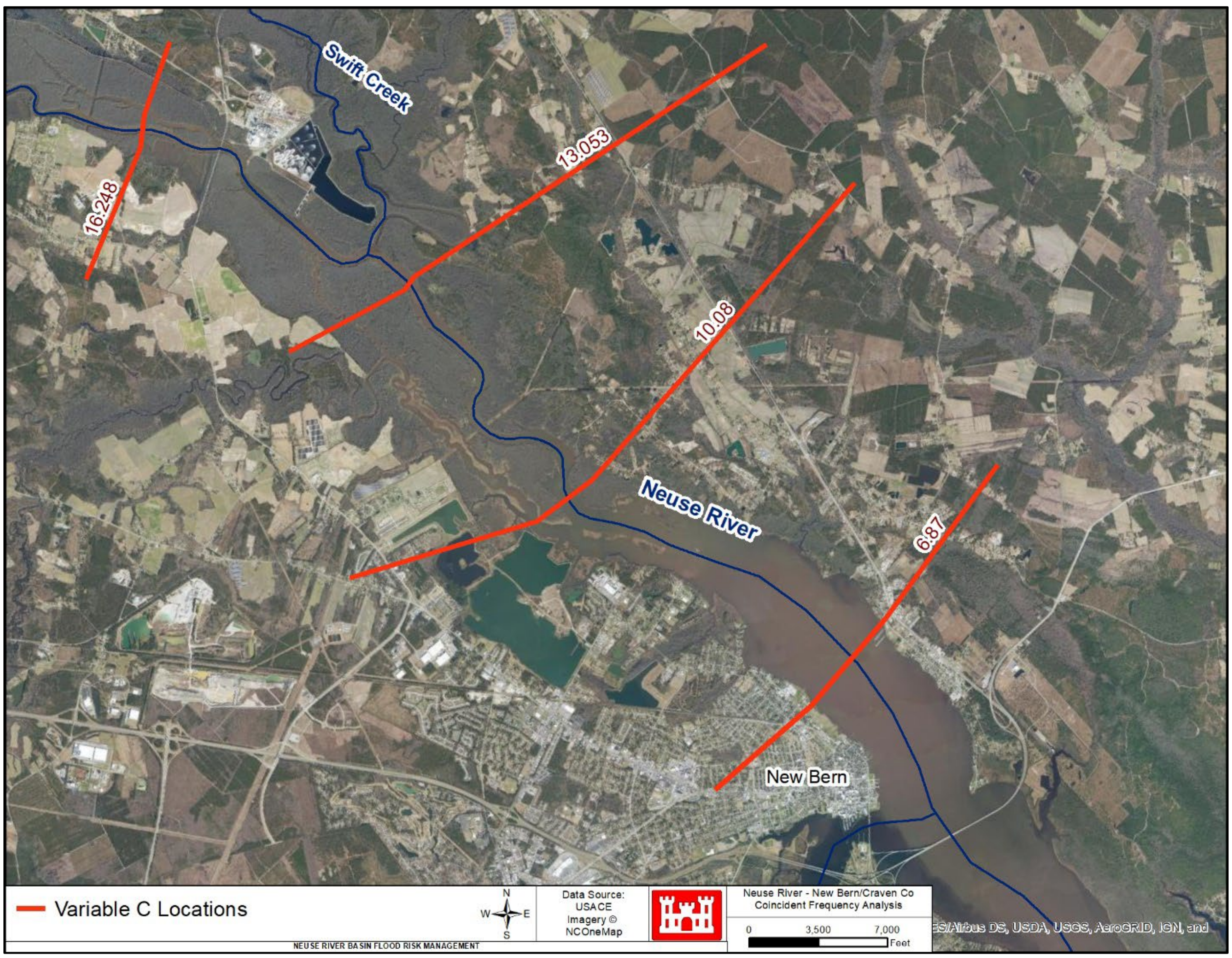

Figure 137. HEC-RAS Cross Sections for HEC-SSP Coincident Frequency Analysis Variable C Location

Coincident frequency analysis results of the eight configurations are shown in Figure 138 through Figure 141. The exceedance probability at which the two configuration lines cross indicate the extent of influence between the two flood sources (Neuse River/Pamlico Sound). Based on Figure 138, the two lines cross at an exceedance probability less than approximately 0.2%. This indicates that all exceedance probabilities >= ~0.2% produce water surface elevations that are more influenced by Configuration #2. It is assumed that peak water surface elevations for any location downstream of HEC-RAS XS 6.87 will predominantly be caused by flooding from coastal sources. As the Variable C location progressed upstream, the two lines crossed at increasingly more frequent annual exceedance probabilities, which indicated flooding was more influenced by the riverine source. It is assumed that peak water surface elevations for any location upstream of HEC-RAS 16.248 will predominantly be caused by flooding from riverine sources.

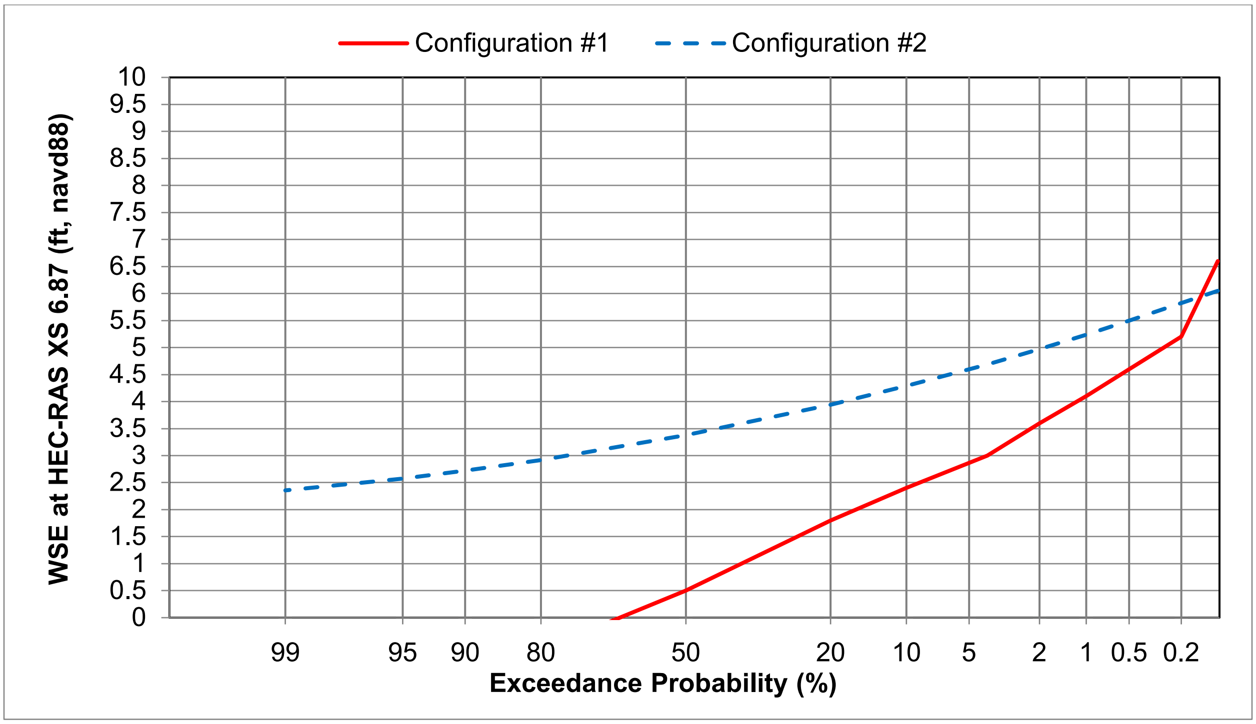

Figure 138. HEC-SSP Coincident Frequency Analysis Results – Configuration#1-2

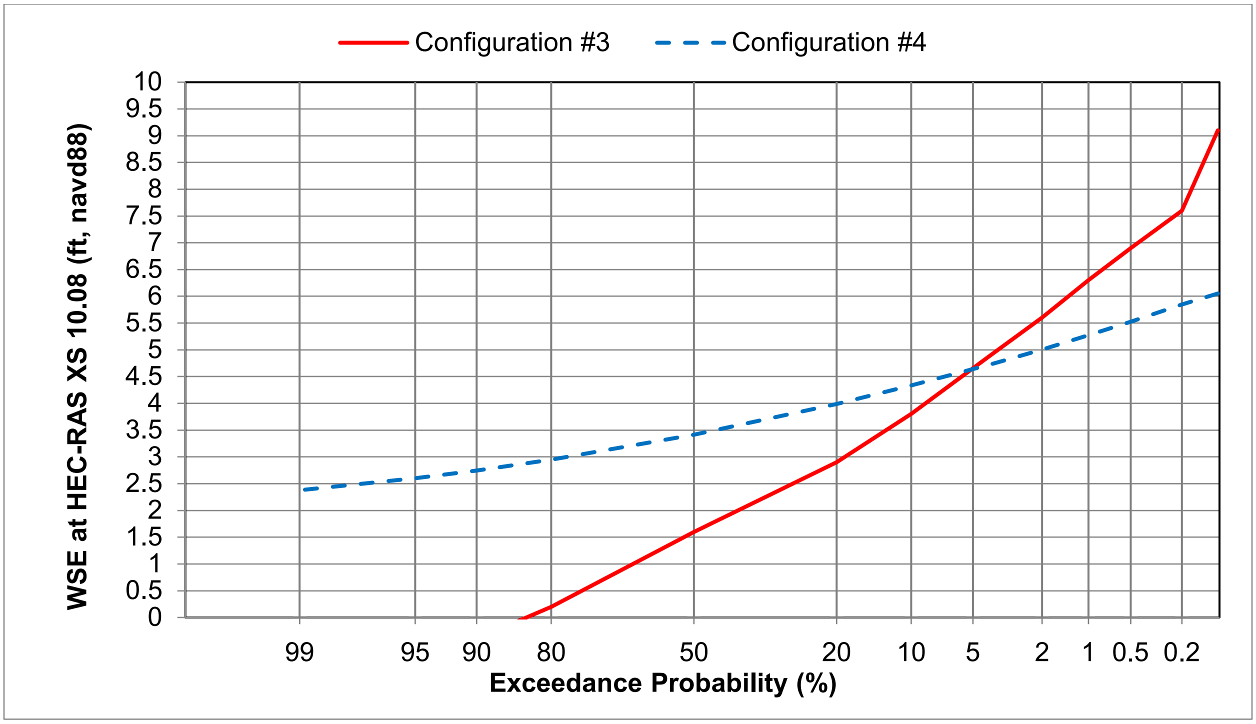

Figure 139. HEC-SSP Coincident Frequency Analysis Results – Configuration#3-4

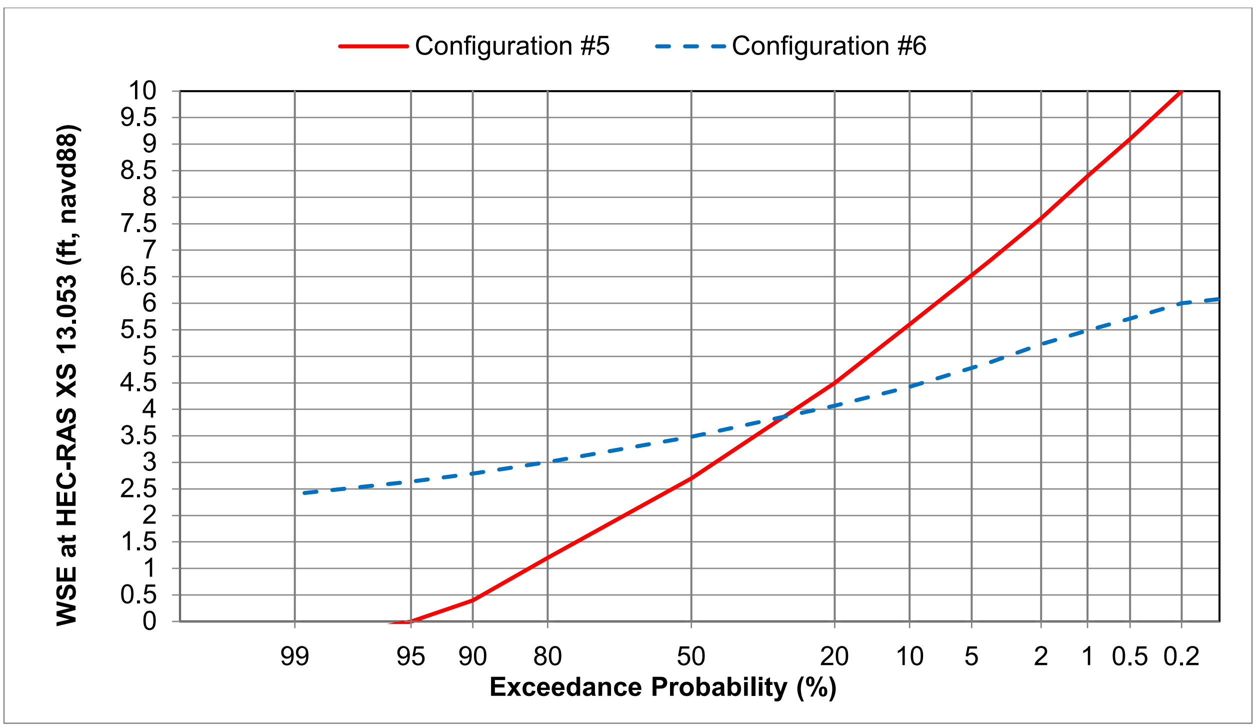

Figure 140. HEC-SSP Coincident Frequency Analysis Results – Configuration#5-6

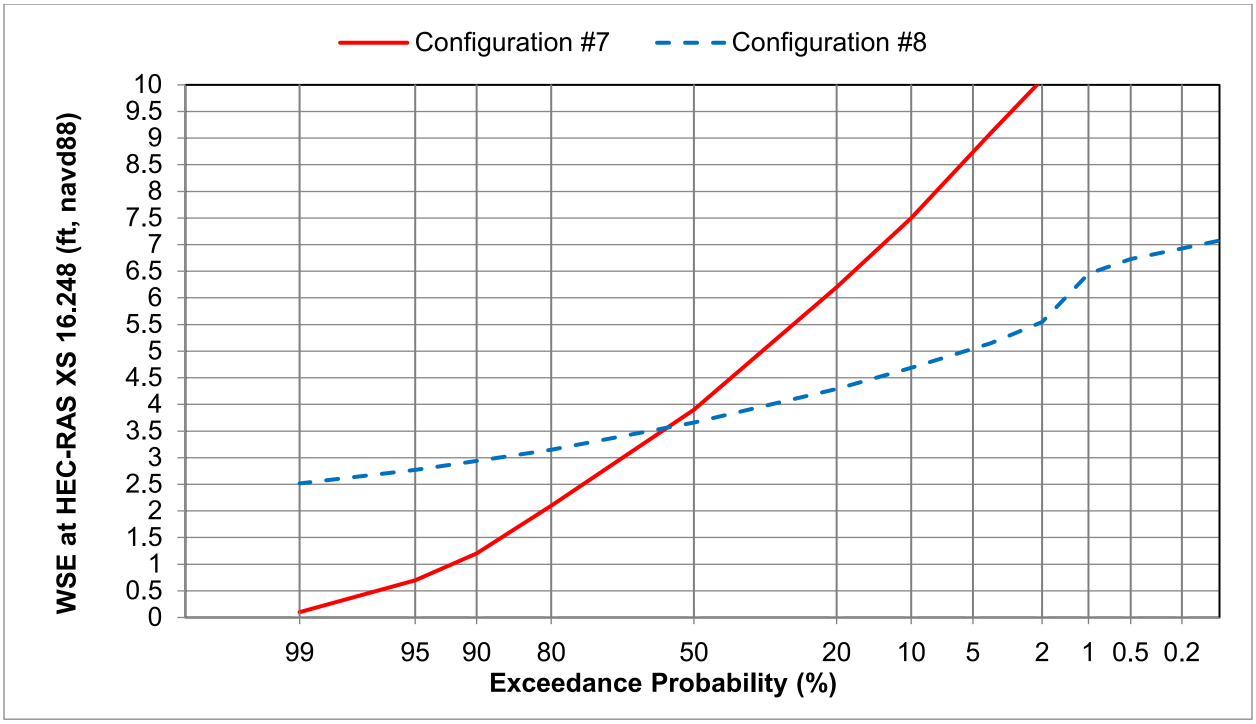

Figure 141. HEC-SSP Coincident Frequency Analysis Results – Configuration#7-8

The perceived correlation (correlation coefficient = 0.392) between the USGS Fort Barnwell station flows and Beaufort, NC NOAA tide buoy daily data was investigated through the use of Monte Carlo simulation in Microsoft Excel. While HEC-SSP did provide an option for Variables A and B to be dependent, this characterization required a family of frequency curves for Fort Barnwell flows conditioned on Beaufort stages. Records available were not sufficient enough to develop such a distribution. Instead, the Monte Carlo simulation approach utilized correlated sampling to investigate the correlation between the two variables. Similar to the HEC-SSP independent coincident frequency analyses, a general frequency analysis characterized the USGS Fort Barnwell station, and a daily-derived duration analysis characterized the Beaufort, NC NOAA station. The general frequency analysis Pearson Type III distribution at Fort Barnwell allowed for sampling flows based on mean, standard deviation, and skew values. Also similar to the HEC-SSP analysis, the spreadsheet simulation relied upon a response curve, which was based HEC-RAS results at XS 6.87. The spreadsheet performed random sampling of 20,000 pairs of annual maximum flow at the USGS Fort Barnwell gage (variable A) along with a Beaufort, NC NOAA station daily water surface elevation (variable B), with a specified correlation coefficient. An initial 100 simulations of sampling 20,000 data pairs were performed assuming a correlation coefficient of 0.0. Another 100 simulations were then performed assuming a correlation coefficient of 0.392, as calculated from the HEC-SSP correlation analysis described in 5.3.1. The two sets of simulation results were averaged for a range of exceedance probabilities, as listed in Table 68. Comparison of these results show an increase in water surface elevations for the more extreme exceedance probability events when assuming some degree of correlation (correlation coefficient = 0.392).

Table 68. Coincident Frequency Analysis – Monte Carlo Simulation – Correlation Comparison

Exceedance Probability (%)	Correlation Coefficient = 0.0 (ft, NAVD88)	Correlation Coefficient = 0.392 (ft, NAVD88)
0.2	5.6	5.8
0.5	5.0	5.2
1	4.5	4.7
2	4.1	4.3
4	3.6	3.8
10	3.1	3.2
20	2.7	2.7
50	2.0	2.0

5.3.3 Summary

A series of correlation and coincident frequency analyses indicated weak to some degree of correlation between data that represented Neuse River flows and Pamlico Sound water surface elevations. Without strong correlation, there is uncertainty in assuming how influential the two flooding sources will act to form compound flooding. Likewise, enough correlation was perceived such that the potential for compound flooding cannot be ignored. Given the complex nature of coastal processes that produce extreme flooding, and the limitations of assessment in this current basin study, it is recommended compound flooding be further investigated in a coastal-based study, specific to the tidally influenced regions of the Neuse River basin and Pamlico Sound estuary.

6 Future Without Project Conditions

6.1 Development

6.1.1 Background

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

Future conditions were modeled by adjusting the percent impervious surface of the subbasins in the models to reflect expected future land use based on projections from city/county watershed master plans and the Environmental Protection Agency (EPA) Integrated Climate and Land Use Scenario (ICLUS) models.

For future conditions in the Crabtree Creek basin HEC-HMS model, locally provided future land use data for the Raleigh and Wake County areas were analyzed for estimating changes in impervious surface area for the applicable subbasins. This analysis showed a notable change in land cover related to increased development in the area. Therefore, future conditions for the Crabtree creek basin were developed by modification of hydrologic lag times and curve numbers to reflect the expected increase in urbanization. For FWOP conditions in the Crabtree Creek basin, curve numbers were projected to increase on average 1.1% over their existing conditions values. Increases ranged from no change up to +24% (subbasin HC1 went from a CN of 56.2 to 81.2). With increased curve numbers and impervious surface in urban areas, subbasin lag times were effectively reduced to 90% of existing conditions value to represent lower penetration and infiltration and increase in flow velocity.

6.1.2 Integrated Climate and Land-Use Scenarios

ICLUS future scenario A1 was selected to represent future change in impervious areas along the Neuse River mainstem study reach. This scenario projection is comprised of moderate-to-rapid economic and population growth, and climate-induced migration. A target year of 2070 was selected to represent the conditions expected for this study's period of analysis. Future loss parameter curve numbers were determined by converting land designated as forest in 2016 NLCD (Deciduous, Evergreen, and Mixed) to Developed, Medium Intensity for each subbasin to reflect an equivalent amount of change in percent impervious area. Zonal statistics were used to calculate an average percent impervious value based on the NLCD 2019 urban imperviousness dataset for each subbasin in the Neuse River mainstem basin HEC-HMS model. ICLUS scenario A1 for year 2020 was compared to the future 2070 scenario to gage the relative change in impervious area. This percent change was then applied to the NLCD 2019 value. This method was used due to the coarse resolution of the ICLUS model. Results of this exercise at the subbasin level revealed insignificant changes to existing conditions curve numbers. There was an absolute value increase by 0.19, or about a 1.003% difference. Based on results of this analysis, future without project conditions were not projected to differ from existing conditions for the study areas outside of the Crabtree Creek basin.

For convenience, a basin-wide overview and breakdown of the forecasted changes in NLCD land use classifications for the 4 ICLUS scenarios are included below: Land use and land cover (LULC) for the conterminous United States was modeled from 1992-2005 using historical LULC data and from 2006-2100 based on 4 scenarios from the Intergovernmental Panel on Climate Change (IPCC) Special Report on Emissions Scenarios. These models forecast 17 land use classes on a 250 m grid and produce an annual map of LULC. The A scenarios are more economically driven while the B scenarios are more environmentally driven. The A1B and B1 scenarios have the same global population assumptions (growth in population until 2050 followed by decline), the A2 scenario has the highest population assumption with steady, and the B2 scenario has the second highest global population assumption with steady growth (but at a slower rate than A2).

The annual maps were analyzed for pixel coverage to give a percentage of each land cover type. Annual percent coverage in 2006, 2021, and 2100 are listed in Table 69 through Table 72 below. The tables also include the percent change from 2021- 2100 with a positive percent change showing an increase in that land coverage and a negative percent change indicating a decrease in that land cover type. Figure 142 through Figure 145 Show the annual maps for 2006, 2021 and 2100 for each of the 4 scenarios.

Table 69. LULC Change A1B Scenario

Land Cover Type	Coverage 2006	Coverage 2021	Coverage 2100	Percent Change 2021-2100
Water	1.18%	1.20%	1.14%	-0.06%
Developed	7.95%	10.30%	26.13%	15.84%
Mechanically Distributed Public Lands	0.02%	0.02%	0.02%	0.00%
Mechanically Distributed Private Lands	2.48%	1.69%	2.00%	0.31%
Mining	0.17%	0.19%	0.18%	-0.01%
Barren	0.13%	0.13%	0.13%	0.00%
Deciduous Forests	18.33%	16.93%	7.72%	-9.21%
Evergreen Forests	15.21%	14.11%	7.04%	-7.08%
Mixed Forests	7.32%	6.72%	2.79%	-3.93%
Cropland	26.01%	26.51%	32.73%	6.22%
Pasture Land	6.36%	6.32%	4.77%	-1.54%
Herbaceous Wetlands	0.31%	0.32%	0.32%	0.00%
Woody Wetlands	14.53%	15.56%	15.02%	-0.54%

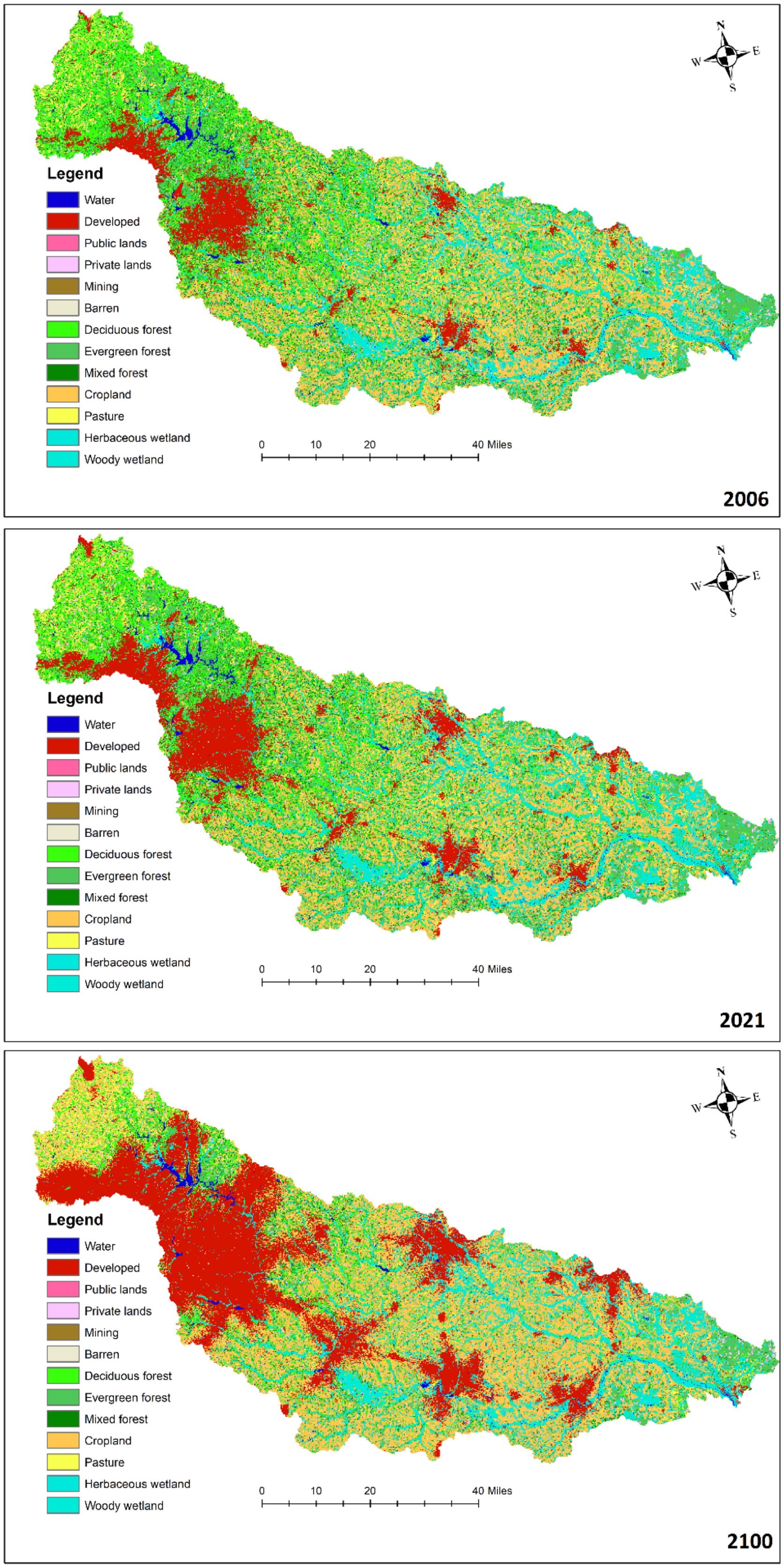

Figure 142. Land Cover Projections for Scenario A1B

Table 70. LULC Change A2 Scenario

Land Cover Type	Coverage 2006	Coverage 2021	Coverage 2100	Percent Change 2021-2100
Water	1.18%	1.15%	1.09%	-0.06%
Developed	7.95%	11.20%	31.22%	20.02%
Mechanically Distributed Public Lands	0.02%	0.02%	0.00%	-0.02%
Mechanically Distributed Private Lands	2.48%	1.39%	0.86%	-0.53%
Mining	0.17%	0.19%	0.20%	0.01%
Barren	0.13%	0.13%	0.13%	0.00%
Deciduous Forests	18.33%	16.22%	4.47%	-11.74%
Evergreen Forests	15.21%	13.68%	4.74%	-8.94%
Mixed Forests	7.32%	6.47%	1.60%	-4.87%
Cropland	26.01%	27.72%	35.26%	7.55%
Pasture Land	6.36%	6.39%	6.06%	-0.33%
Herbaceous Wetlands	0.31%	0.31%	0.30%	-0.01%
Woody Wetlands	14.53%	15.14%	14.06%	-1.08%

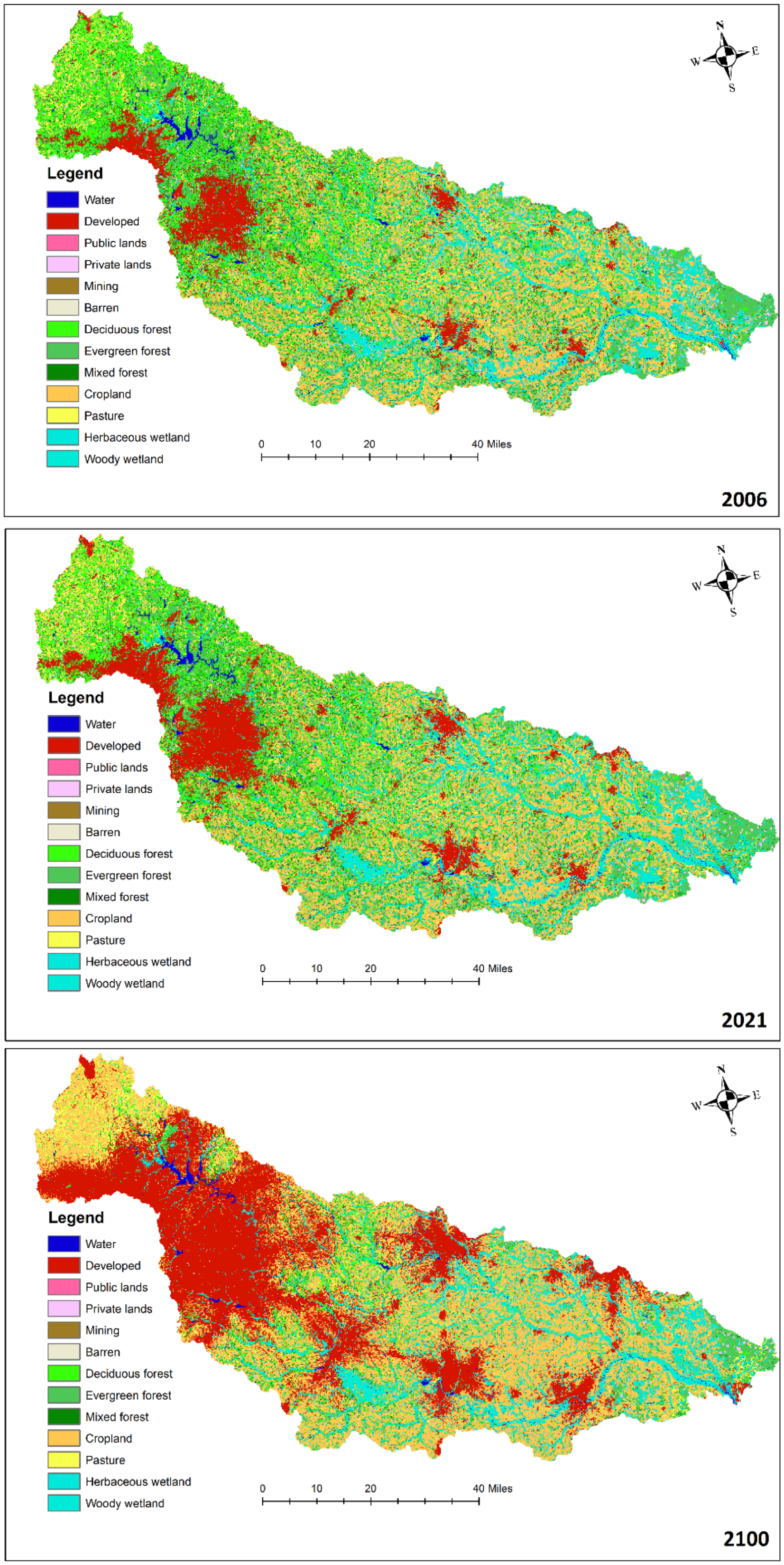

Figure 143. Land Cover Projections for Scenario A2

Table 71. LULC Change B1 Scenario

Land Cover Type	Coverage 2006	Coverage 2021	Coverage 2100	Percent Change 2021-2100
Water	1.18%	1.20%	1.25%	0.05%
Developed	7.95%	10.27%	20.24%	9.97%
Mechanically Distributed Public Lands	0.02%	0.01%	0.01%	-0.01%
Mechanically Distributed Private Lands	2.48%	0.50%	0.41%	-0.08%
Mining	0.17%	0.18%	0.15%	-0.03%
Barren	0.13%	0.13%	0.13%	0.00%
Deciduous Forests	18.33%	18.84%	16.22%	-2.61%
Evergreen Forests	15.21%	15.49%	13.45%	-2.04%
Mixed Forests	7.32%	7.28%	5.79%	-1.49%
Cropland	26.01%	23.85%	20.49%	-3.36%
Pasture Land	6.36%	6.36%	5.49%	-0.87%
Herbaceous Wetlands	0.31%	0.33%	0.36%	0.03%
Woody Wetlands	14.53%	15.56%	16.00%	0.44%

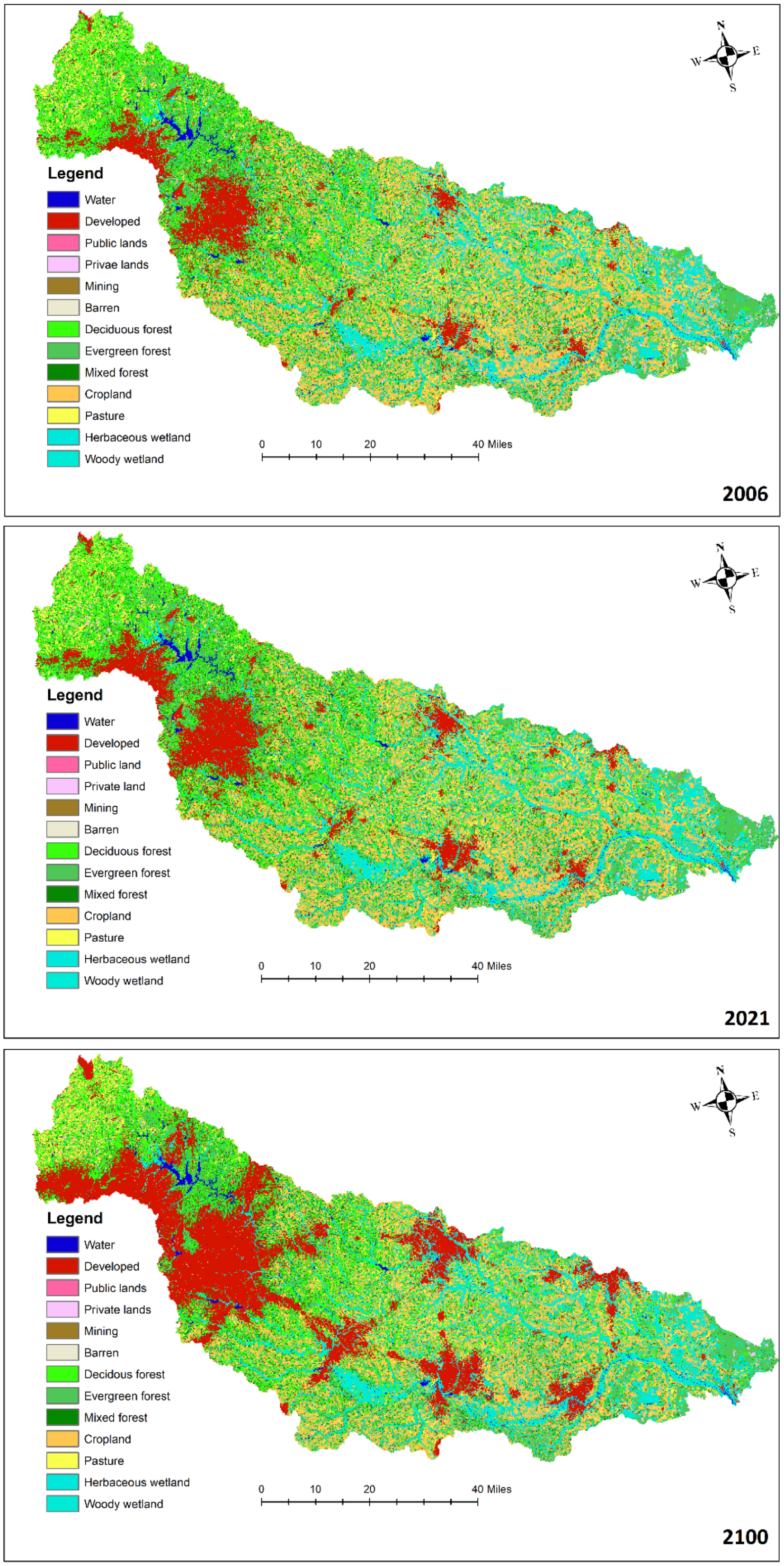

Figure 144. Land Cover Projections for Scenario B1

Table 72. LULC Change B2 Scenario

Land Cover Type	Coverage 2006	Coverage 2021	Coverage 2100	Percent Change 2021-2100
Water	1.18%	1.16%	1.43%	0.27%
Developed	7.95%	10.46%	13.89%	3.44%
Mechanically Distributed Public Lands	0.02%	0.02%	0.02%	0.00%
Mechanically Distributed Private Lands	2.48%	0.46%	0.99%	0.53%
Mining	0.17%	0.19%	0.23%	0.04%
Barren	0.13%	0.13%	0.13%	0.00%
Deciduous Forests	18.33%	17.66%	21.13%	3.47%
Evergreen Forests	15.21%	14.79%	17.65%	2.86%
Mixed Forests	7.32%	6.97%	7.55%	0.58%
Cropland	26.01%	26.35%	12.69%	-13.67%
Pasture Land	6.36%	6.20%	6.69%	0.49%
Herbaceous Wetlands	0.31%	0.32%	0.43%	0.12%
Woody Wetlands	14.53%	15.29%	17.17%	1.88%

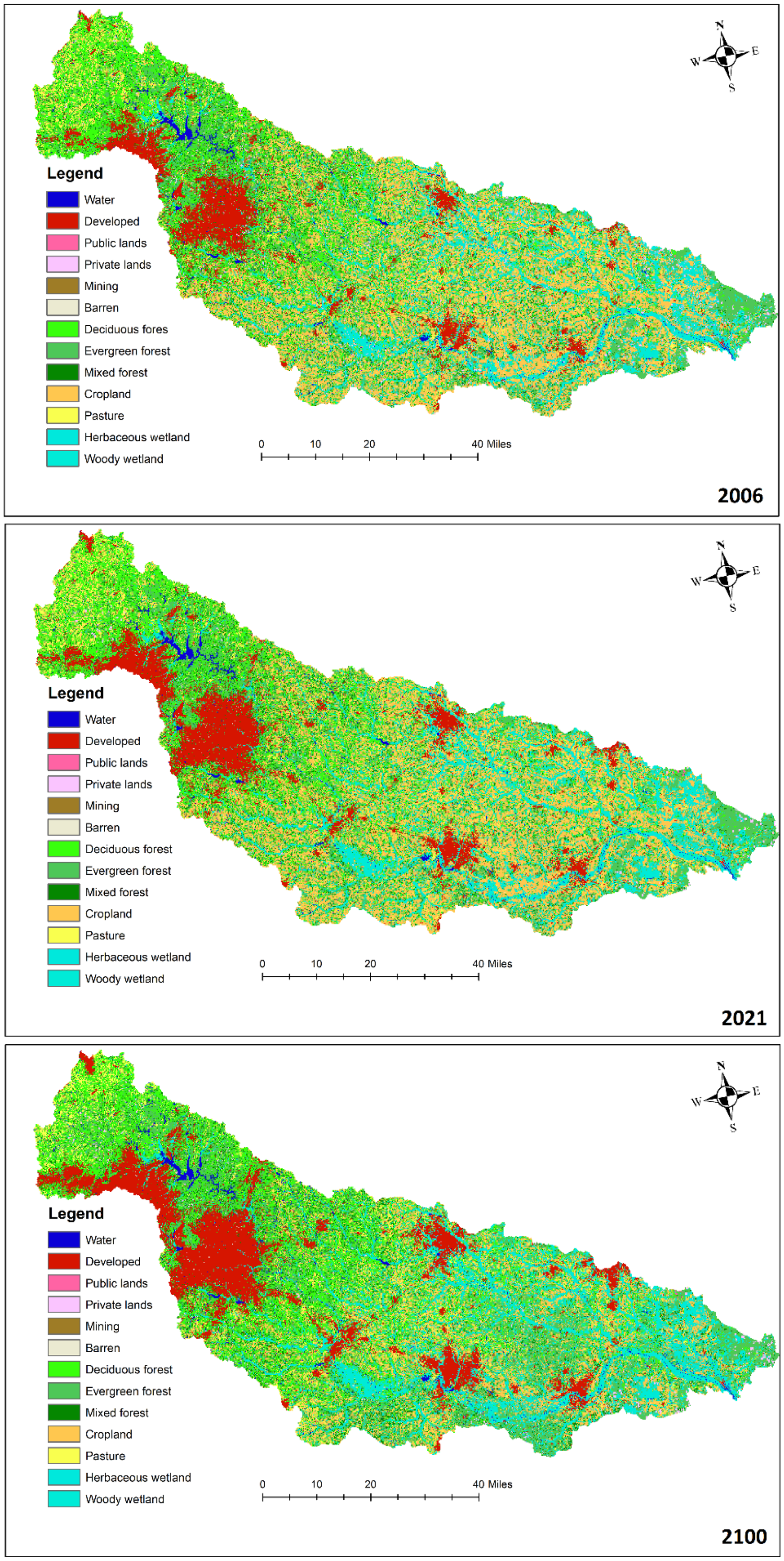

Figure 145. Land Cover Projections for Scenario B2

All four modeled scenarios predict an increase in developed land cover from 2021 to 2100 ranging from 3% to 20%. All four models also show minimal (<0.05%) change in public lands, barren land cover, and mining land cover. Forecasted changes in deciduous, evergreen and mixed forest land cover range from increasing 3.5%, 3%, and 0.6% respectively to decreasing by 12%, 9%, and 5 % respectively. Private land cover change ranged from decreasing by 0.5% to increasing by 0.5%. Cropland land cover change ranged from decreasing by 13.7% to increasing by 7.5%. Pasture land cover change ranged from decreasing by 1.5% to increasing by 0.5%. Herbaceous and woody wetland land cover change ranged from decreasing by 0% and 1% respectively to increasing by 0.12% and 2% respectively. While forecasted LULC changes vary widely, the four scenarios all predict an increase in developed land cover as population increases.

6.2 Future Projected Sea Level Change

6.2.1 Applicability to Study Model Domains

Per Engineering and Construction Bulletin 2018-14, determination was made as to whether sea level rise would affect river stage by increasing (or decreasing) water surface elevation downstream of the five study model domains. Based on developed floodplain topography within the HEC-RAS hydraulic model, minimum elevation (NAVD88 datum) for project areas of the Crabtree Creek, Hominy Swamp Creek, and Big Ditch model domains were approximately 165 feet, 75 feet, and 65 feet, respectively.

For the Adkins Branch model domain, an approximate 3-mile length (out of a total model length of 5.3 miles) of developed floodplain adjacent to the stream had a minimum ground elevation that ranged from 25 feet to 50 feet, NAVD88. The Adkins Branch and Neuse River confluence is located at least 50 river miles upstream of the Neuse River and Pamlico Sound confluence. Remarks of the USGS Neuse River at Kinston, NC station (02089500), located 4 river miles upstream of Adkins Branch do not describe flows as being affected by astronomical or wind tides. The NOAA Sea Level Rise Viewer (https://coast.noaa.gov/slr/) was used to visualize extreme conditions of water level increase associated with projected future sea level rise as determined by the Beaufort, NC NOAA tide buoy (8656483). This viewer displays resulting water levels from incremental effects of sea level rise on top of mean higher high water (MHHW) through a modified “bathtub” approach that attempts to account for local and regional tide variability and hydrological connectivity (NOAA, 2017). As shown in Figure 146, even after assuming a 10 feet water level on top of MHHW, the inundation boundary does not extend upstream to Adkins Branch.

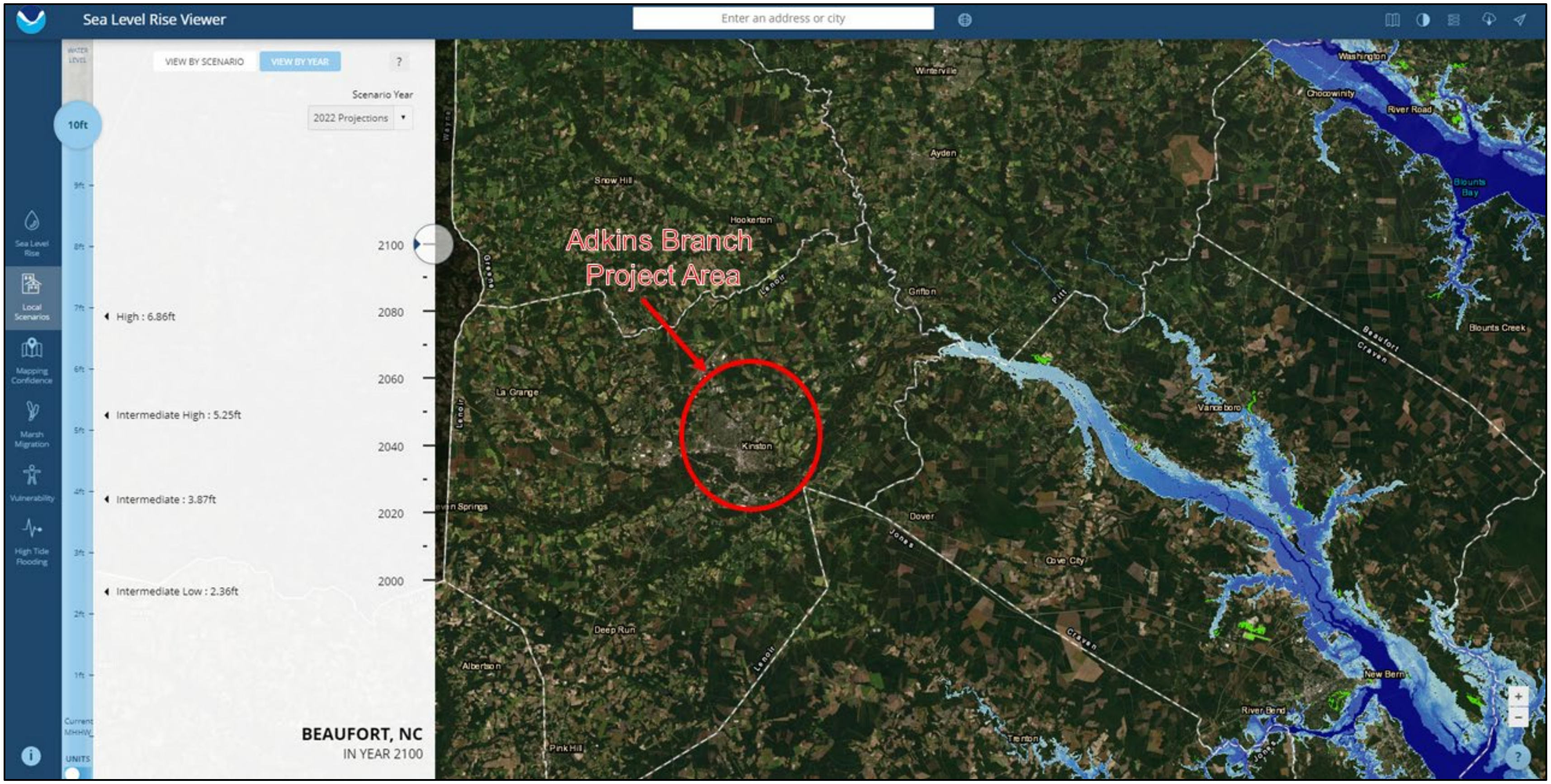

Figure 146. NOAA Sea Level Rise Viewer – Adkins Branch Extreme SLR Conditions

Based on information in the preceding two paragraphs, the study model domains of Crabtree Creek, Hominy Swamp Creek, Big Ditch, and Adkins Branch did not warrant policy and procedures outlined in Engineering Regulation 1100-2-8162 and Engineering Technical Letter 1100-2-1.

It was assumed that the Neuse River mainstem model domain required a more detailed assessment of sea level change due to its confluence with the Pamlico Sound. Review of aerial imagery of the Neuse River floodplain that stretched downstream of the City of Kinston, NC to the City of New Bern, NC revealed a >= 1.0-mile width of undeveloped swamp and marsh landcover surrounding the main channel. Beyond this low-lying region, minimum developed floodplain ground elevations at or below 50.0 feet, NAVD88 extended approximately 70 river miles upstream of the Neuse River and Pamlico Sound confluence. This total distance appeared to extend well beyond the area that is normally tidally influenced and therefore it was necessary to determine the segment of the Neuse River that would be affected by sea level change. Remarks of the USGS Neuse River near Fort Barnwell, NC station (02091814), located 25 river miles upstream of the City of New Bern, NC describes flows as being affected by astronomical and wind tides. Therefore, a minimum distance upstream of the Neuse River and Pamlico Sound confluence in which river stages would be appreciably affected by sea level change was set to the USGS Fort Barnwell, NC station. The maximum 10-foot water level increase on top of MHHW from the NOAA Sea Level Rise Viewer and locations of nearby USGS stations are shown in Figure 147. 1-foot increments of inundation rasters were extracted from the viewer and are shown in Figure 148, Figure 149, and Figure 150. As shown in Figure 147, the maximum inundation boundary extended to the Neuse River and Contentnea Creek confluence, also near the Town of Grifton, NC. Remarks of the USGS Contentnea Creek at Hookerton, NC station (02091500), also included in Figure 147, located about 20 river miles upstream of its confluence with the Neuse River, do not describe flows as being affected by astronomical or wind tides.

Figure 147. NOAA Sea Level Rise Viewer – Neuse River Extreme SLR Conditions

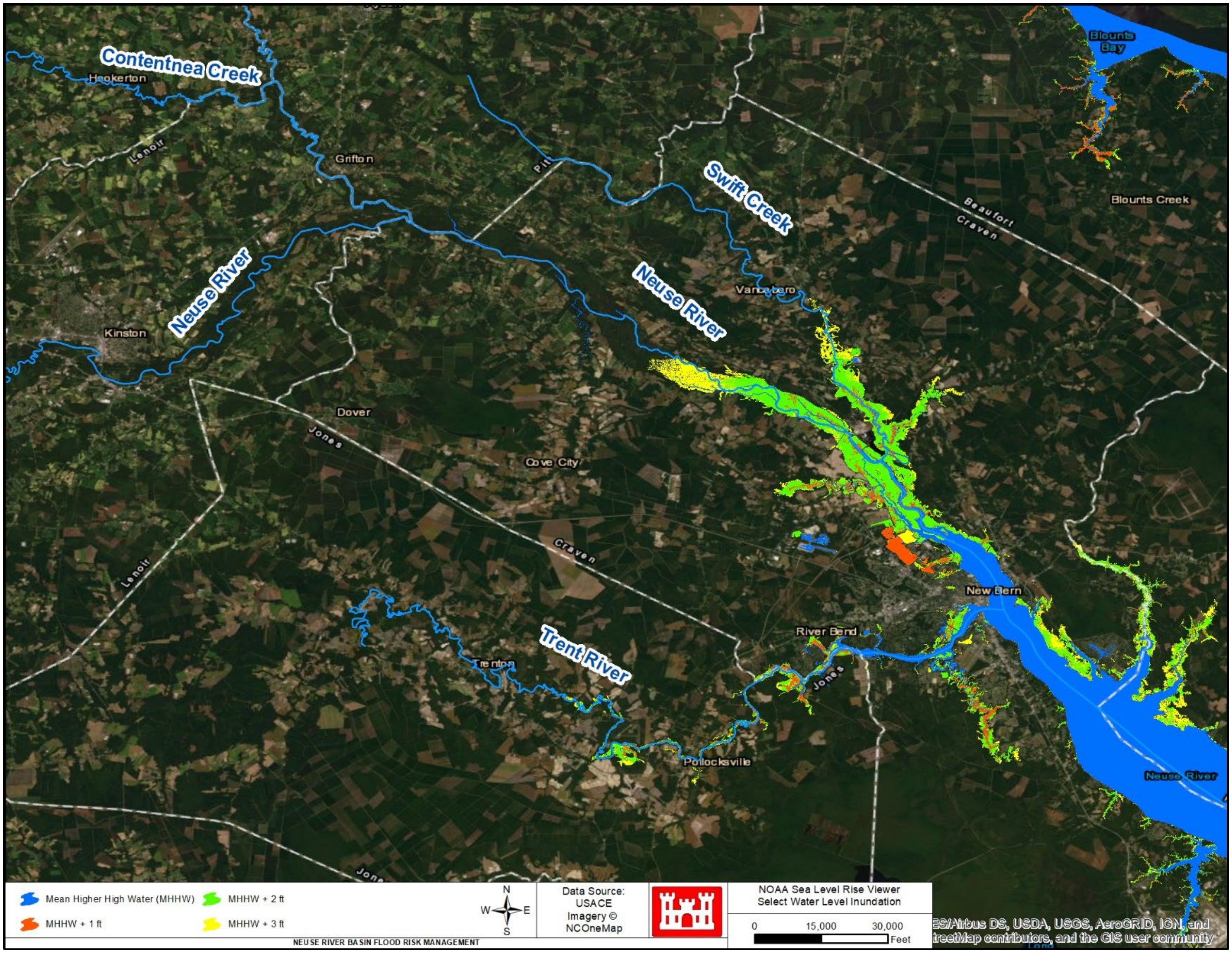

Figure 148. NOAA Sea Level Rise Viewer – MHHW & 1-, 2-, 3-ft Water Level Increases

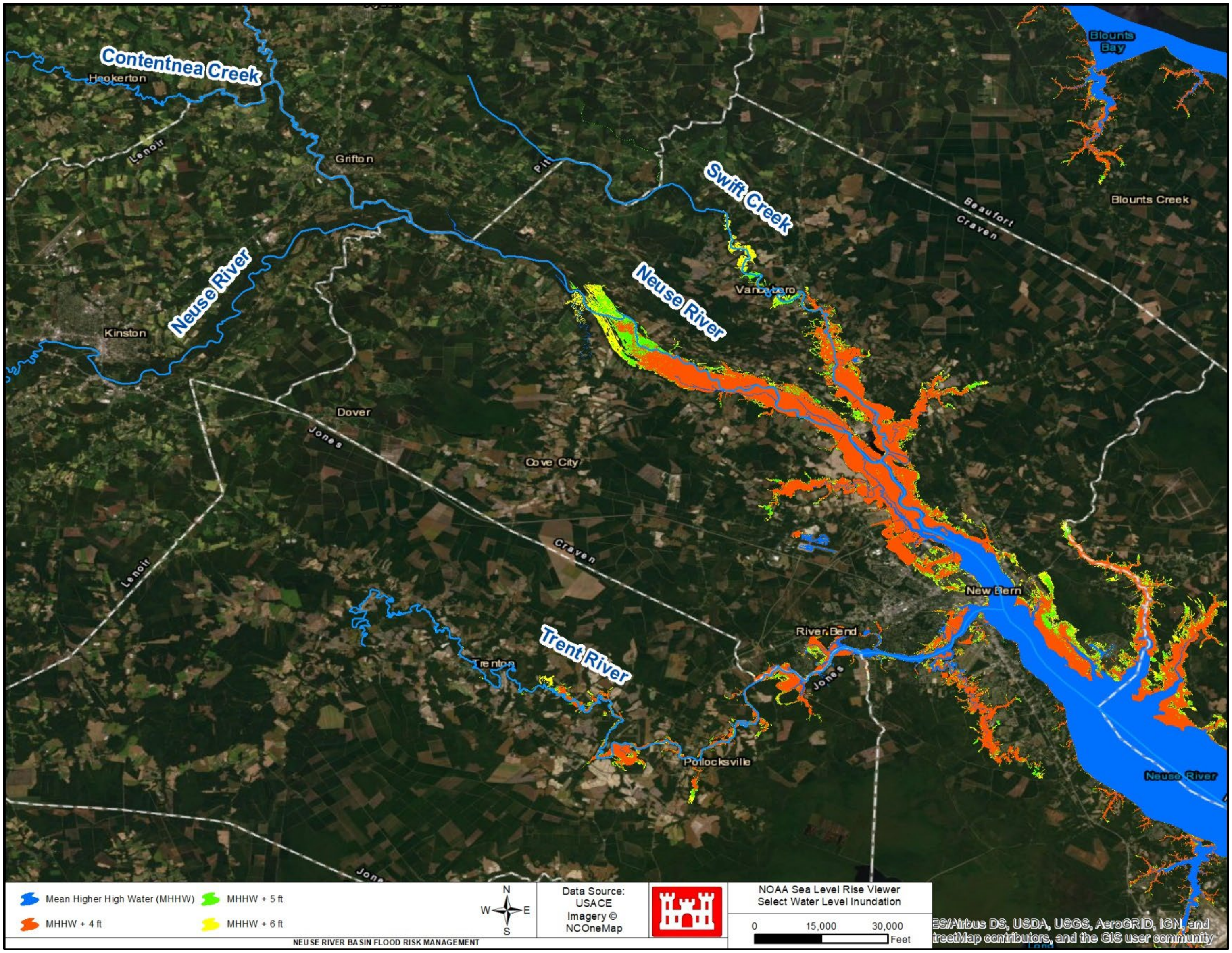

Figure 149. NOAA Sea Level Rise Viewer – MHHW & 4-, 5-, 6-ft Water Level Increases

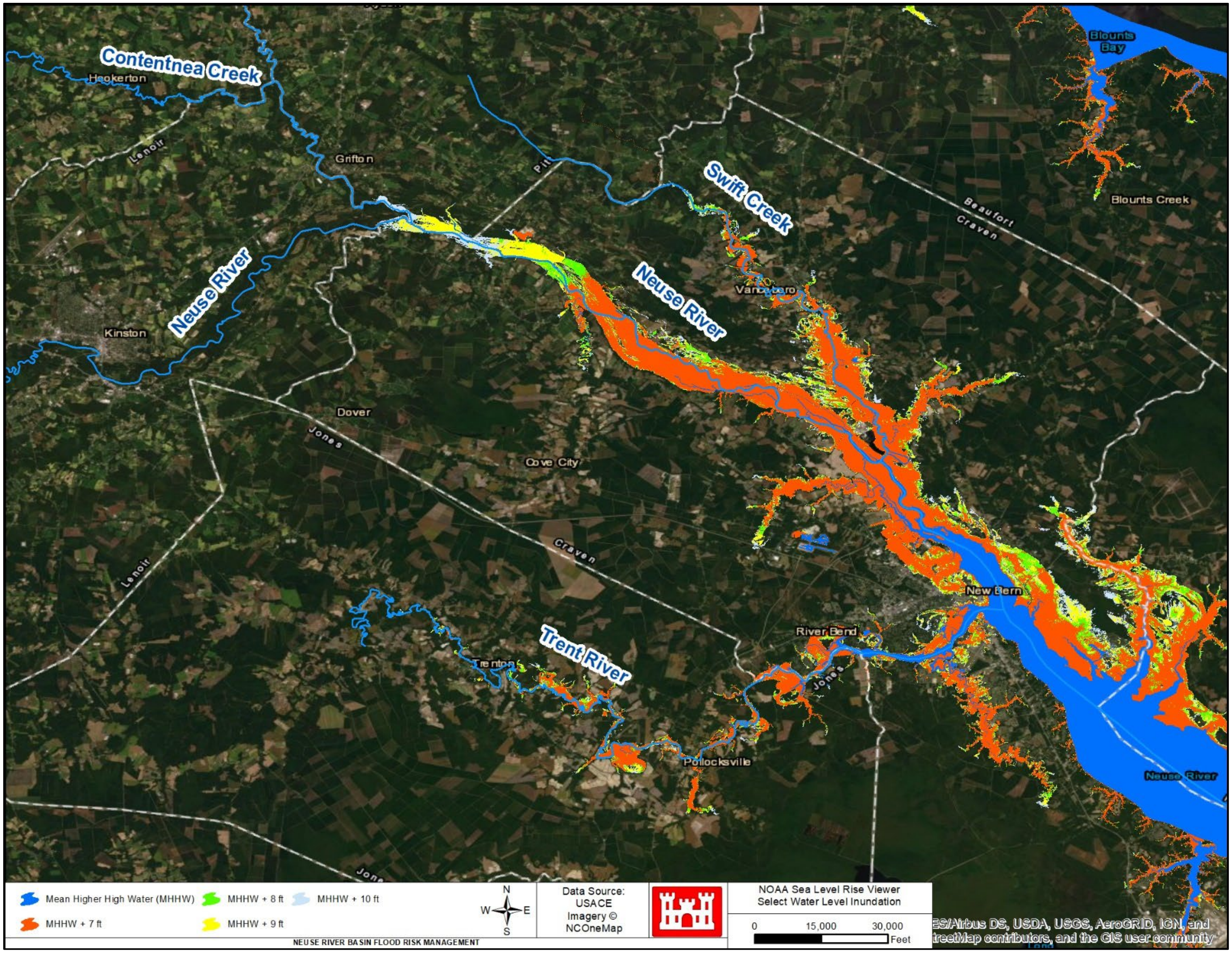

Figure 150. NOAA Sea Level Rise Viewer – MHHW & 7-, 8-, 9-, 10-ft Water Level Increases

Review of USGS station remarks related to effects of astronomical and wind tides as well as use of the NOAA Sea Level Rise Viewer suggested that the projected future tidally influenced region of the Neuse River basin will likely extend well into the western-most portions of Craven County, NC. As such, policies and procedures within ER 1100-2-8162 and Engineering Technical Letter (ETL) 1100-2-1 were applied to the Neuse River mainstem model domain. Specifically, backwater profiles within the HEC-RAS hydraulic model included potential relative sea level change in starting water surface elevations.

6.2.2 Neuse River Mainstem

Section 11.6.3 describes an assessment of sea level change at the Beaufort, NC NOAA tide buoy station (8656483). This was the nearest NOAA station to the study area that possessed a robust data record for which water level statistics could be derived. The USACE Sea Level Tracker tool (https://climate.sec.usace.army.mil/slr_app/) was used to determine an appropriate projected increase in water level that has potential to occur in the future. For a 50-year project life ending at the end of calendar year 2090, low, intermediate, and high sea level change values were 0.13 meters, 0.40 meters, and 1.23 meters, respectively. An initial concern about applying the relative sea level change (RSLC) analysis conducted at the Beaufort, NC station directly to the Neuse River mainstem hydraulic model was mitigated through the use of ERDC CHS (https://chs.erdc.dren.mil). The nodal dataset available within CHS allowed for a comparison between the two locations and development of a nonlinear residual adjustment factor that was applied to the USACE Sea Level Tracker RSLC low, intermediate, and high values. The adjustment factor accounted for not only the relative difference in geographic location but also for the different SLC scenario methodology used in the CHS analysis. CHS modeled three SLC scenarios for the studied region: SLC#0 = 0 m SLC, SLC#1 = 0.832m (2.73 ft), and SLC#2 = 2.24 m (7.35 ft). These SLC values were chosen for the South Atlantic region because 0.832 m is approximately equal to the typical increase in sea level using the USACE intermediate curve for the year 2120 and 2.24 m is approximately equal to the USACE high curve for the same year. Final adjustment factor values per annual exceedance probability is listed in Table 73. As the positive values in Table 73 suggest, CHS analysis showed there to actually be a minor increase in SLC when transferred from the Beaufort, NC NOAA tide buoy station to near the mouth of the Neuse River and downstream boundary of its HEC-RAS hydraulic model. This increase may be explained by the change in bathymetry within the Pamlico Sound as it constricts towards the mouth of the Neuse River.

Table 73. CHS Adjustment Factor for Sea Level Change

AEP	Adjustment Factor per Meter of SLC
0.5	0.040
0.2	0.059
0.1	0.066
0.04	0.084
0.02	0.096
0.01	0.109
0.005	0.113
0.002	0.127

The methodology described in Section 5.2.3 for establishing the downstream boundary condition for the Neuse River mainstem HEC-RAS model was revised utilizing the high SLC curve value of 1.23 meters, adjustment factors in Table 73, and using CHS nodal data that had modeled significant wave heights based on their regional CHS SLC#1 scenario (SLC represented by sea level change of 0.8321 meters). Use of the CHS SLC#1 scenario significant waves heights was an attempt to acknowledge the overall impact of increased water levels related to future conditions, a comparison to base condition (CHS SLC#0 scenario) values is listed in Table 74.

Table 74. Significant Wave Heights – CHS SLC Scenario #0 versus Scenario #1

AEP	Base Condition (No SLC) (ft)	CHS SLC#1 Scenario (ft)
0.5	2.14	2.30
0.2	2.99	3.11
0.1	3.71	3.83
0.05	4.34	4.48
0.02	5.03	5.15
0.01	5.46	5.56
0.005	5.85	5.90
0.002	6.32	6.32

Final steps in determining FWOP water surface elevations for the Neuse River mainstem HEC-RAS model downstream boundary involved combining the existing conditions Beaufort, NC NOAA tide buoy MHHW (0.445 m) and CHS SCL#1 scenario Significant Wave Height (in meters), then adding the product of 1+adjustement factor and high SLC value of 1.23 m. Final peak water surface elevation in feet, NAVD88 per annual exceedance probability is listed in Table 75. While the methodology described above is more complex than simply applying Beaufort, NC NOAA tide buoy derived RSLC to the study area, its resulting increase in water levels ranged from 4.4 to 4.5 feet, which was close to the USACE High value in year 2090 of 4.39 feet.

Table 75. Neuse River Mainstem HEC-RAS Peak Water Surface Elevations for Downstream Boundary Condition

Return Frequency (AEP)	Starting Water Surface Elevation (ft, NAVD88)
0.5	7.95
0.2	8.84
0.1	9.59
0.4	10.54
0.2	11.03
0.01	11.49
0.005	11.86
0.002	12.32

The stage hydrograph time series was developed using the same methods under existing conditions, described in Section 5.2.3. The FWOP water surface elevation boundary condition time series was based on a ratio between the peak stage observed during the historic Hurricane Matthew event and the different return frequency values listed in the preceding table. Hydrograph ordinates of each 15-minute timestep were multiplied by this ratio to produce the final stage hydrograph.

6.3 Frequency Simulation Results

6.3.1 Hydrology

The implementation of FWOP hydrologic conditions produced flow rates larger than existing conditions for the suite of design storm within the Crabtree Creek basin. Differences between future without project conditions and existing conditions at select HEC-HMS model junctions along the Crabtree Creek mainstem is listed in Table 76.

Table 76. Crabtree Creek FWOP and EC Comparison of Design Storm Flows at Select Model Junctions

		Design Storm Frequency Discharge (cfs)							
Location	Drainage Area (sq mi)	0.5	0.2	0.1	0.04	0.02	0.01	0.005	0.002
ctc27c	54.1	205	186	528	940	1026	1067	1096	1048
ctc28c	55.0	205	123	460	885	963	999	1022	1037
ctc29c	60.6	538	1000	1438	1697	1865	2007	2130	2266
ctc30c	76.9	1010	1684	2090	2405	2611	2822	2949	3298
ctc31c	84.8	1075	1703	2102	2446	2679	2945	3102	3327
ctc32c	86.3	1093	1711	2105	2549	2709	3003	3164	3378
ctc33c	95.0	1135	1729	2129	2924	2979	3306	3631	3646
ctc34c	98.7	1164	1718	2142	3030	3498	4221	4629	4305
ctc35c	110.1	1319	1612	1940	3007	3450	3922	4399	4310
ctc35ac	110.1	1320	1610	1943	3007	3449	3921	4398	4312
ctc35bc	110.3	1322	1618	1945	3002	3467	3920	4390	4314
ctc36c	115.8	1512	1873	2172	2315	3082	4030	4045	4378
ctc125c	121.7	1515	1892	2124	2374	2987	4255	4067	3616
ctc126c	122.1	1471	1866	2052	2366	3143	3835	4416	4715
ctc39c	127.8	1462	1875	2017	2445	3154	3834	4124	4388
ctc40c	140.4	1478	1906	1889	2308	3096	3981	4332	4628
ctc41c	144.1	1501	1939	1936	2317	3091	3958	4351	4638
ctc42c	145.2	1507	1947	1946	2320	3096	3960	4355	4647

As detailed earlier, there were insignificant differences between existing conditions and future without project conditions for projected increased impervious area within the Neuse River mainstem, and other tributary models. As such, existing conditions frequency simulation results described in the previous section are assumed to be representative of FWOP conditions.

6.3.2 Hydraulics

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 Crabtree Creek basin produced profiles representative of the flooding potential for floodplain conditions that include anticipated future development. For the Hominy Swamp Creek, Big Ditch, and Adkins Branch study model domains, FWOP hydraulic simulations were considered equivalent to existing conditions.

Select FWOP design event inundations and corresponding water surface profiles for Crabtree Creek and the tidally influenced portion of Neuse River specific study reaches are shown in the following figures within this section.

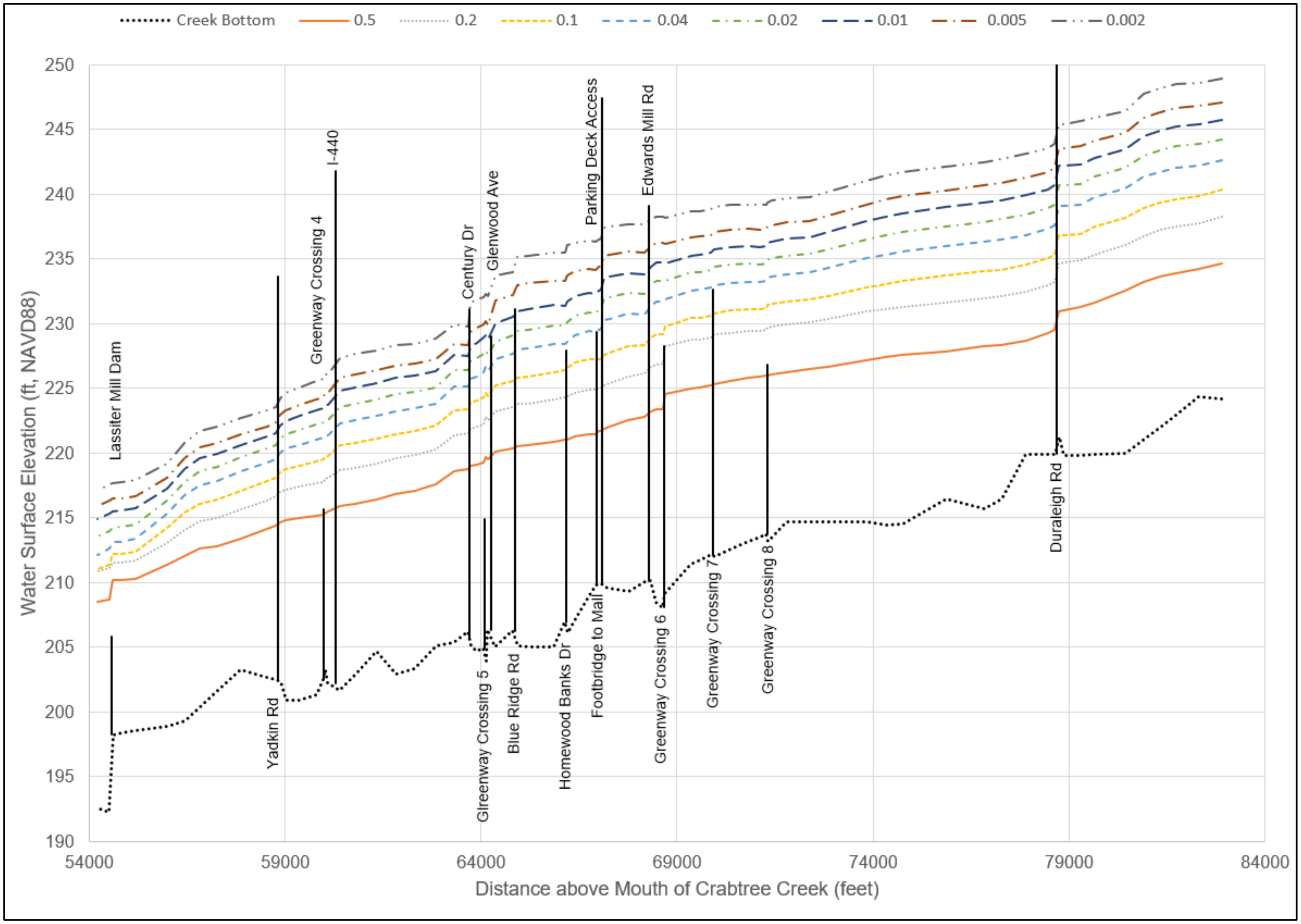

Figure 151. Crabtree Creek FWOP Modeled Water Surface Profiles for Select Design Events from Ebenezer Church Rd to Lassiter Mill Rd

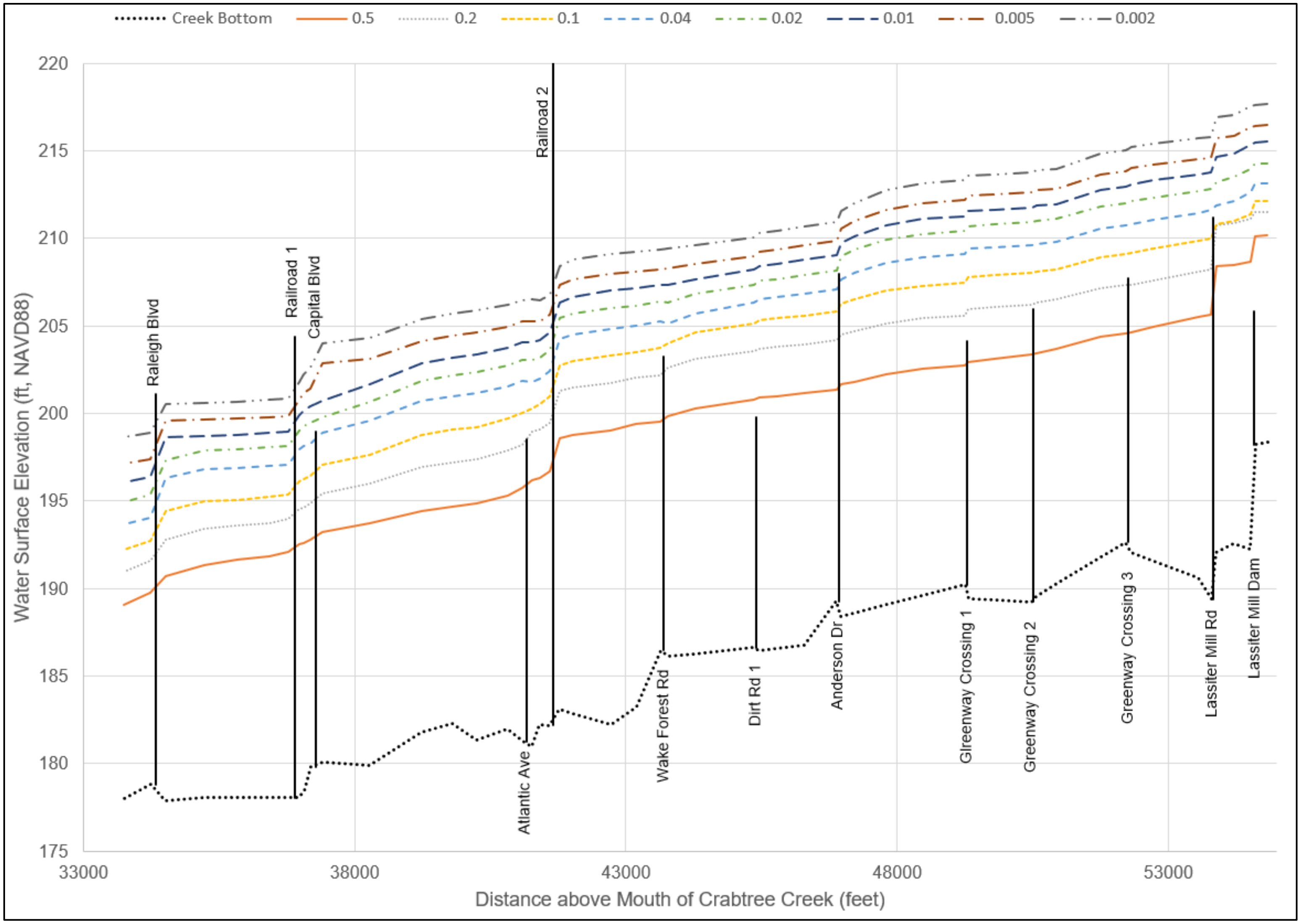

Figure 152. Crabtree Creek FWOP Modeled Water Surface Profiles for Select Design Events from Lassiter Mill Rd to Raleigh Blvd

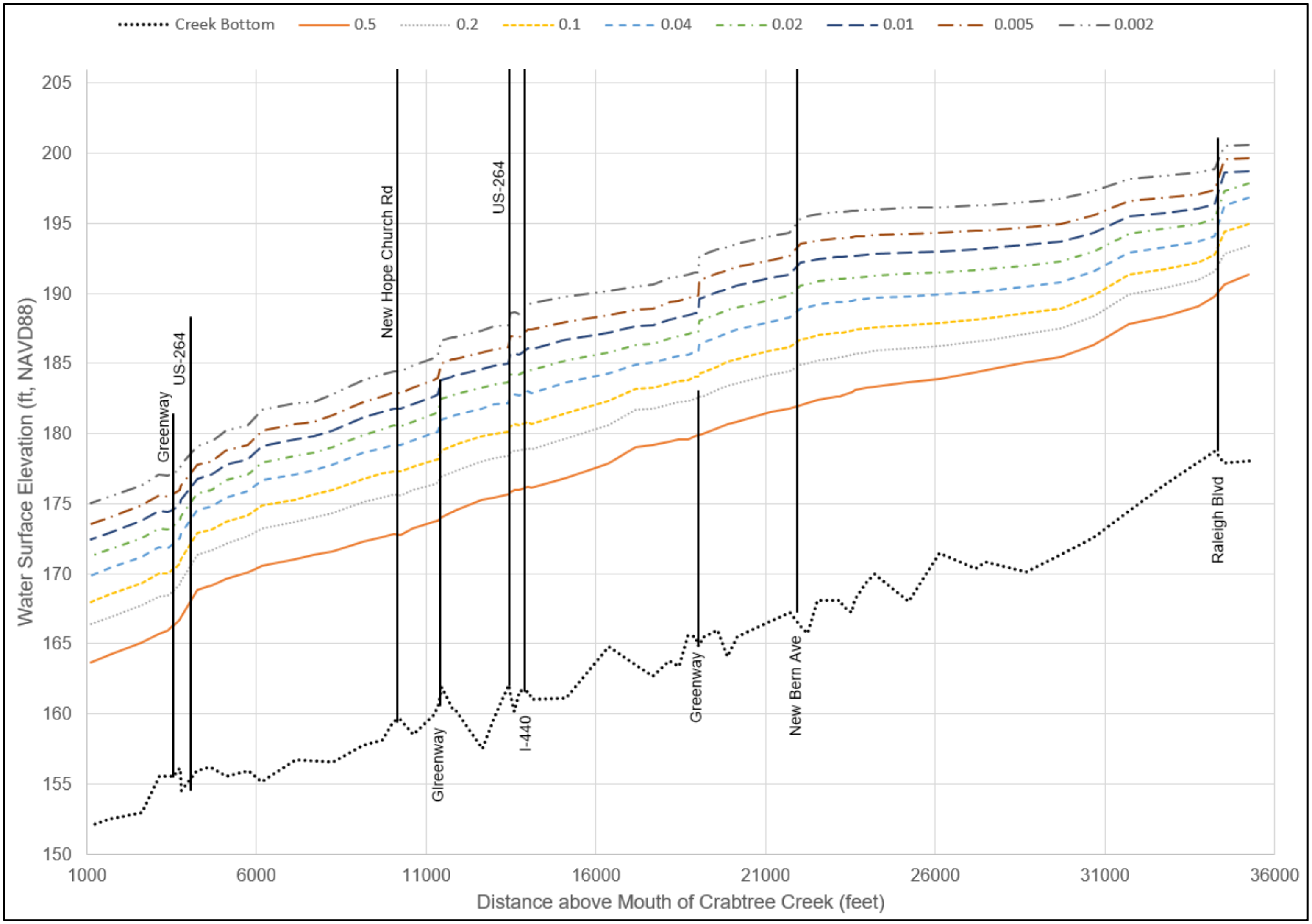

Figure 153. Crabtree Creek FWOP Modeled Water Surface Profiles for Select Design Events from Raleigh Blvd to Mouth

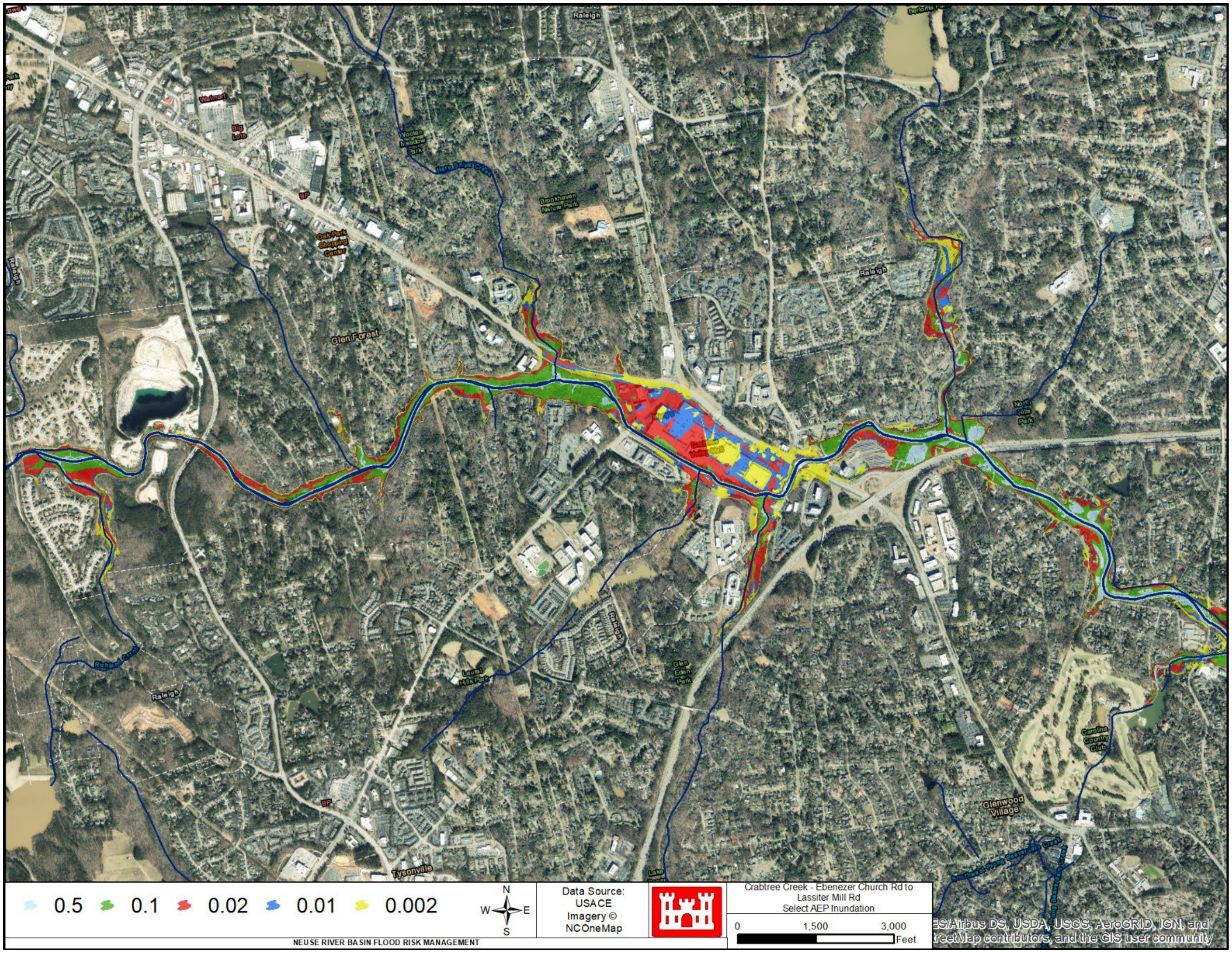

Figure 154. Crabtree Creek FWOP Modeled Inundation for Select Design Events from Ebenezer Church Rd to Lassiter Mill Rd

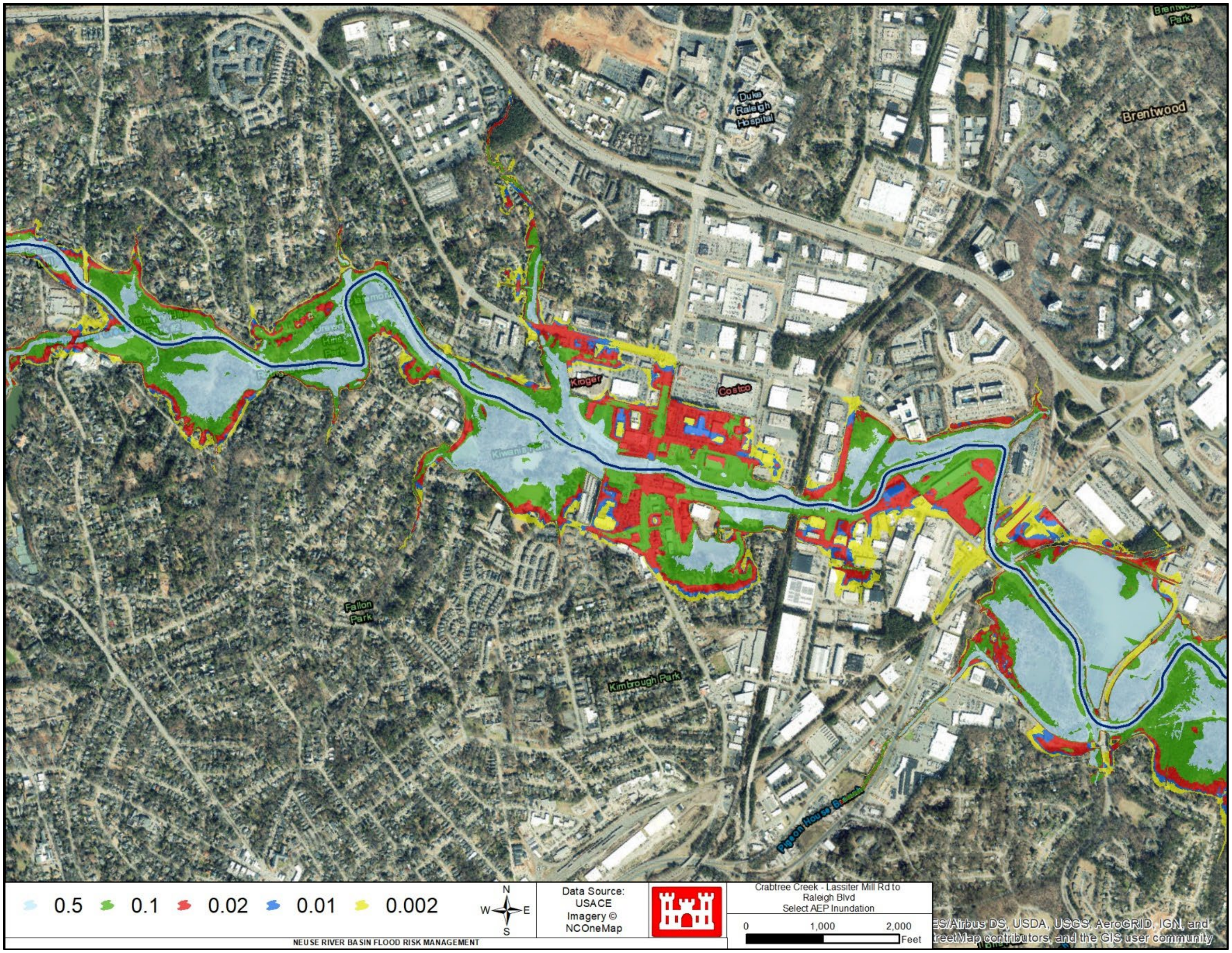

Figure 155. Crabtree Creek FWOP Modeled Inundation for Select Design Events from Lassiter Mill Rd to Raleigh Blvd

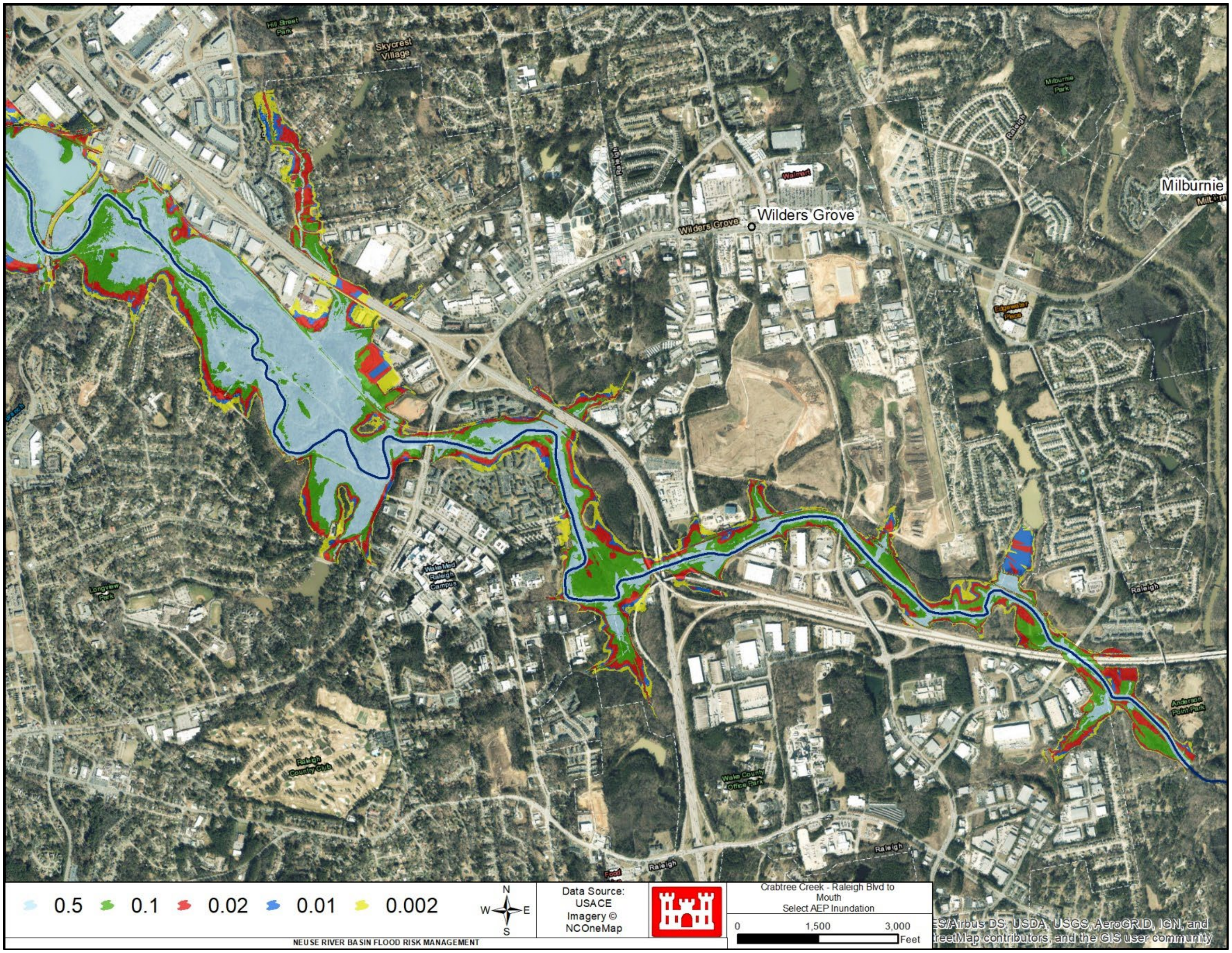

Figure 156. Crabtree Creek FWOP Modeled Inundation for Select Design Events from Raleigh Blvd to Mouth

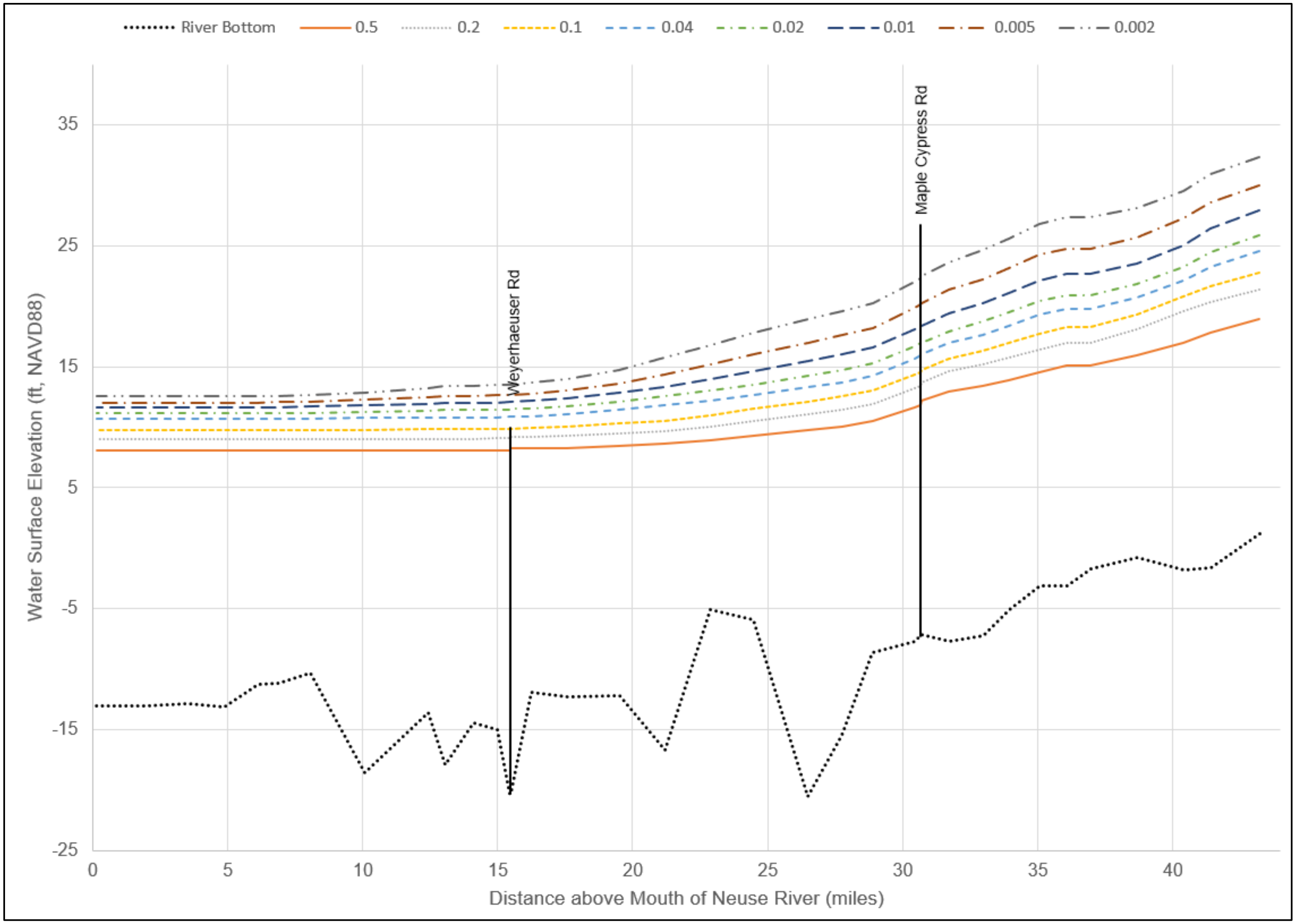

Figure 157. Neuse River FWOP Modeled Water Surface Profiles for Select Design Events from NC-55 to Mouth in Craven County

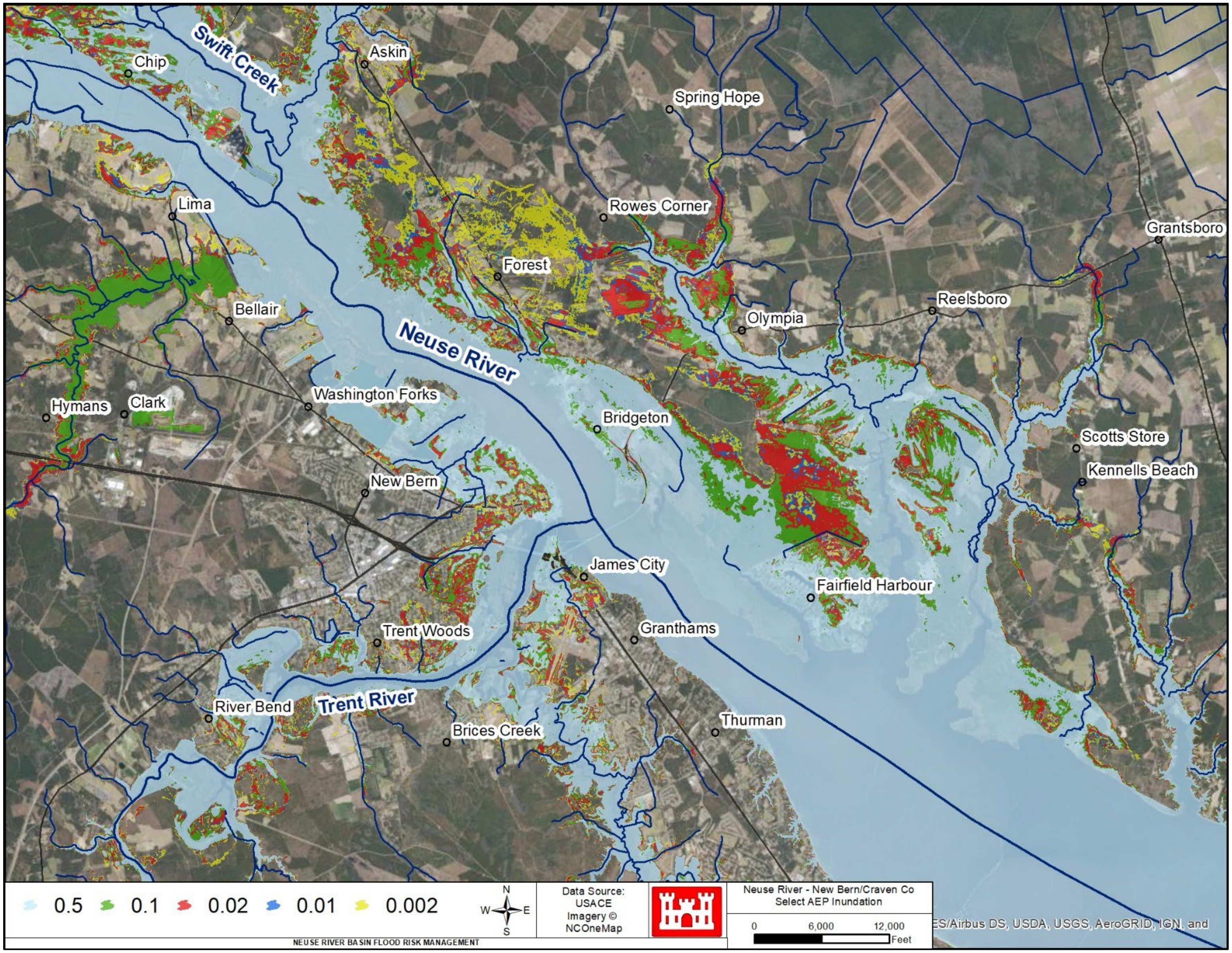

Figure 158. Neuse River FWOP Modeled Inundation for Select Design Events near New Bern, Craven County

7 Flood Risk Management Measures

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

7.1 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 Neuse 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 the North Carolina Floodplain Mapping Program). 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.

Detention sites were selected based on information provided in existing basin assessment studies (USACE, 1965 & NCEM, 2018), as well as watershed master plans (Marck, 2016), 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 North Carolina Floodplain Mapping Program and North Carolina State University). 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 (data provided by North Carolina Emergency Management, 2017) performed following Hurricane Matthew in 2016 as well as historical documentation related to the initial assessment of Falls Lake Dam (USACE, 1960). Levee 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.

7.1.1 Engineering Regulation 1165-2-21 Screening

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

There were multiple tributaries to the Neuse River that have documented flooding concerns at the state and local community level. During this study's screening process NCDOT and other state agencies were undertaking assessments of localized flooding in the communities of Smithfield, Goldsboro, and Kinston (*Evaluating the Capacity of Natural Infrastructure for Flood Abatement at the Watershed Scale: Goldsboro, NC Cast Study*, 2020, *Flood Abatement Assessment for Neuse River Basin, 2020, and Identification and Prioritization of Tributary Crossing Improvements, 2019*). These assessments focused on Buffalo Creek and Spring Branch in Smithfield, Big Ditch, Billy Bud Creek, and Stoney Creek in Goldsboro, and Adkins Branch, Jericho Run, and Taylors Branch in Kinston and developed tributary crossing improvements to improve flood risk management.

During community outreach for the Neuse River basin study, additional streams were considered in addition to those included in the state assessments: Contentnea Creek South Tributary in Grifton, Jack Smith Creek in New Bern, Goose Creek, Ellerbe Creek, and South Ellerbe Creek Tributary in Durham, and Fork Swamp in Winterville. Early measures visualized for implementation, prior to quantitative analyses and economic consideration, were in line with state interests (ex. focus on tributary crossings) in addition to preserving evacuation routes and overall efficiency of road networks. Road berms and/or road raises were examples of potential measures that would scale well to these smaller watershed areas.

All the forementioned tributaries were affected by 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. Buffalo Creek and Spring Branch in Smithfield were screened from further consideration in their entirety. Prior to screening, NCFRIS was utilized to see if enough structural damages were occurring at the tributary confluences with the Neuse River mainstem to justify formulating measures based on the more significant mainstem flood inundation. However, Spring Branch and Buffalo Creek were ultimately screened because there did not appear to be sufficient existing damages near the confluences. Similarly, Billy Bud Creek and Stoney Creek in Goldsboro, Contentnea Creek South Tributary in Grifton, Jack Smith Creek in New Bern, Goose Creek, Ellerbe Creek, and South Ellerbe Creek

Tributary in Durham, Fork Swamp in Winterville, and Jericho Run and Taylors Branch in Kinston were screened from further consideration in their entirety.

At this preliminary screening level, upon ER 1165-2-21 application, there appeared to be sufficient structural damages occurring in Big Ditch in Goldsboro, NC, and Adkins Branch in Kinston, NC. 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. It ultimately demonstrated that the Big Ditch and Adkins Branch study areas were unlikely to possess enough damages to support any structural measures. As such, the two tributary study areas were effectively screened at this point and no structural FWP modeling was conducted.

7.2 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 NCFRIS was vital in helping identify vulnerable structures within established effective and/or preliminary FEMA flood zones. The NCFRIS utility 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 North Carolina. The inventory was taken in the mid-2000s and included numerous structure attributes such as building footprint, foundation type, and estimated first floor elevation. In general, first floor elevations were derived from either LiDAR or an averaged vertical distance above adjacent LiDAR topology. An example of the NCFRIS is shown in Figure 159.

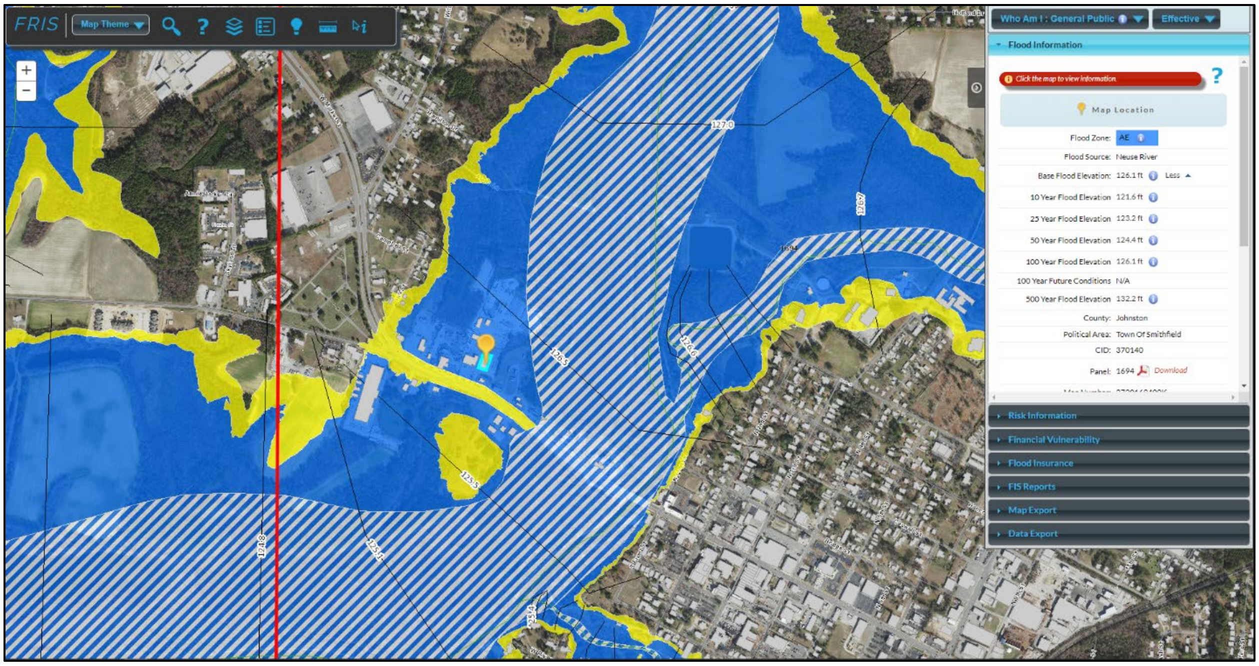

Figure 159. Screenshot of FEMA Flood Zones within the North Carolina Flood Risk Information System

A number of measures screened in this section were located in the tidally influenced coastal area of the Neuse 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 Neuse River basin study. In-depth, compound event analysis is warranted because coastal hazards from hurricanes and extreme extratropical storms can include storm surge, waves, wind, rainfall, compound coastal-inland flooding, seiche, and extreme tides, among others. Climate change and sea level rise are expected to significantly exacerbate coastal flooding in the upcoming decades. These coastal hazards can threaten the lives of millions of people living in coastal regions, and devastate coastal communities and infrastructure, resulting in profound adverse social, economic, and environmental impacts. Consequently, it was determined that 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.

7.2.1 New Detention Structures

The measure involving new construction of large-scale detention structures was the largest risk driver of the initial array. Detention sites within the Neuse River basin has also been extensively investigated historically by multiple agencies, with the most recent investigation being completed by the State of North Carolina as part of their Neuse River Basin Flood Analysis and Mitigation Strategies Study (NCEM/NCDOT, 2018). This study detailed 5 proposed detention facilities within the Neuse River basin in multiple configurations related to how the sites would be managed (ex. wet versus dry detention). These 5 sites would be considered new construction and all but one site is located along a tributary to the Neuse River mainstem. A map of detention structure locations from this 2018 report is shown in Figure 160. Some of these proposed sites were also investigated by USACE as part of the initial Fall Lake Dam reconnaissance study in the 1960s.

The “Swift Creek” site near Smithfield, NC lacked a natural pinch point in the surrounding natural terrain which is typically sought after in dam construction. Consequently, its dam embankment was rather long at several thousand feet in length, depending on a wet/dry scenario. It was also located in an area that has multiple rare, threatened, or endangered aquatic animals, concerning environmental considerations. The 2018 NCEM report cited a concern for sedimentation given a limited permanent pool depth (average >= 10 feet). Due to the generally adverse project site, which presented engineering challenges, and environmental considerations, “Swift Creek” was screened from further consideration.

The “Neuse River Main” site was located in the very wide floodplain between Smithfield, NC and Goldsboro, NC. Due to this floodplain width, the proposed dam length was >5 miles. Furthermore, the dam embankment would be located within the Coastal Plain province and its reservoir would be shallow with an average depth of <4 feet. Due to the overall engineering challenges with this site, “Neuse River Main” was screened from further consideration.

The “Beulahtown” site was also similar in that its reservoir would only have an average depth of <5 feet with a dam length of nearly 1 mile. Sediment loading within its reservoir was a noted concern in the report. Due to these engineering concerns, “Beulahtown” was screened from further consideration.

The remaining explored sites, “Baker’s Mill” and “Wilson’s Mill” were screened by considering the 2018 report’s economic results. According to the 2018 report, the “Wilson’s Mill” site was only able to produce positive benefit-to-cost ratios when configured with the 3 forementioned screened sites. Furthermore, there was concern about the ability to maintain sufficient flood release operations from the upstream Falls Lake Dam without negatively impacting conditions at this proposed site, given its limited storage capacity and elongated detention shape within the narrow floodplain. Finally, the “Baker’s Mill” site was not successful in producing a positive benefit-to-ratio as a standalone site, and as such, it was screened out. In addition to the screening criteria above, the 2018 report noted that the benefits calculations carried out did not consider relocation and elevation projects that have been performed and will be performed related to Hurricane Matthew recovery efforts. Furthermore, there was also overall concern expressed about the ability of these proposed detention structures to meet USACE dam safety regulation (ER 1110-2-1156).

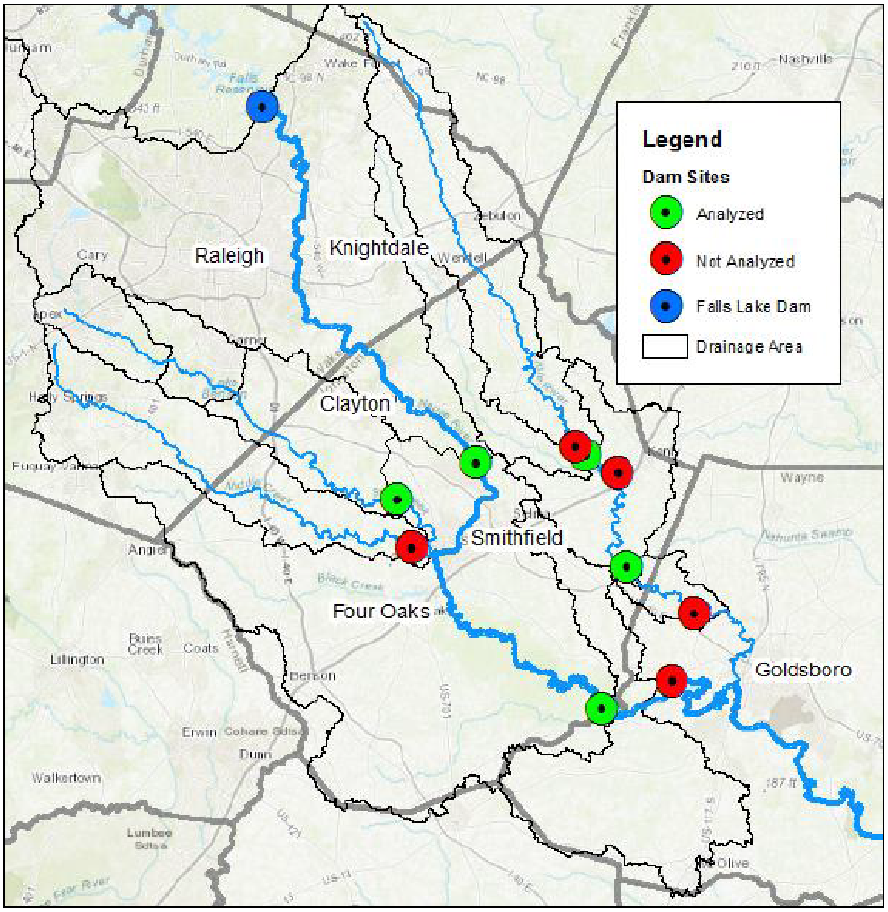

Figure 6.1.1: Potential Detention Storage Sites and Drainage Area Delineations

Figure 160. Locations of Assessed Detention Structure from NCEM Neuse Basin Report

7.2.2 Existing Critical Detention Structure Removal

From a previous study collaboration between USACE SAW and the North Carolina DEQ – Dam Safety Section a number (>150) of medium size (per NC Dam Safety Law of 1967) or larger detention structures were identified within the Neuse River basin (SAW FPMS, 2019). The majority of these structures were privately-owned or maintained by a local community/agency. An assessment of these sites showed that a subset is in a state of disrepair and/or have the potential for failure during a severe flood event. Through removal of at-risk detention structures, it was theorized there would be an improvement to life safety risk. Uncertainty in available data increased as this measure was further investigated due to the inconsistent levels of engineering detail that went into structure construction. There was also concern in induced impacts as a result of removing detention structures in the form of adverse environmental impacts and sedimentation downstream of structure sites. In addition, removing these detention structures may also increase the existing flooding depth and/or velocity for areas downstream of the site. Due to these concerns, this measure was screened from further consideration.

A specific dam outside of the collaboration effort described above was also considered for removal. Lassiter Mill Dam along Crabtree Creek in Raleigh was selected for removal during initial screening. The local community had expressed interest in removal of this structure at various points during the last half century but for various reason the structure has remained in place. Lassiter's Mill was originally utilized for minor power development but has not been operated in such a manner for multiple decades. Furthermore, its value for recreation was limited given its location along Crabtree Creek. The dam is located just upstream of Lassiter Mill Road and is approximately 2 miles upstream from some of the most flood-prone overbank areas along Crabtree Creek. The dam itself is a concrete structure, roughly 9 feet in height taken from the downstream toe with a low-flow weir near its abutments. It has no form of flow regulation other than simple overtopping. It still serves to form an upstream pool that backs water up several miles, most evident during low flow conditions. It is not uncommon for the low-head dam to be overtopped during a moderate rainfall event. Given its size, it is unlikely to pose a significant life safety threat immediately downstream if the structure were to fail. The dam does serve to reduce flows downstream due to its permanent backwater effects. Therefore, if the dam were removed it would potentially impact flooding conditions downstream. Based on a review of existing structures within preliminary FEMA flood zones surrounding the site, there appeared to be more relative flood risk downstream that would be negatively affected by dam removal. Due to this reason and the assumed low life safety risk, this site was screened from further consideration.

7.2.3 Bridge Span Modification along Neuse River Mainstem

This measure involved modification of existing bridges to increase their span opening over the width of the Neuse River mainstem. There were multiple crossings identified along the river where constricted flow may have influenced upstream flooding.

At the time of initially investigating this measure there were multiple similar efforts being undertaken by the State of North Carolina. The 2018 Neuse River Mitigation Strategies Report and the 2020 NCDOT Flood Abatement Assessment were also looking into ways of increasing conveyance through major bridge structures over the Neuse River mainstem.

During this preliminary screening process hydraulic modeling was completed for these state efforts with data and results being shared with USACE SAW. Overall, a comprehensive approach to improving conveyance at key river crossings through structural modifications provided only minimal flood reduction with changes in upstream water surface elevation of less than a foot, and often less half a foot during a Hurricane Matthew-scale event (NCDOT, 2020).

The general intent of the proposed bridge improvements by the NCDOT report were simulated in the Neuse River basin study hydraulic model to validate their findings. Improvements proposed at certain locations, such as at railroad crossings, were not as extensive as described in the NCDOT report, so there were minor differences in WSEL improvements at the various bridge crossings between the two study models. A common effect experienced after bridge improvements that wasn't explicitly detailed in the NCDOT report was induced WSEL as a result of removing existing floodplain constrictions. Hydraulic performance using the Neuse River basin study unsteady HEC-RAS model showed a 0.2 to 0.3-foot WSEL increase that persisted from immediately downstream of I-95 in Smithfield, NC downstream to Arrington Bridge Rd in Goldsboro, NC. This effect was also seen at other bridge improvements in Kinston, NC. Due to the limited reduction in WSEL upstream of improved bridges, upon validating results from the NCDOT study, and concern for the induced flooding downstream of improvements, this measure was screened from further consideration.

As stated in the last paragraph of Section 7.2, components of this measure that were within the tidally influenced region of the Neuse River basin were not assessed using coastal modeling tools. These components may warrant re-assessment as part of a study specific to tidally influenced areas of the Neuse River basin.

7.2.4 Neuse River Channel Modification near Kinston, NC

This measure was documented in the 2018 Neuse River Basin Flood Analysis and Mitigation Strategies Study. It involved channel modification of approximately 11 miles of the Neuse River mainstem in the vicinity of Kinston. There were multiple concerns related to this type of measure, given its large footprint and area of effect, that the report acknowledged, and were echoed in this preliminary screening assessment. There would be potential for significant operations and maintenance required for this measure to function properly. Sediment transportation would also be a significant concern and would involve considerable effort to fully understand the hydrodynamics in this portion of the Neuse River basin, given its location within the Coastal Plain province. There may be increased chances of erosion and bank stability issues related to increased flow velocity, and induced damages downstream of the measure. There would most likely be major environmental consideration related to this measure, however, due to the engineering concerns during this preliminary screening, it was not carried forward for further consideration.

7.2.5 New Levee at Seven Springs, NC

This measure was documented in the 2018 Neuse River Basin Flood Analysis and Mitigation Strategies Study. The Town of Seven Springs appeared to be ideally situated for a levee system. While the town is located in a lower floodplain terrace, south of the Neuse River mainstem, a levee alignment could successfully tie into higher ground both upstream and downstream of the town. Such a levee system could provide a significant improvement to flood risk management. It is noted that while the 2018 report did mention interior drainage, the cost of such a system was not included in their analyses. It is likely that such a system (likely requiring a pumping solution) could be challenging in this location. Otherwise, modeling results and economic assessment from the 2018 report showed a positive benefit-to-cost ratio. However, upon further investigation of this type of structural measure for the Town of Seven Springs, the majority of the town was under consideration for a comprehensive buy-out plan by the State of North Carolina. Such a plan would have a significant negative impact to the potential benefit-to-cost ratio. Based on this ongoing assumption and coordination with the North Carolina Office of Recovery and Resiliency (NCORR), this measure was screened from further consideration.

7.2.6 Floodwall near New Bern, NC

This measure was selected early in the study process, partially due to the potentially significant impact to scope of engineering analyses required to adequately assess and address the flooding problems in the vicinity of New Bern, NC. The study team acknowledged the complex hydrology and hydraulics present at the mouth of the Neuse River, Trent River, Pamlico Sound, and other smaller tributaries. This area of the basin is subject to both riverine and coastal flooding. Assessment of compound flooding from both sources would necessitate specialized modeling tools and was assumed to be beyond the capabilities of traditional riverine modeling (HEC-RAS). A preliminary screening exercise was conducted to determine the likelihood of measure viability. Existing data was utilized from SACS to help facilitate this assessment. SACS data included a library of measures and related costs at a per unit level. This dataset allowed the team of apply an array of flood risk management measures for a site-specific design. A comprehensive design selected for the overbank floodplain near New Bern consisted of a permanent structural barrier (floodwall) that would conservatively prevent floodwaters from entering developed land for events up to the 0.01-AEP. Two separate rough barrier alignments were proposed, a 7,000 linear foot feature adjacent to downtown New Bern (west bank) and a 6,000 linear foot feature adjacent to the Town of Bridgeton (east bank). NCFRIS was used to designate the 0.01-AEP flood extents, based on the FEMA Effective Base Flood. Measure performance was determined by eliminating Hazards United States (HAZUS) damages by census block that were confined to the leveed area behind the barriers. A follow-on measure was investigated related to placement of a flood barrier slightly upstream of the downtown area along the right bank. The intent in this alignment was to prevent backwater from propagating into the Jack Smith Creek tributary and causing flooding to the Duffy Field area. Due to the lack of relief in the nearby terrain it would challenging to tie in a floodwall structure to natural high ground. This constraint resulted in a length of wall nearly equivalent to the downtown portion. Furthermore, volume of floodplain along the right bank gave significant concern for adequate interior drainage if a structure were possible.

Based on this preliminary economic assessment, cost to benefit ratio appeared to be disproportionately low. Furthermore, no costs related to interior drainage systems were estimated, and it was assumed inclusion of such estimate would only further reduce the cost to benefit ratio. Due to the above analysis, it was recommended that this measure be screened from further consideration.

As stated in the last paragraph of Section 7.2, components of this measure that were within the tidally influenced region of the Neuse River basin were not assessed using coastal modeling tools. These components may warrant re-assessment as part of a study specific to tidally influenced areas of the Neuse River basin.

7.2.7 Trent River Channel Modification in Jones County, NC

This measure was selected based on initial community outreach with the Towns of Pollocksville and Trenton, NC as well as Jones County, and follow-up coordination. These communities are located along the Trent River and have experienced flooding problems caused by both intense localized rainfall and wind-tides or storm surge associated with tropical storms or hurricanes (FEMA, 2020). The Trent River has a drainage area of 550 square miles at its mouth in New Bern, NC. The communities can be exposed to backwater flooding due to their proximity to the mouth of the Neuse River and Pamlico Sound estuary. They have experienced prolonged or delayed flooding following events when the Trent River is unable to adequately drain and return to normal water levels. According to local feedback following recent significant flood events (Hurricane Matthew, 2016 and Florence, 2018), the nature of overbank flooding is sensitive to both direction and duration of the storm system in the immediate Trent River area as well as the rest of the Neuse River basin. The communities had expressed interest in assessing the measure of Trent River channel modifications to determine its viability within the Neuse River basin study. Channel modifications were to be in the form of widening and/or dredging. A preliminary hydraulic assessment was conducted using existing FEMA-based HEC-RAS modeling. This simplified approach assumed no changes in flow regime or sediment transport, stable channel geomorphology, and minimal environmental considerations. The assessment results would help direct the PDT in the further scoping of hydrology and hydraulics, and economic efforts necessary to perform detailed measure analysis. Channel widening templates of a 50-foot and 75-foot bottom width were proposed for a length of approximately 10 miles of the Trent River. Channel dredging templates focused on creating a consistent slope, often needed near bridge structures, and proposed several feet of material excavation along the channel bottom. Dredging was limited by downstream constraints of the Neuse River and Pamlico Sound. Assessment results showed <0.5-foot reduction in water surface elevation for the 0.01-AEP event. The most significant WSEL reductions were experienced during the more frequency, less severe events (i.e. 0.1-AEP) where the flood waters were more confined to the river channel and consequently would have less overall impact related to existing structural damages.

The efficiency of dredging decreased as the severity of flood event increased and involved more of the overbank floodplain. Based on the measures' minor effect and conservative assumptions, it was screened from further consideration.

As stated in the last paragraph of Section 7.2, components of this measure that were within the tidally influenced region of the Neuse River basin were not assessed using coastal modeling tools. These components may warrant re-assessment as part of a study specific to tidally influenced areas of the Neuse River basin.

7.2.8 Dispersed Water Management

Dispersed Water Management (DWM), also referred to as Water Farming, is a practice that provides temporary shallow water storage, retention, and detention through the use of existing infrastructure and simple structures (weirs, berms, and culverts). Water is retained on-site and removed through natural means of evaporation, transpiration, or seepage (SFWMD, 2014). An example of this practice is Water management entities in Florida that work with farmers who are paid to keep stormwater on their properties and receive water from other areas to store on their properties. Assessment of this type of measure was limited given its application in existing USACE project portfolios. The presence of expansive, low-lying floodplains characteristic of Florida seemed crucial in this measure's viability. While the Neuse River basin contains some floodplain areas similar to that of the Everglades in Florida, they are confined to the lowest portions of the basin nearest to the Pamlico Sound. Another difference between the two locations is the extensive system of existing water management features in Florida operated and maintained by water management districts, where water surface elevations are maintained depending on the time of year. Lastly, DWM appeared to primarily impact water quality and groundwater conservation, in addition to flood-related issues. With an assumed preferred measure location near the Pamlico Sound, it was difficult to quantify how any improvements to flood risk management would be transferable to areas most vulnerable to flooding that exist upstream in the basin. There were numerous considerations beyond just engineering in implementing this measure, though due to the technical reasoning described above this measure was screened from further consideration for this study.

As stated in the last paragraph of Section 7.2, components of this measure that were within the tidally influenced region of the Neuse River basin were not assessed using coastal modeling tools. These components may warrant re-assessment as part of a study specific to tidally influenced areas of the Neuse River basin.

7.2.9 Johnston County Wastewater Treatment Plant Levee

This measure was selected to represent additional FRM improvements that would be made to the existing Johnston County Wastewater Treatment Plant (WWTP). The plant is located near Smithfield, NC, and is near the southeastern bank of the Neuse River.

The site is entirely within the FEMA 0.01-AEP flood zone and partially in the regulatory floodway. Prior to coordination with the WWTP, review of the site within NCFRIS showed some degree of existing earthen levee embankment surrounding the operations. The current status of the site was confirmed during a coordination call with Johnston County Public Utilities (phone conversation, Feb-2021). The WWTP had long-term goals of relocating the primary plant operations to a site completely outside of the floodplain, and in the interim had secured FEMA grant funding to engineer and construction more robust FRM features for the current plant. Conceptual drawings supplied to the PDT proposed a parapet wall on top of the existing earthen levee to extend overtopping frequency. Due to this existing grant and engineering effort in place, this measure was screened from further consideration.

7.2.10 Cherry Research Farm Levee Repair

This measure was proposed based on previous coordination with Cherry Research Farm and the City of Goldsboro, NC. Cherry Research Farm has a levee system meant to provide FRM improvements for several structures on their campus, located west of Goldsboro city limits. The levee was damaged and partially breached during Hurricane Matthew in 2016. USACE SAW District conducted a site visit in 2017 to investigate potential repair as part of a Continuing Authorities Program or similar effort. The PDT reached out to the campus to determine the status of levee repair as of 2020. It was confirmed that the levee system was already undergoing repair outside of USACE partnership. Therefore, this measure was screened from further consideration.

7.2.11 Improvements To Rose Lane Bridge Over Walnut Creek

This measure was selected based on a cursory assessment of vulnerable residential clusters using NCFRIS. The communities of Rosalynn Place and Maplewood Forest are located off of Rose Lane in southeast Raleigh, NC. Rose Lane, to the north, is the only means of egress for the residents of these communities as the inner I-40 beltline demarcates the southern edge of the residential area. Rose Lane crosses over Walnut Creek approximately 1,000 feet north from the intersection of Rose Lane and Jimmy Carter Way. If this crossing were to be inundated by a flood event, there would be a potentially significant impact to evacuation and/or emergency services accessibility. As there appeared to be limited structural damages due to flooding, this measure was developed to improve life safety risk, rather than traditional economic justification. During coordination with the City of Raleigh, the city acknowledged this flood risk and as of January 2021, were pursing bridge improvements with conceptual design already completed. This measure was screened from further consideration due to this information and challenges related to non-economic justification.

7.2.12 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 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-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.

7.2.13 Neuse River Channel Modification near Smithfield, NC

This measure was selected based on community outreach with the Towns of Smithfield and Four Oaks. Anecdotal evidence was provided that the Neuse River mainstem had lost a significant amount of flow capacity due to sedimentation within the channel. This flooding concern may have also been related to the natural floodplain constriction south of Smithfield, in addition to multiple bridge spans over a short distance. No recent channel surveys were provided, nor could any new survey be conducted as part of this preliminary screening iteration. Neuse River channel Bathymetry surveyed for the FEMA effective hydraulic modeling showed a moderately consistent slope of about 0.03%. A review of the 0.01-AEP water surface gradient within the FEMA effective model revealed differing segments of sloped water surfaces separated by bridge openings. The number of bridge spans in close proximity made it technically challenging to apply a modified template that included excavation below existing grade. To do so would potentially involve structural modification to a number of bridges. The floodplain in this area did not appear to be heavily populated with most structures outside of the flood hazard area, according to NCFRIS. Based on these limited potential damages, and the inability to apply a comprehensive excavation profile due to the number river crossings, this measure was screened from further consideration.

7.3 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.

7.3.1 Neuse River Channel Modification in Kinston, NC

The proposed channel modification is located within the left and right overbanks of the Neuse River mainstem as it flows through the City of Kinston, NC. The primary feature involved in this measure was excavation of channel benches that functioned as floodplains and created natural alluvial channel processes. The resulting Neuse River primary flow path would consist of a dominant discharge channel (existing bankfull conveyance) and a floodplain bench. The channel-forming discharge channel would provide the necessary sediment conveyance, while the floodplain bench would provide for design flood conveyance. Two segments of benched channel were positioned along the river's banks with a bottom invert set roughly 2 feet above the water surface elevation expected from an average annual discharge (1.0-AEP). The benched surface included a minor slope away from the river to ensure adequate drainage. The perimeter of the benched surface assumed 3H:1V side slopes to tie back into existing grade. A total channel bench length of almost 3 miles extended from the downstream face of US-11 (King St) bridge to the upstream face of the railroad bridge that parallels Young St within the city limits. A typical cross section depicting a channel bench placed within the left overbank of a stream's floodplain is shown in Figure 161.

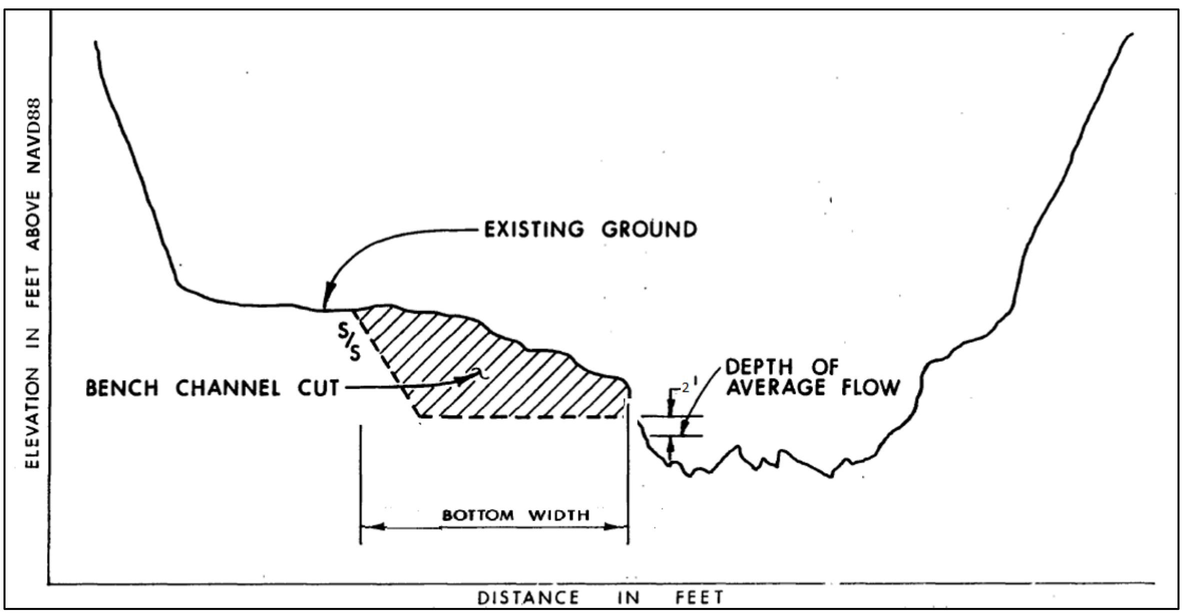

Figure 161. Typical Cross Section of a Channel Bench

The first bench segment (RB01) was placed within the right overbank floodplain between the US-11 and HWY-258 (S Queen St) bridges and had an approximate length of 1.3 miles. RB01 had an average benched width of 500 feet, based on a footprint width that ranged from 100 feet near the tie-in points at the bridge embankments up to 900 feet near the midpoint of its length. There were some areas within the bench footprint that required about 9 feet of vertical cut in order to bring the existing surface (based on QL2 LiDAR) down to the final design grade. There were also several areas within the RB01 footprint that required about 4 feet of vertical fill to bring low-lying floodplain up to the final design grade.

The second bench segment (LB01) was placed within the left overbank floodplain between the HWY-258 and railroad bridges. According to the city, the railroad bridge is co-owned by Norfolk Southern Railroad and North Carolina Railroad. LB01's footprint length adjacent to the river's edge was about 1.5 miles. LB01 had an average benched width of 1,000 feet. There was not a significant deviation from the average width throughout its length due to the wide, unobstructed floodplain in this area. One constraint to LB01's footprint was the presence of a leveed waste-retaining facility off Peachtree St. Some areas within LB01's footprint required nearly 30 feet of vertical cut in order to bring the existing surface down to the final design grade. Though not nearly as significant, some areas within its footprint required about 1.5 feet of fill in order to reach final design grade. An overview of this measure is shown in Figure 162.

Both segments were modeled within the same HEC-RAS geometry by modifying the terrain over a series of cross sections that represented the segment footprints. Manning's roughness values were reduced within the footprint areas to represent improved conveyance due to change in land cover from woody wetland to developed open space. Proposed conditions were simulated under the suite of design storms and inundation footprints were generated in Ras Mapper, as shown in Figure 163.

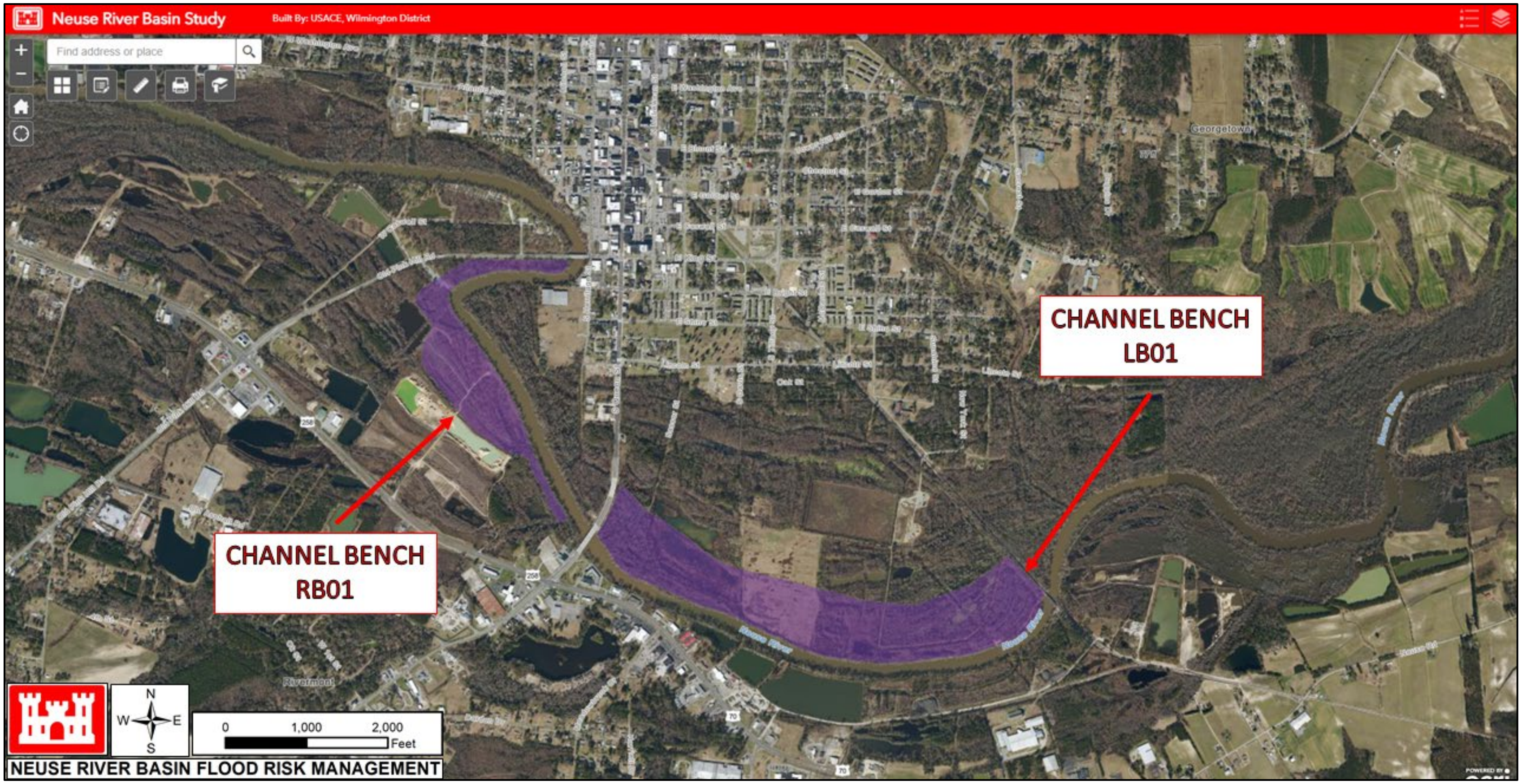

Figure 162. Kinston Channel Bench Overview

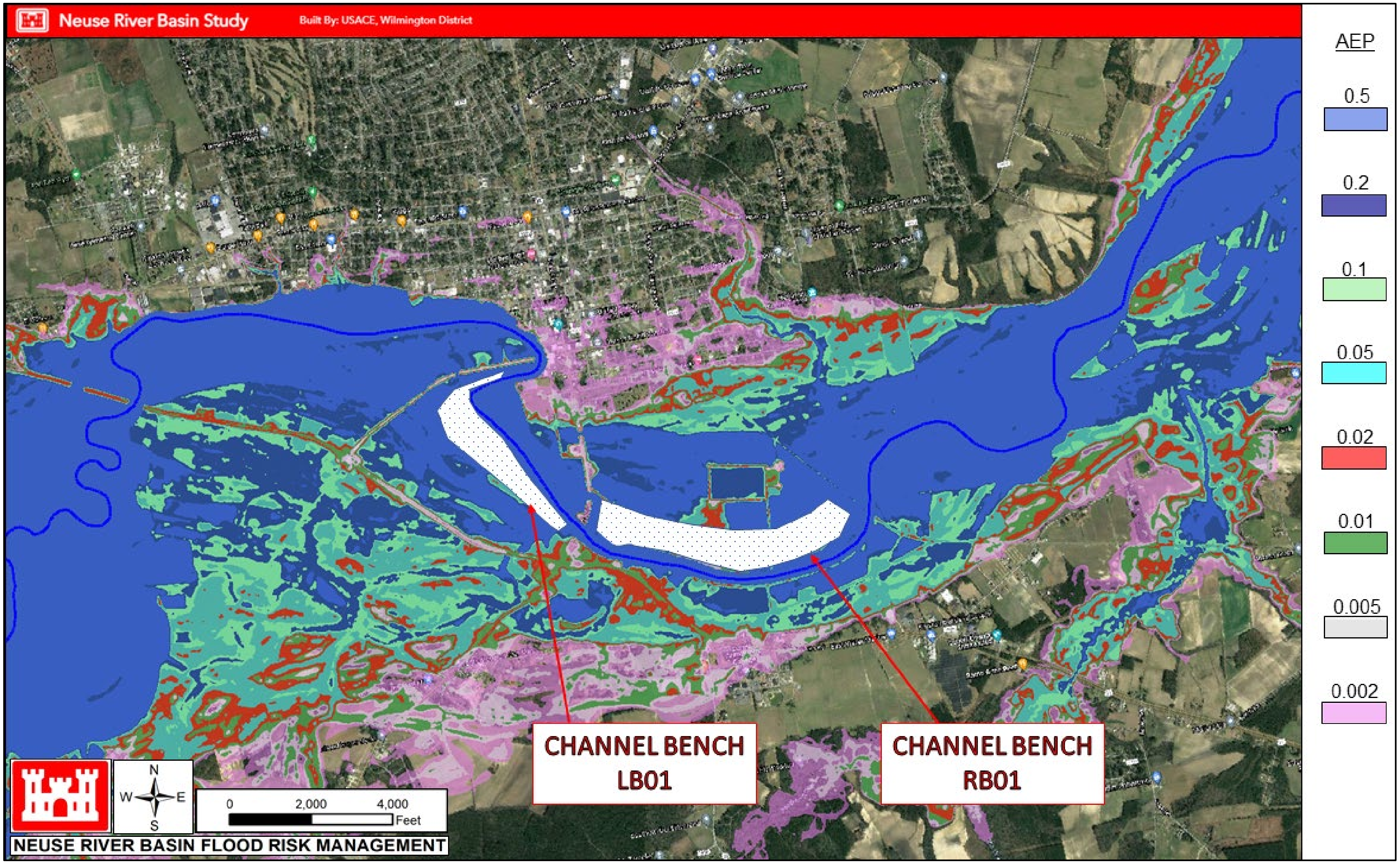

Figure 163. Kinston Channel Bench Measure – Design Storm Inundation

The design storms most frequent, 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 (south of Lincoln City). The majority of structures in these developments have been removed from the floodplain and what is left is a network of abandoned paved roads. 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 city, within the 0.01-, 0.005-, and 0.002-AEP impacted areas, the added benefit from this measure was not as prominent. Water surface profiles for select design storms are shown in Figure 164.

In general, while this measure was effective at reducing flood elevations for the more frequent design storms, it was unable to provide significant WSEL 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.

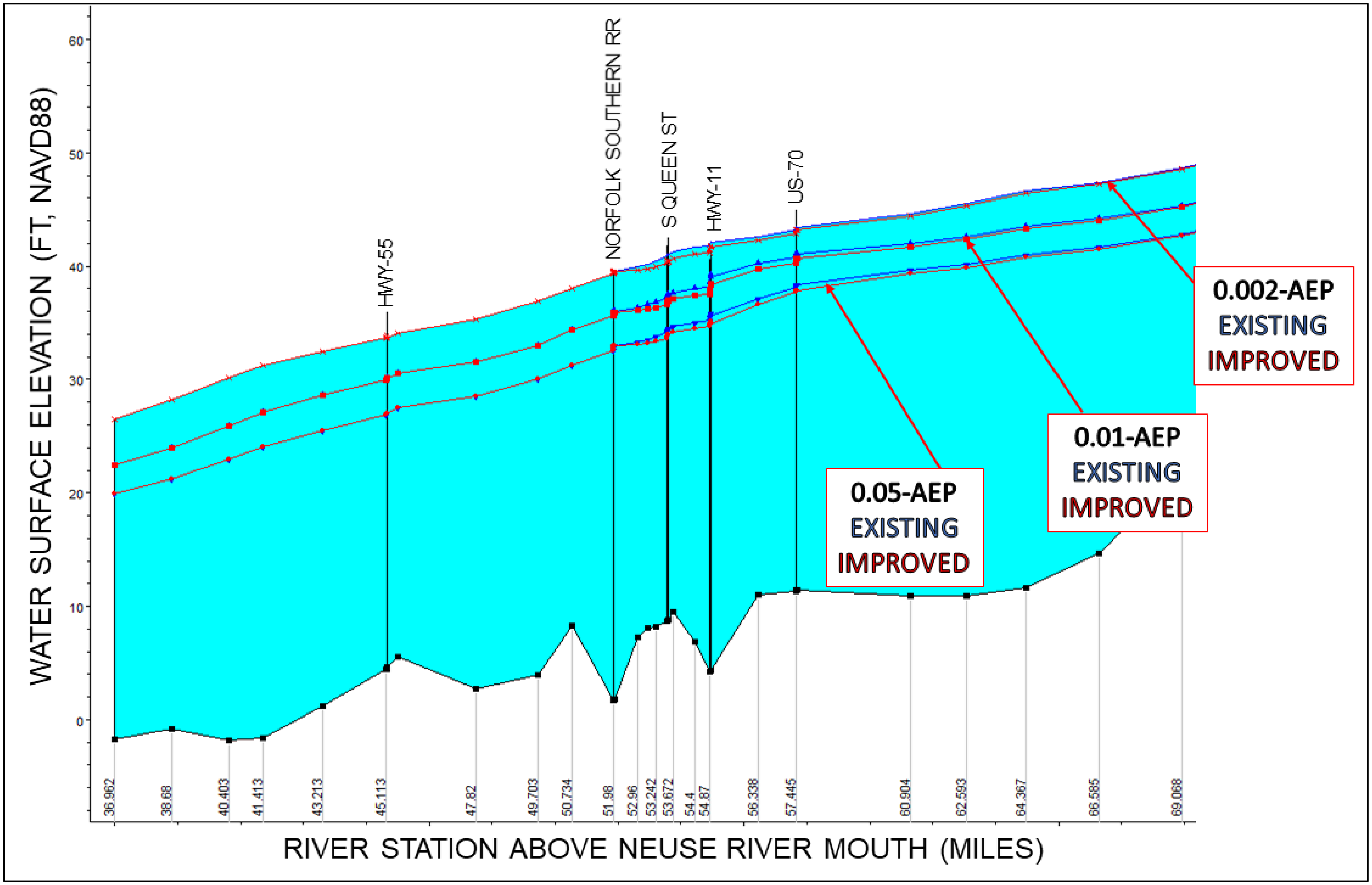

Figure 164. Comparison of Water Surface Profiles for Select Design Storms – FWOP vs. FWP (LB01+RB01)

7.3.2 Hominy Swamp Creek Channel Modification in Wilson, NC

Traditional channel modification was represented by applying a widened channel template at existing grade or by excavating to a new design surface for the channel bottom invert. Based on a review of the existing channel grade in the FEMA effective hydraulic model, there appeared to be a consistent slope throughout most of the study reach with a few exceptions. A 0.5-mile segment of Hominy Swamp Creek, located between the Forest Hills Rd and NC-42 crossings, had a flattened creek gradient relative to segments both up and downstream of it, and there was potential to provide a more hydraulically efficient slope. An averaged 10-ft channel bottom width template that included excavation of roughly 2 vertical feet was selected for assessment. There were two other short segments of the creek that exhibited similar inefficient slopes, located upstream and downstream of the Tarboro St crossing. The same 10-ft channel bottom width template was applied to these segments but with a proposed excavation of about 1 vertical foot in order to reach design grade.

The Hominy Swamp Creek HEC-RAS model was used to apply these channel templates. A new geometry was created that included the three improved channel segments, and simulations were run for the full range of design storms. Manning's roughness value for the channel was set to 0.04. Model results showed there to be a negligible difference in WSEL (<= 0.1-ft) when compared to FWOP conditions across all design storms. Based on these results, channel excavation was screened from further consideration for Hominy Swamp Creek.

Due to historically documented channel incision for Hominy Swamp Creek (Marck, 2016), channel widening was pursed using an alternate design that was not focused on widening the existing channel bottom. Instead, a design template was proposed that focused on overall channel width, up to the top of bank. The proposed channel modification was located within the left and right overbanks of the Hominy Swamp Creek as it flowed through the City of Wilson, NC. The primary feature involved in this measure was excavation of channel benches that functioned as floodplains and created natural alluvial channel processes. The resulting Hominy Swamp Creek primary flow path would consist of a dominant discharge channel (existing bankfull conveyance) and a floodplain bench. The channel-forming discharge channel would provide the necessary sediment conveyance, while the floodplain bench would provide for design flood conveyance. Eleven segments of benched channel were positioned along the river's banks with a bottom invert set roughly 2 feet above the water surface elevation expected from an average annual discharge (1.0-AEP). The benched surface included a minor slope away from the river to ensure adequate drainage. The perimeter of the benched surface assumed 3H:1V side slopes to tie back into existing grade. A total channel bench length of almost 3.2 miles extended from the downstream face of NC-42 (Ward Blvd) bridge to approximately 300 feet downstream of the CSX railroad culvert. Refer to Figure 161 for a typical cross section of this channel bench design. An overview of these measures along Hominy Swamp Creek is listed in Table 77.

Table 77. Channel Modification Details for Hominy Swamp Creek in Wilson, NC

Bench Cut ID	Channel Overbank Side	Location		Footprint Area (sq ft)	Width
		From	To		
BC402	Right	NC-42	Kincaid Ave	290000	100
BC374	Right	Kincaid Ave	Raleigh Rd	150000	100
BC351	Right	Raleigh Rd	Norfolk S. RR	141000	100
BC331	Right	Elizabeth Rd	Park Ave	240000	200
BC326	Left	Elizabeth Rd	Park Ave	90000	100
BC313	Right	Park Ave	Tarboro St	10000	100
BC286	Right	Goldsboro St	Lodge St	130000	250
BC278	Right	Lodge St	Phillip St	280000	150
BC266	Left	Lodge St	Phillip St	120000	250
BC256	Right	Phillip St	CSX RR	110000	300
BC244	Left	CSX RR	Ward Blvd	49000	200

As detailed in Table 77, channel bench segments were separated by bridge and/or culvert structures that crossed over the main flow path of Hominy Swamp Creek. A design constraint of minimizing impacts to existing utilities and infrastructure prevented a more hydraulically efficient merge of segments. Furthermore, most segments were limited to channel and floodplain modification on one side of the creek, leaving the alternate bank in its natural state. Notable exceptions were measure IDs BC326 and BC331 between Elizabeth Rd and Park Ave, and BC266 and BC278 between Lodge St and Philip St. Both sides of the creek were modified in the segment between Elizabeth Rd and Park Ave due to the availability of developed open space that currently existed.

Following field investigation and coordination with state environmental agencies, two channel bench segments were eliminated from consideration. BC374 was removed due to the presence of an existing stream restoration project within the right floodplain overbank (EEP Project No. 180). BC266 was eliminated from the array due to the need for Norris Blvd to remain accessible, a desire expressed by the City of Wilson.

The final nine segments were modeled within the same HEC-RAS geometry by modifying the terrain over a series of cross sections that represented each segment footprint. An example of this geometry modification is shown in Figure 165.

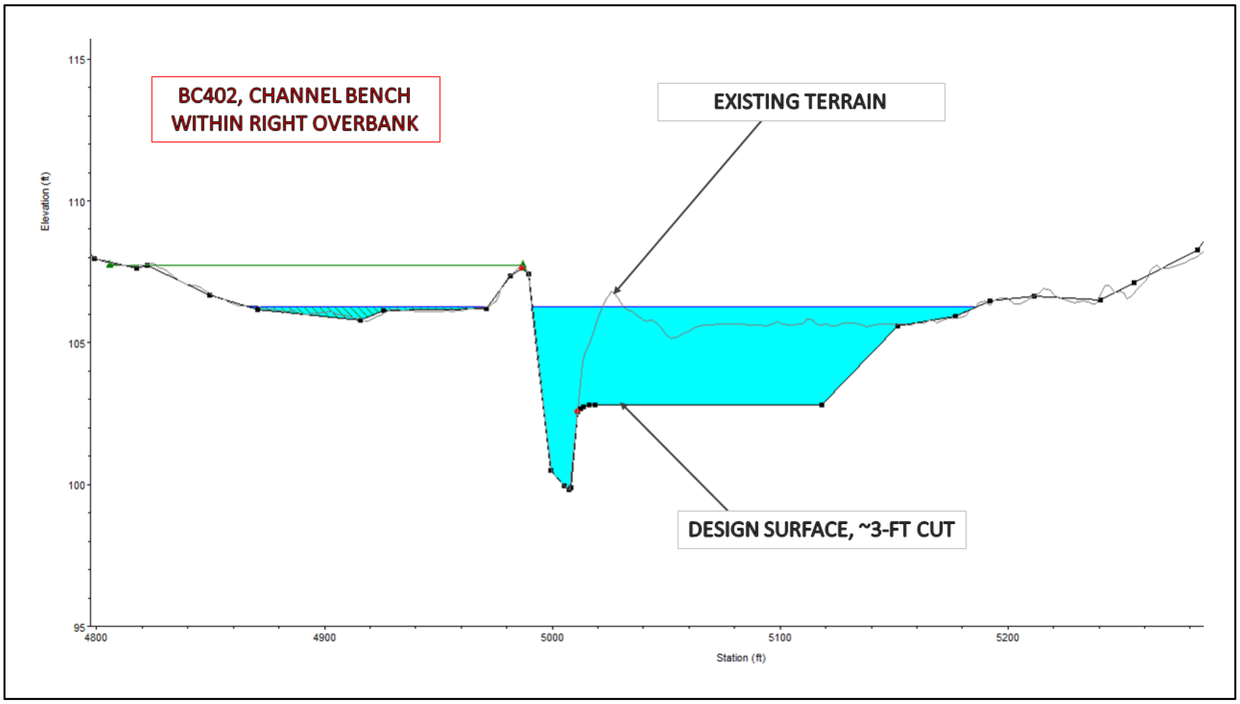

Figure 165. Example of Channel Bench Geometry, Hominy Swamp Creek

Manning's roughness values were reduced within the footprint areas to represent improved conveyance due to change in land cover from woody wetland, herbaceous, and forest to developed open space. Proposed conditions were simulated under the suite of design storms and inundation footprints were generated in HEC-RAS Ras Mapper, as shown in Figure 166 through Figure 173.

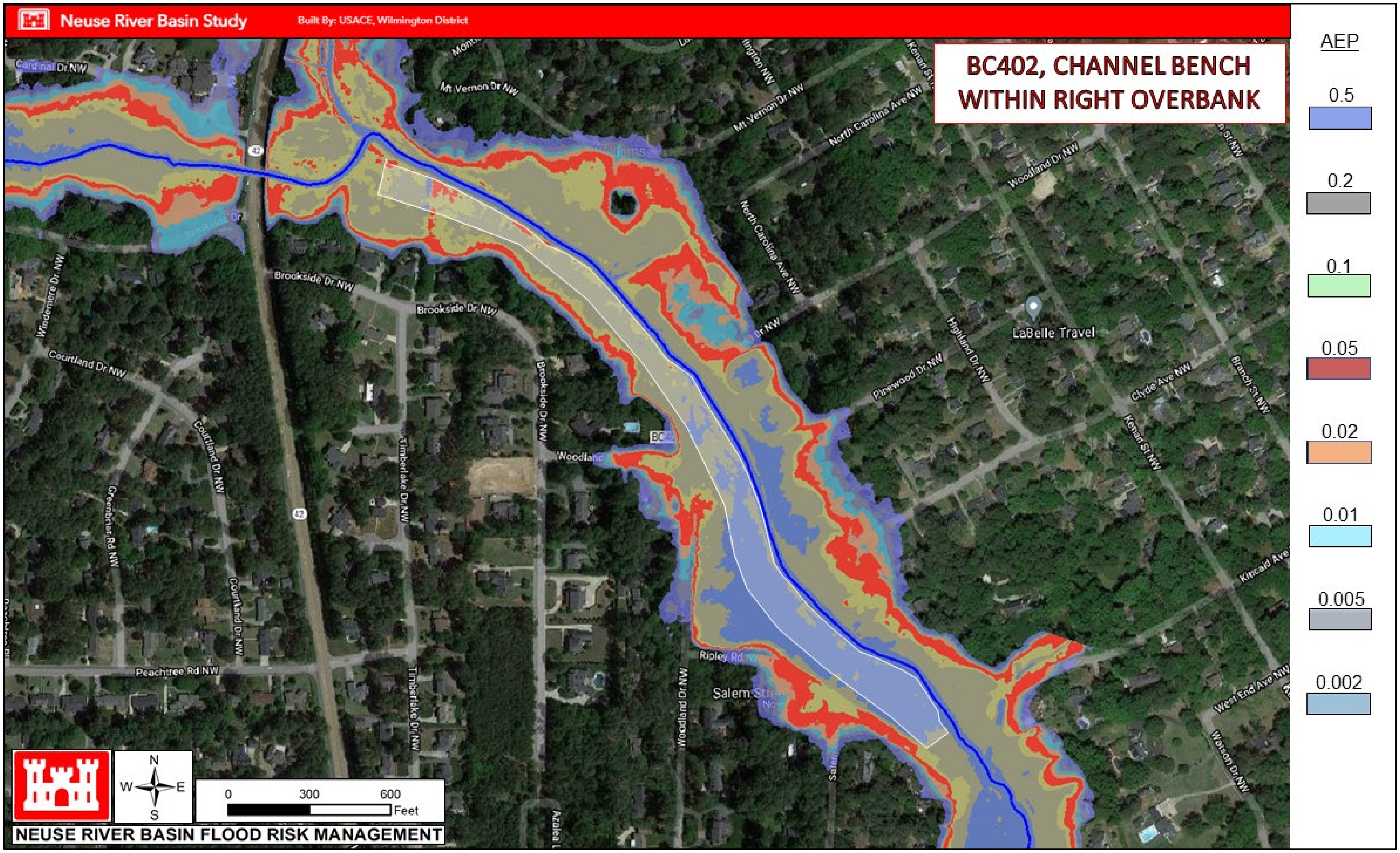

Figure 166. Hominy Swamp Creek Channel Bench BC402 Design Storm Inundation

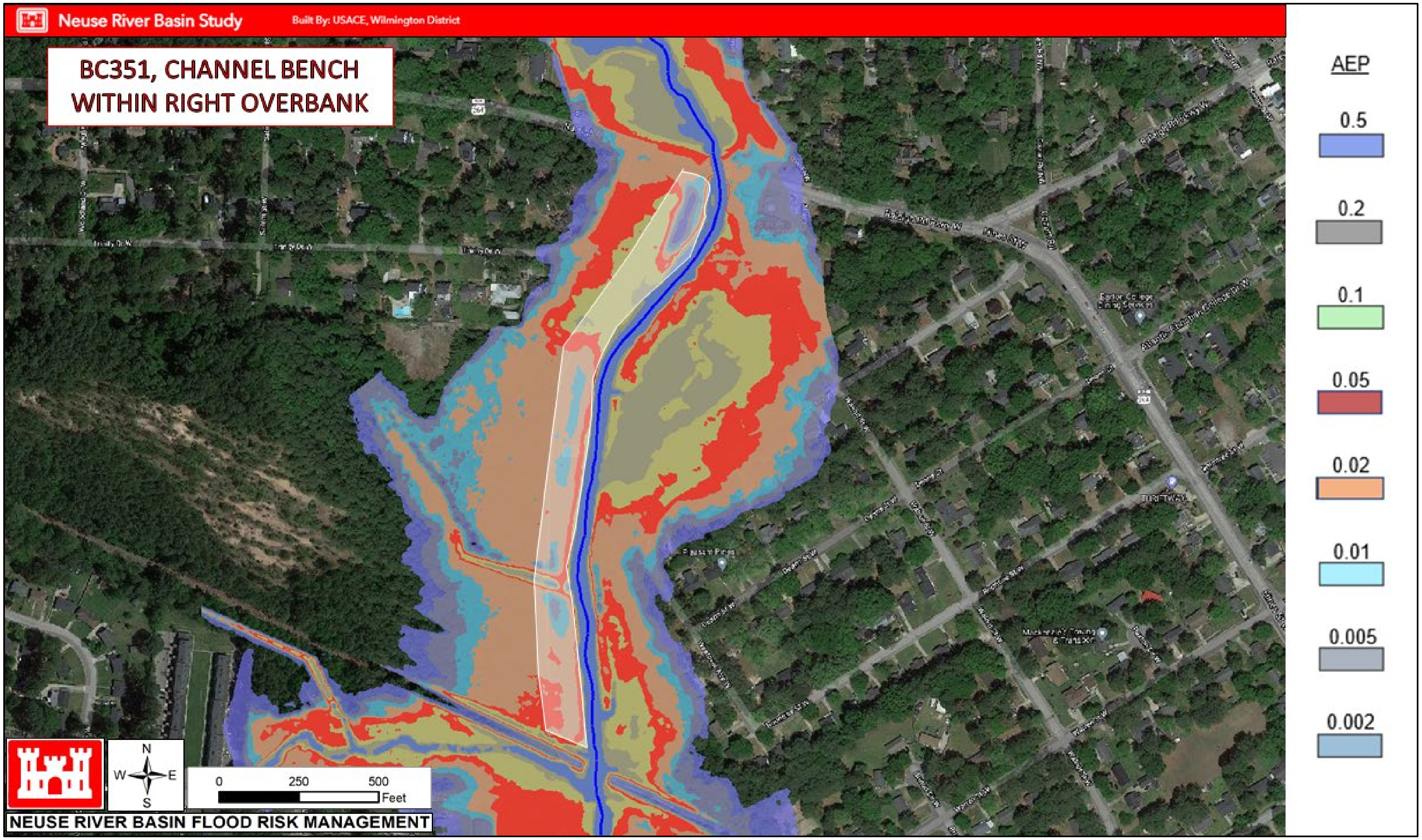

Figure 167. Hominy Swamp Creek Channel Bench BC351 Design Storm Inundation

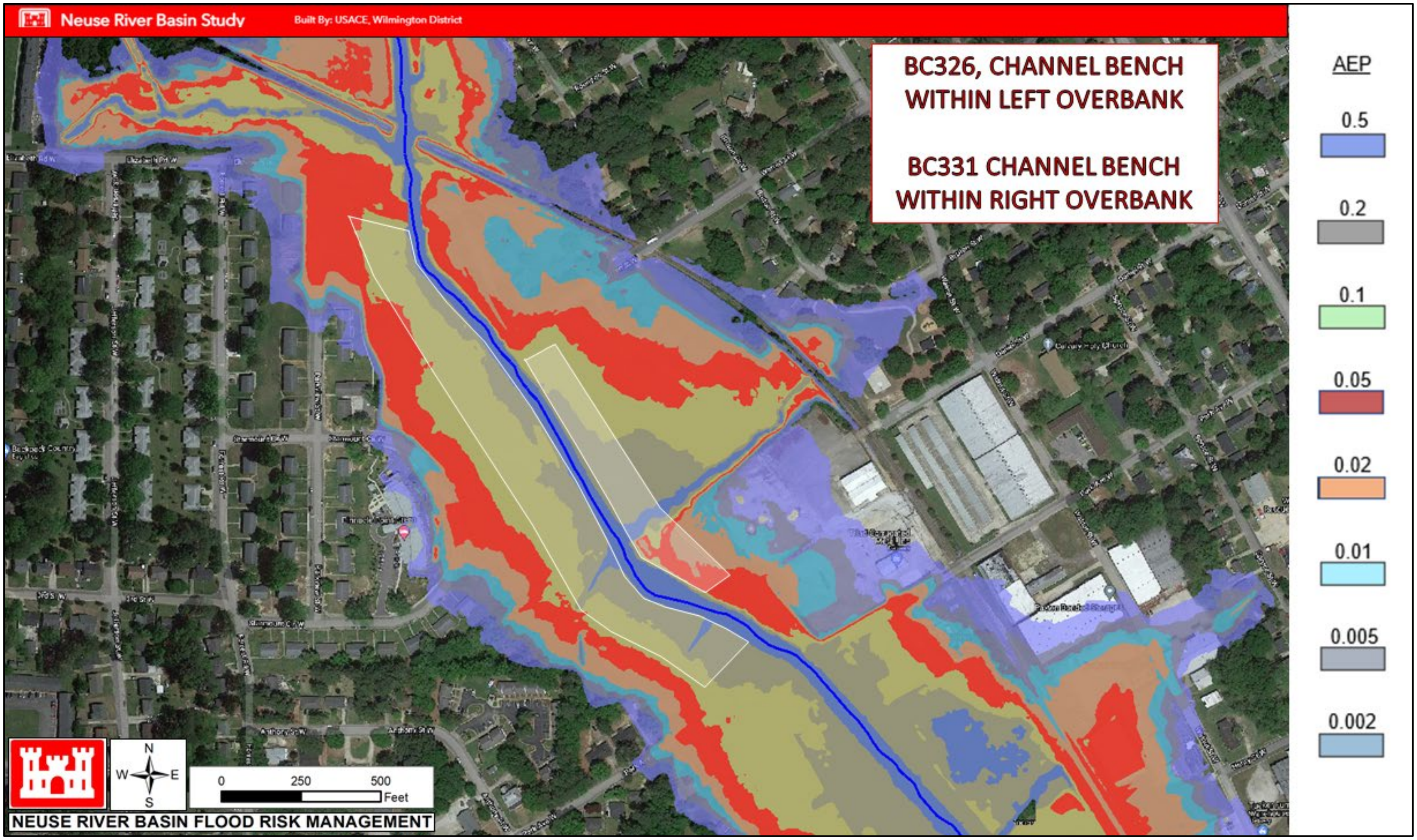

Figure 168. Hominy Swamp Creek Channel Bench BC326 & BC331 Design Storm Inundation

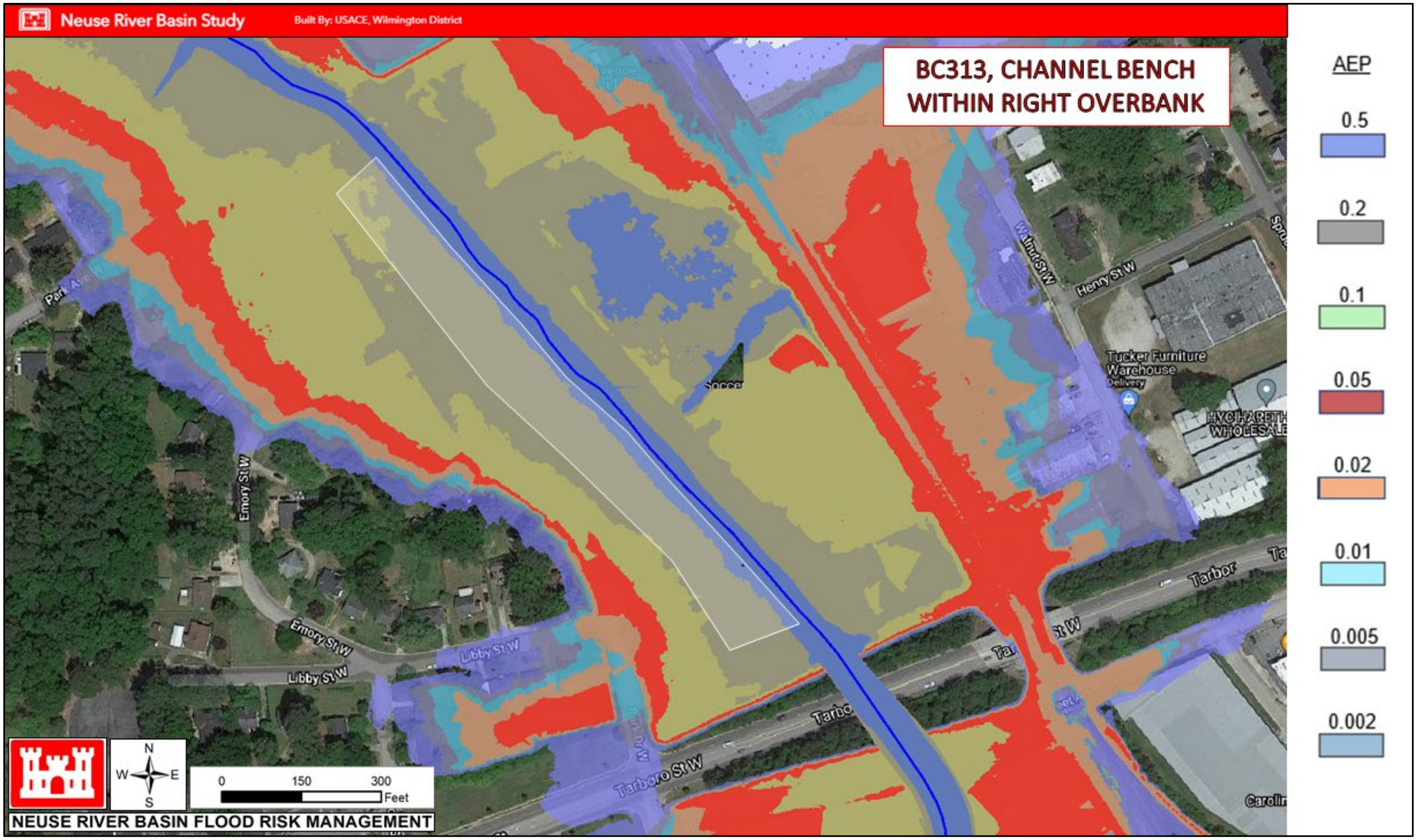

Figure 169. Hominy Swamp Creek Channel Bench BC313 Design Storm Inundation

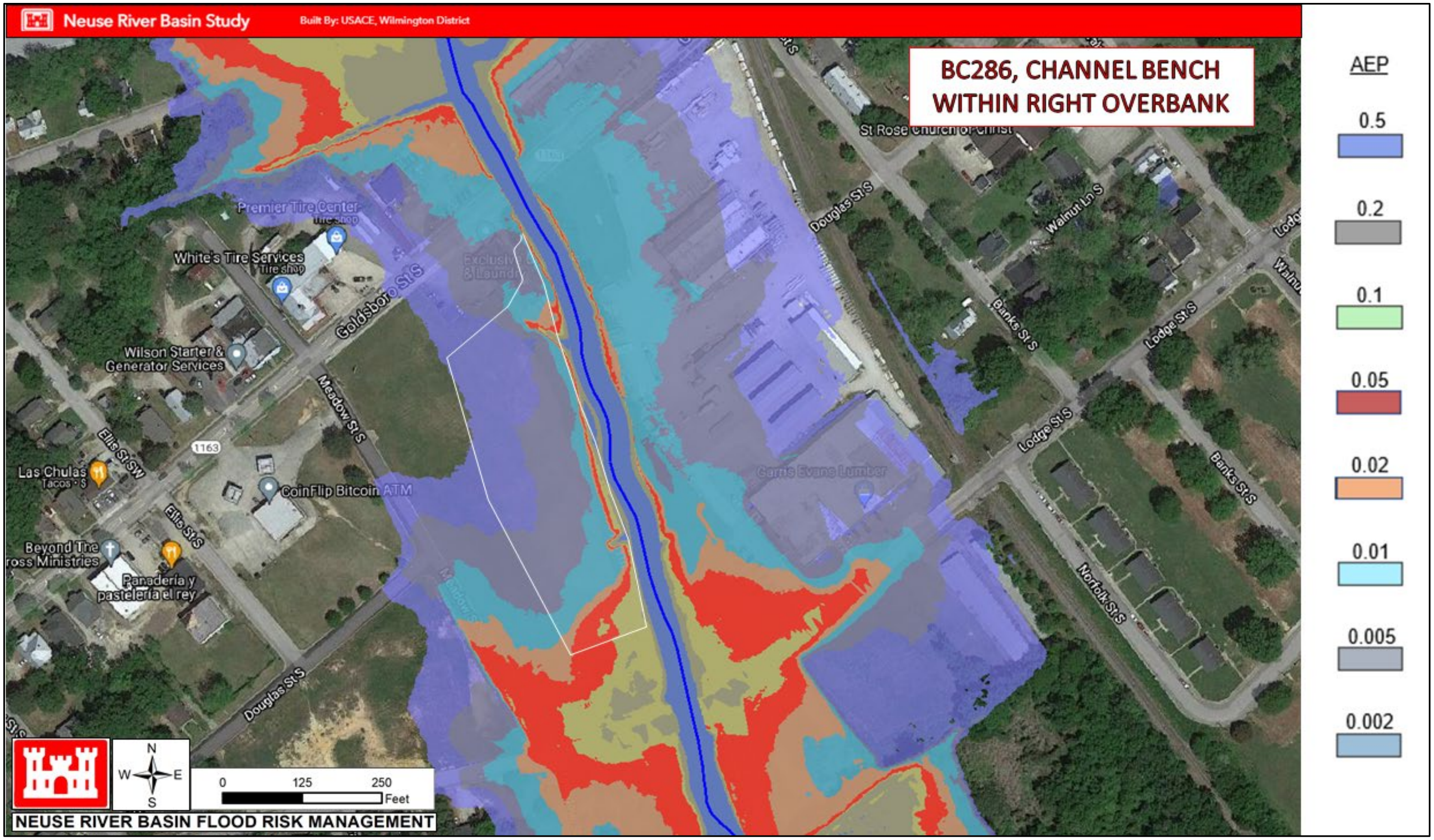

Figure 170. Hominy Swamp Creek Channel Bench BC286 Design Storm Inundation

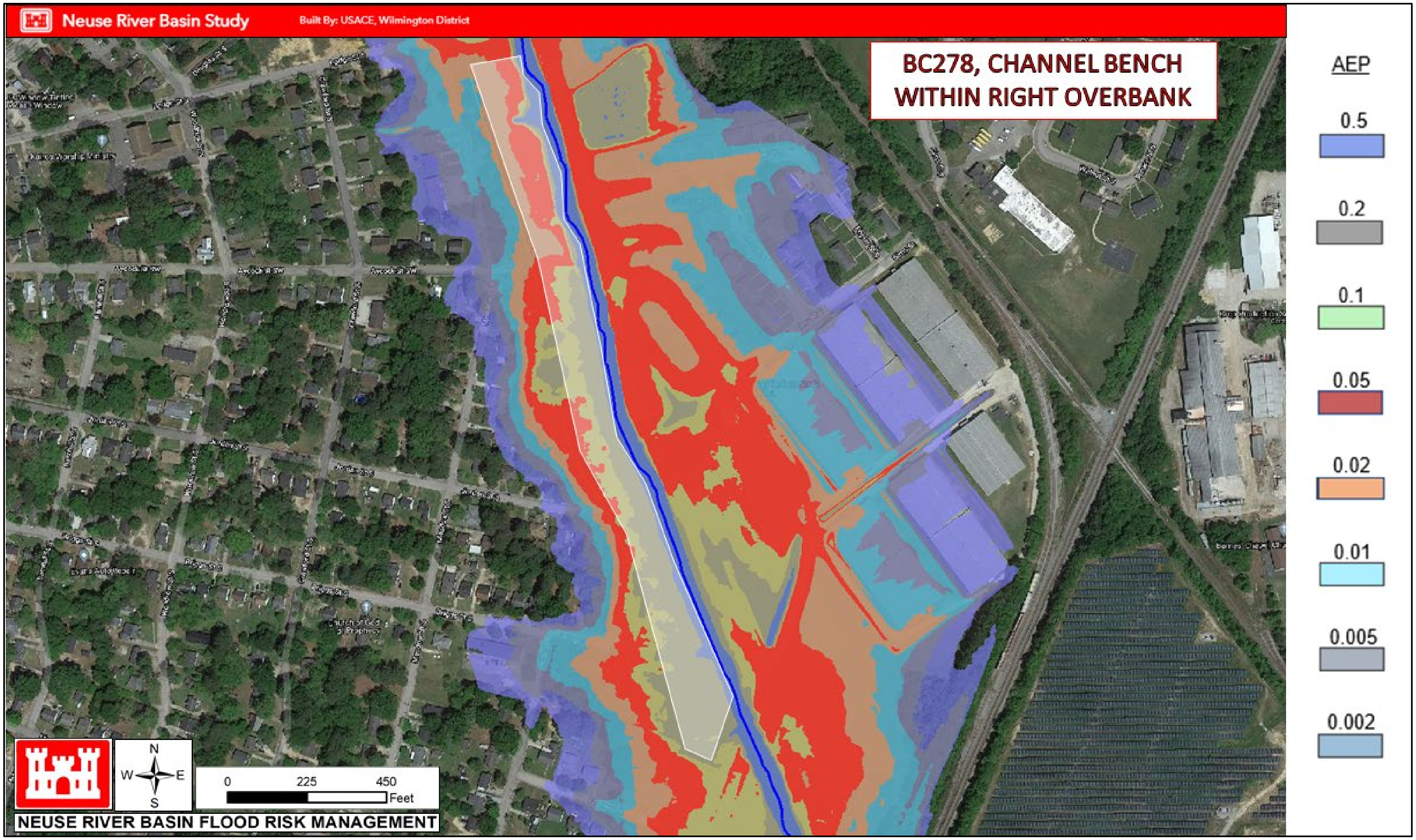

Figure 171. Hominy Swamp Creek Channel Bench BC278 Design Storm Inundation

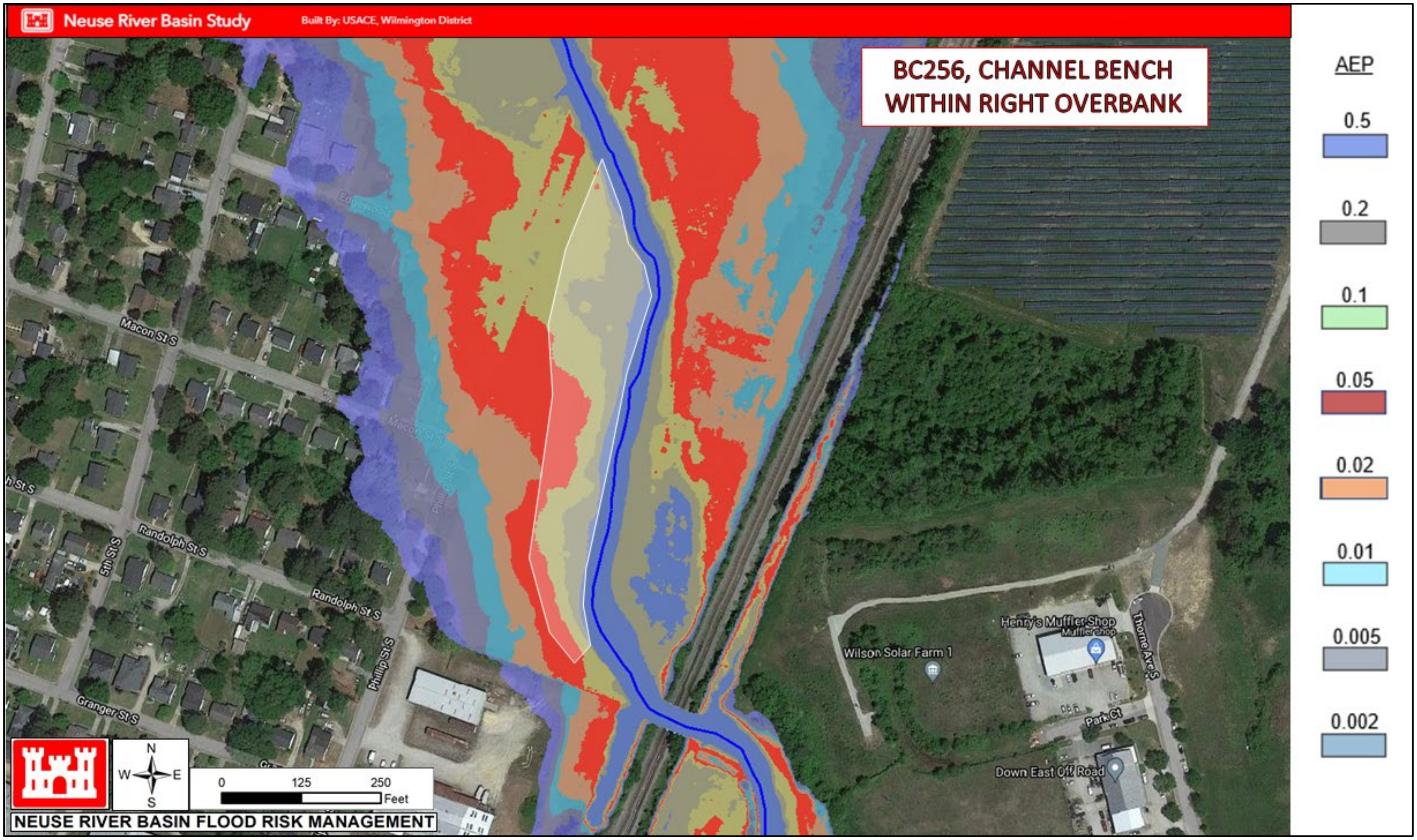

Figure 172. Hominy Swamp Creek Channel Bench BC256 Design Storm Inundation

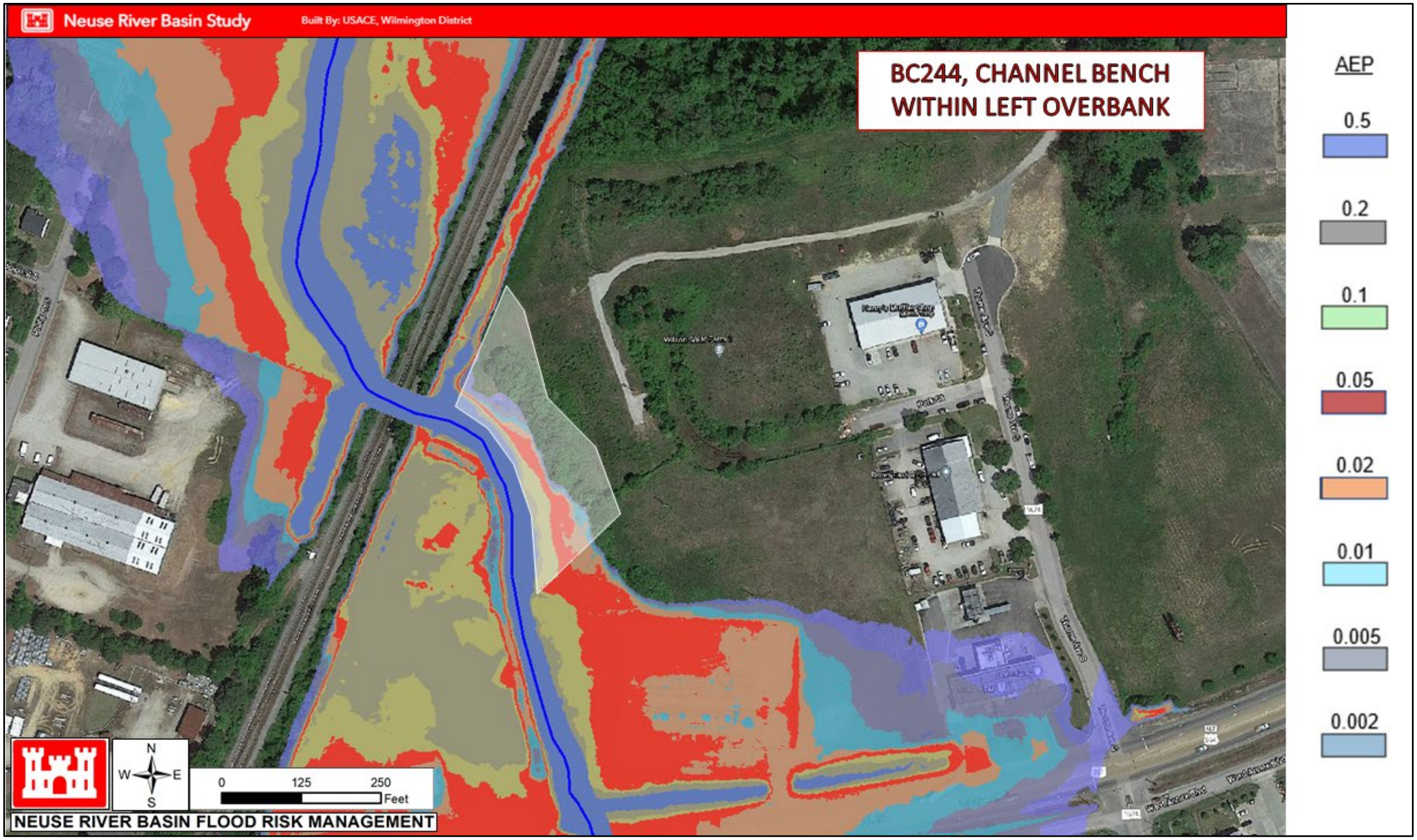

Figure 173. Hominy Swamp Creek Channel Bench BC244 Design Storm Inundation

The design storms inundation footprint appeared to be confined to a floodplain width between 600-ft and 900-ft. The widest portions were immediately upstream of bridge/culvert crossings, which suggested inadequate cross-sectional area of the channel that passed under bridge decks and/or through undersized culverts. The narrow floodplain also helped explain the amount of incision that has historically occurred within the Hominy Swamp Creek channel. Water surface profiles of select design storms for FWOP- and with FWP (channel bench)-conditions are shown in Figure 174.

A review of WSEL reductions under channel modification conditions showed improvements immediately upstream of the NC-42 crossing, at the start of BC402. Improvements continued downstream for approximately 2 miles until the creek reached the Tarboro St crossing. This crossing, which consisted of a relatively large earthen embankment that included a lower elevation, secondary route (Tarboro St Annex), appeared to not allow improvements to efficiently propagate downstream. Roughly one mile further downstream, a similar condition was seen where the creek had trouble conveying flow through the CSX railroad culvert. Regardless of these issues, the channel bench measures were successful at improving FRM by reducing WSEL for the design storms. There was an average WSEL reduction of 0.5-ft for the 0.01-AEP event in the reach between the NC-42 and CSX crossings. Due to the improved conditions with this measure in place, it was carried forward for consideration as either a stand-alone alternative or combination with other viable measures.

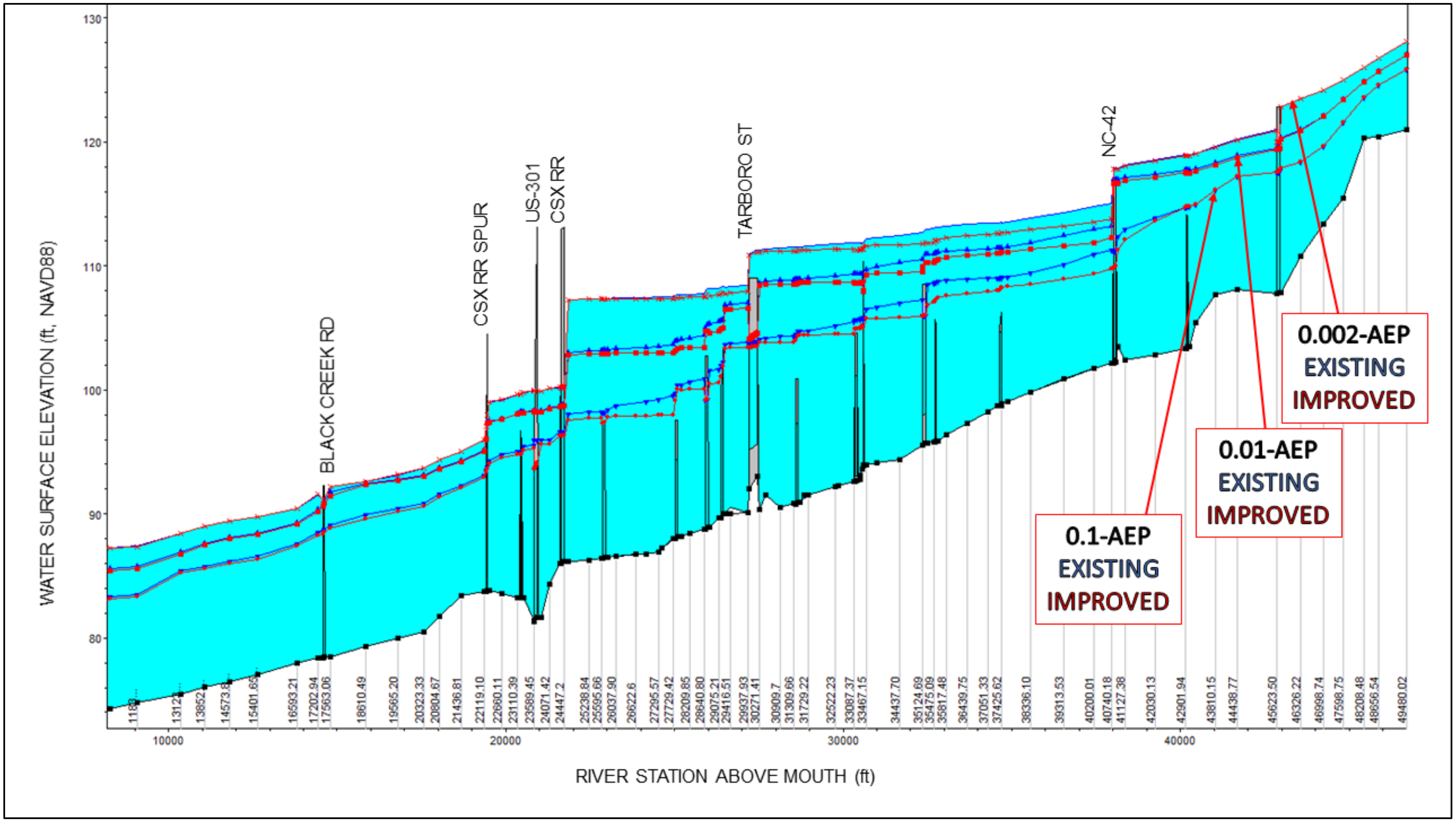

Figure 174. Comparison of Water Surface Profiles for Select Design Storms – FWOP vs. FWP (9 channel benches in place)

7.3.3 Crabtree Creek Channel Modification in Raleigh, NC

Traditional channel modification was represented by applying a widened channel template at existing grade or by excavating to a new design surface for the channel bottom invert. Based on a review of the existing channel grade in the FEMA effective and preliminary hydraulic model, there appeared to be a consistent slope throughout most of the study reach with several exceptions. Due to the high number of creek crossings throughout the study reach, it was impractical to apply a comprehensive template without having a significant impact to existing infrastructure. Furthermore, the highly urbanized Crabtree Creek corridor constrained the magnitude of channel templates that could be applied without negatively impacting nearby structures. Short segments of the Crabtree Creek channel exhibiting inefficient gradients were identified as candidates for an excavated channel template to determine their relative impact to flooding magnitude and inundated footprint. With Crabtree Creek having a well-defined channel bottom, templates widths were based on surrounding cross section geometry so that channel bottom widths were consistent throughout the study area. The Crabtree Creek HEC-RAS model was used to apply these channel templates. A new geometry was created that included the three improved channel segments, and simulations were run for the full range of design storms. Manning's roughness value for the channel was slightly reduced to represent the new channel efficiency. Model results showed there to be a negligible difference in WSEL (<= 0.15-ft) when compared to FWOP conditions across all design storms. Based on these results, channel excavation was screened from further consideration for Crabtree Creek.

Similar to measures developed for the Hominy Swamp Creek study area (Section 7.3.2), channel modification through widening was assessed by including overbank floodplain, rather than just the channel bottom width. The proposed channel modification was located within the left overbank of the Crabtree Creek as it flowed through the City of Raleigh, NC. Preliminary assessment of existing flooding along Crabtree Creek revealed a critical portion of the floodplain that existed between the Anderson Dr and Atlantic Ave creek crossings. In this location, the floodplain width quickly expanded from about 600 feet to over 2,500 feet. The primary feature involved in this measure was excavation of channel benches that functioned as floodplains and created natural alluvial channel processes. The resulting Crabtree Creek primary flow path would consist of a dominant discharge channel (existing bankfull conveyance) and a floodplain bench. The channel-forming discharge channel would provide the necessary sediment conveyance, while the floodplain bench would provide for design flood conveyance. Seven segments of benched channel were positioned along the river's banks with a bottom invert set roughly 2 feet above the water surface elevation expected from an average annual discharge (1.0-AEP). The benched surface included a minor slope away from the river to ensure adequate drainage. The perimeter of the benched surface assumed 3H:1V side slopes to tie back into existing grade. A total channel bench length of almost 1.5 miles extended from the downstream face of Anderson Dr bridge to approximately 2,000 feet downstream of Atlantic Ave. An overview of these measures along Crabtree Creek is shown in Table 78.

Table 78. Channel Modification Details for Crabtree Creek in Raleigh, NC

Channel Bench ID	Channel Overbank Side	Location		Approx. Length (ft)	Footprint Area (sq ft)	Width (ft)
		From	To			
BC469	Left	Anderson Dr	Greenway Br (Dirt Rd 1)	1400	136000	100
BC454a	Left	Greenway Br (Dirt Rd 1)	Big Branch tributary	560	51300	100
BC454b	Left	Big Branch tributary	Wake Forest Rd	900	91100	100
BC436	Left	Wake Forest Rd	Railroad Br (RS41.7)	1900	176100	100
BC416	Left	Railroad Br (RS41.7)	Atlantic Ave	300	33300	100
BC411a	Left	Atlantic Ave	Unnamed tributary (RS40.8)	120	12000	100
BC411b	Left	Unnamed tributary (RS40.8)	Unnamed tributary (RS38.7)	2200	198000	100

As detailed in Table 78, channel bench segments were separated by bridge structures that crossed over the main flow path of Crabtree Creek. Additionally, two segments (BC454 and BC411) were split to allow smaller tributaries to maintain drainage paths to Crabtree Creek. All segments were in the left overbank floodplain, leaving the right bank in its natural state. Due to the highly urbanized corridor adjacent to Crabtree Creek, it was impossible to completely avoid utility and infrastructure impacts. Implementation of this measure would require re-alignment of the existing Crabtree Creek greenway trail over an approximate 1.1-mile length. The conceptual design re-located the trail along the channel bench boundary, on natural high ground. There was a recognized potential to route the trail within the channel bench at the design grade during measure refinement.

The seven segments were modeled within the same HEC-RAS geometry by modifying the terrain over a series of cross sections that represented the segment footprints. Manning's roughness values were reduced within the footprint areas to represent improved conveyance due to change in land cover from woody wetland and deciduous forest to developed open space. Proposed conditions were simulated under the suite of design storms. Profiles for select design storms comparing FWOP and with channel bench designs in place is shown Figure 175.

A review of WSEL reductions under FWP conditions showed improvements immediately upstream of the Lassiter Mill Rd crossing, 1.3 miles upstream of BC469. Improvements continued to be seen downstream for approximately 2.8 miles before WSEL returned to FWOP conditions by the end of the BC411b footprint (2,000 feet downstream of Atlantic Ave.). Conditions were notably improved at the Wake Forest Rd bridge where the FWP 0.1-AEP no longer overtopped the bridge deck (FWOP overtopped this bridge by about 0.5-ft). FWP maximum WSEL reduction was seen near the Anderson Dr crossing at 1.5-ft below the FWOP 0.1-AEP event. At the same location, there was a 1.2-ft WSEL reduction for the 0.002-AEP event. In general, the effectiveness of improvements was reduced as the severity of design storm increased. Due to the improved conditions with this measure in place, it was carried forward for consideration as either a stand-alone alternative or combination with other viable measures.

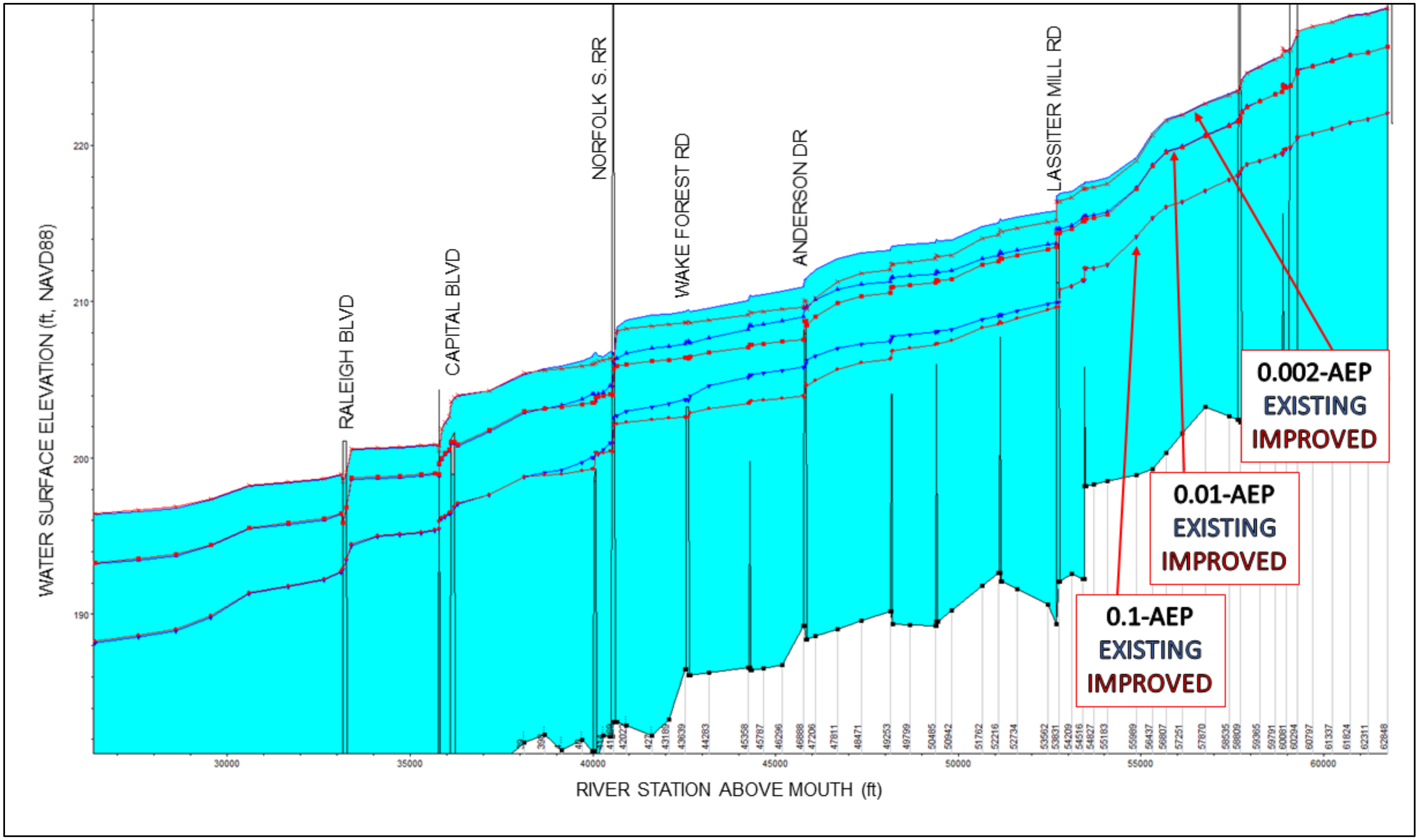

Figure 175. Comparison of Water Surface Profiles for Select Design Storms – FWOP vs. FWP (7 channel benches in place)

7.3.4 New Levees Along Neuse River Mainstem

The measure of new levee alignments was investigated for portions of overbank flooding from the Neuse River mainstem in the vicinities of Smithfield, NC and Goldsboro, NC. These locations were chosen based on the close proximity of existing structures that appeared to be vulnerable to comprehensive flooding from the Neuse River mainstem. The NCFRIS database was utilized to validate structural vulnerability by comparing the Effective and Preliminary, when available, FEMA flood maps with tool output of flood and risk information, and financial vulnerability indexes. Building first floor elevations were compared to water surface elevation rasters to identify cases where building footprints were shown in an inundation boundary, but the habitable space had been elevated above the first floor elevation (FFE). This comparison reduced the chances of overestimating benefits within a leveed area. Furthermore, according to the Water Resources Development Act of 1990 Section 308, new or improved structures built within the 100-year (0.01-AEP) floodplain after July 1, 1991, with first floor elevations lower than the 100-year flood elevation, should be excluded from the structures used to calculate national economic development (NED) benefits for flood damage reduction projects.

Levees were represented as lateral structures in the hydraulic model. Areas behind a levee, also referred to as the leveed area, were modeled as a storage area. In some situations, the leveed area was modeled as a 2-dimensional area. Initial levee crest elevations were based on an overtopping frequency of the 0.002-AEP flood elevation, plus 1-foot to conservatively account for uncertainty, at the upstream extent of the measure locations for expedited screening purposes. Levee crest elevations were gradually sloped from upstream to downstream to reflect the natural sloped water surface of flood event. Screening-level design did not include levee superiority or planned overtopping sections.

7.3.5 New Levee Along Neuse River in Smithfield, NC

A levee alignment in Smithfield, NC was selected to target overbank flooding to a combination of residential and commercial structures, and critical infrastructure in the southwest portion of the city. An earthen levee approximately 2 miles in length was positioned along the left overbank within the FEMA 0.01-AEP flood zone for most of its length, however, a portion was required to encroach into the regulatory Floodway to include the Johnston County Wastewater Treatment Plant. The levee would be elevated to the 0.002-AEP event plus approximately 1 foot to account for hydraulic uncertainty. Overview of the levee alignment is shown in Figure 176.

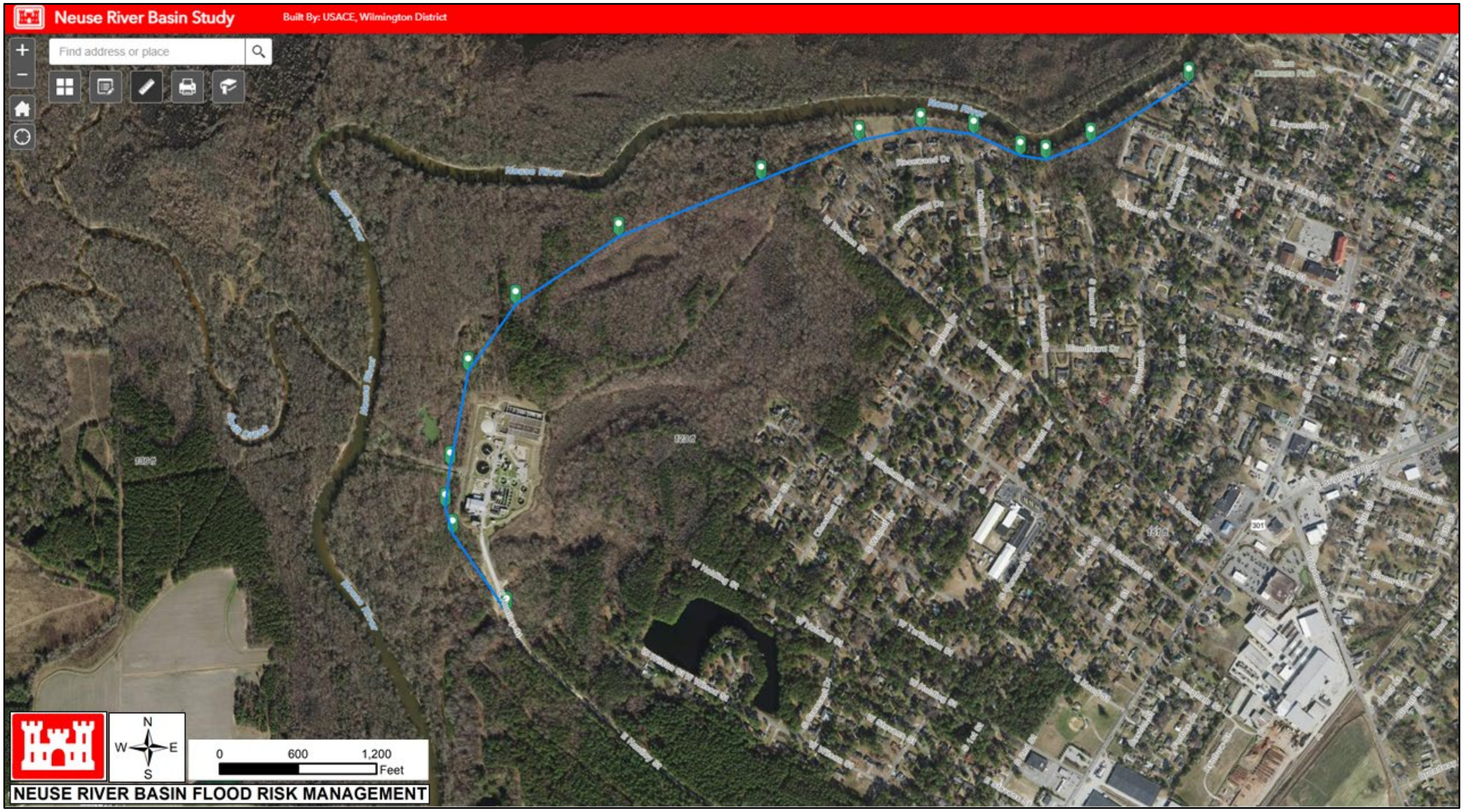

Figure 176. Smithfield Levee Alignment

An initial building count within the leveed area using the NCFRIS building dataset included 6 separate structures related to the WWTP operations and nearly 190 other structures. The majority of these buildings had been elevated above the 0.002-AEP event and were removed from FRM consideration. Furthermore, 5 single family dwellings built after 1991, including 1 mobile home were eliminated from the damage pool. Input from the Johnston County WWTP during development of this measure was used to eliminate its operations from inclusion for federal interest due to their existing levee improvement project. The WWTP’s existing effort, through a FEMA grant, will extend their levee system crest to a roughly 0.002-AEP overtopping frequency. Removal of the WWTP structures from the pool of potential benefits had a significant reduction to overall economic viability, based on building and content value (S_BUILDING_FP dataset, NCFRIS, 2013). The preliminary total number of structures that would be included for determination of federal interest was 32. These buildings were constructed in the mid-1970s on average.

Hydraulic performance of the levee showed for a distance of roughly 7 miles; 4 miles upstream, 2 miles adjacent to its alignment, and 1 mile downstream of the project, there would be an average increase to the water surface elevation during the 0.002-AEP of 0.3 feet. There was concern that a levee near Smithfield may be sensitive to coincidental flooding due to the nearby confluences of Swift Creek and Middle Creek with the Neuse River. While it was not included in this preliminary assessment, it was recommended this concern be validated if the measure were analyzed in more detail.

As stated above, the inability to capture benefits from the Johnston WWTP made this measure more challenging to justify. After including potential mitigation required for the 7-mile length of induced water surface elevation, the overall benefit offered by the levee alignment would be further reduced. Due to the disproportionate cost to benefit offered by this measure, it was screened from further consideration.

7.3.6 New Levee Along Neuse River in Goldsboro, NC

Similar to in Smithfield, several new levee alignments were investigated near Goldsboro, NC. Targeted flooding areas were identified within the left overbank of the Neuse River located along the western portion of the city limits. This floodplain is associated with the 7-mile meander stretch of the river that is bypassed by a federal cutoff channel to the south. Most of the lands within the meander are either undeveloped or used for agriculture, except along main traffic arteries where commercial and residential development has been heavy. This general area has historically been prone to overbank flooding. The nature of flooding is influenced by elevated roadway berms in addition to the low relief of natural terrain, especially along US-117 that serves to bisect the floodplain. As a result, interior drainage and stormwater drainage networks can become stressed during prolonged significant flood events such as tropical storms. The mouth of the Little River, a major tributary to the Neuse River, is located near the northern most point of the mainstem meander. The total Little River basin area is roughly 315 square miles. Its floodplain is about 1.5 miles wide near confluence with the Neuse River and has not been developed extensively for structural purposes because there are few traffic arteries across the floodplain. Most of the development which has occurred is along the roads that cross Littler River floodplain above the FEMA 0.01-AEP WSEL. A notable exception to this is the N.C. State Hospital and Farm (Cherry Hospital) which is located in an area subject to flooding from both Neuse River and Little River. Big Ditch, a highly urbanized, partially channelized smaller tributary, drains into the Neuse River mainstem meander. There is little room left for development within the Big Ditch watershed. The FEMA regulatory floodway for the Neuse River is almost 2 miles in width in this general area and although it has posed significant restrictions to newer development near the river's edge, older structures are still interspersed throughout the floodplain.

A comprehensive line of protection offered by a structural levee had engineering challenges due to the presence of these tributaries and their high potential for backwater effects. This simplified assessment assumed one or more closure structures would be required to maintain adequate interior drainage within the leveed area. It was also acknowledged that a more sophisticated interior drainage system, involving pumping stations, may be required. These assumptions carried sizeable uncertainty as their implementation may not be engineeringly feasible or may result in disproportionate benefit-to-cost ratios.

There were multiple potential routes for a levee system to take along the left overbank of the Neuse River mainstem meander; however, a persistent line of protection was necessary along the southern edge of the targeted flooding area. Further assessment of flooding mechanisms in this area revealed a significant threat of backwater that occurred downstream of the mainstem meander section. The left overbank, beginning immediately downstream of the US-117 and CSX bridges and ending near the Arrington Bridge Rd bridge, would require a line of protection to prevent overbank flooding from entering the intended leveed area from the south. A simplified design to accommodate this line of protection was implemented by elevating Arrington Bridge Rd, beginning at its intersection with US-117, and extending southeast to its intersection with Westbrook Rd. From this point, Westbrook Rd would be elevated northeast to its intersection with S Slocumb St. The total length of elevated road for this southern alignment was about 2.2 miles. The southern alignment is shown in Figure 177.

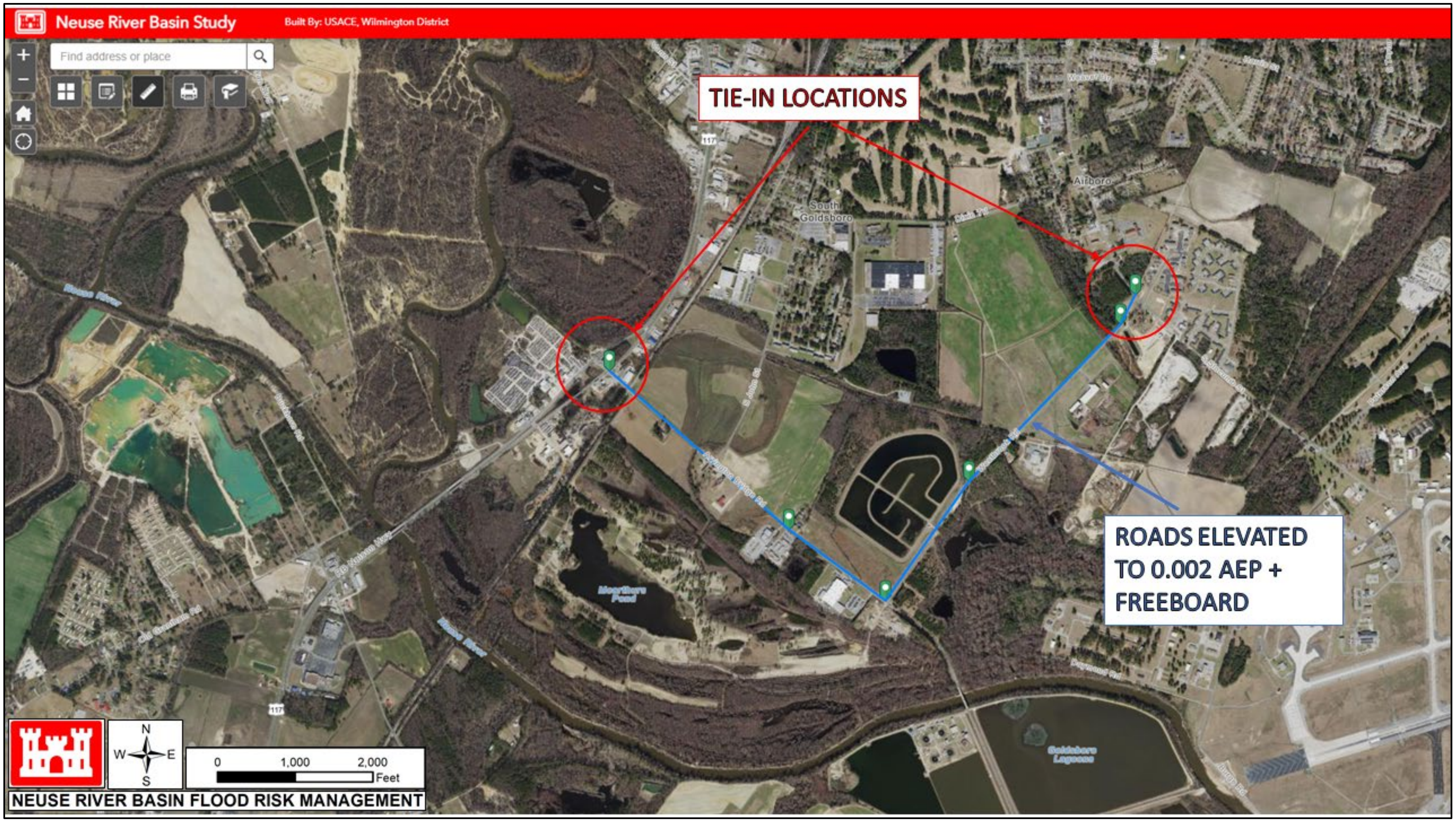

Figure 177. Goldsboro Levee Southern Alignment

It was not practical to provide a leveed area for all identified structures within the floodway and floodplain due to the lack of surrounding natural high ground, both upstream and downstream, that would function as a levee tie-in point. Regardless of which alignment that was assessed, this lack of nearby high ground resulted in existing structures that would still be vulnerable to flooding, even with the levee system in place. Furthermore, the elimination of floodplain storage within the leveed area resulted in a detrimental effect that increased WSEL in the area between the river channel and riverside levee embankment.

A new levee alignment (US-117) involving an extensive road-rise of HWY117 was hydraulically assessed. This roadway improvement would be designed as if it were a stand-alone earthen levee. There is precedence for DOT routes also serving as levees, though it is generally not preferred due to the inherent risk of non-performance or failure involved with a FRM feature that also serves as a major transportation route. Notably, this route was also identified by NCDOT in their 2020 Flood Abatement Assessment as a “resilient route”, where it was desired to improve HWY117 so that it would remain open during extreme events. A figure of resiliency routes for Goldsboro from the 2020 NCDOT report is shown in Figure 178.

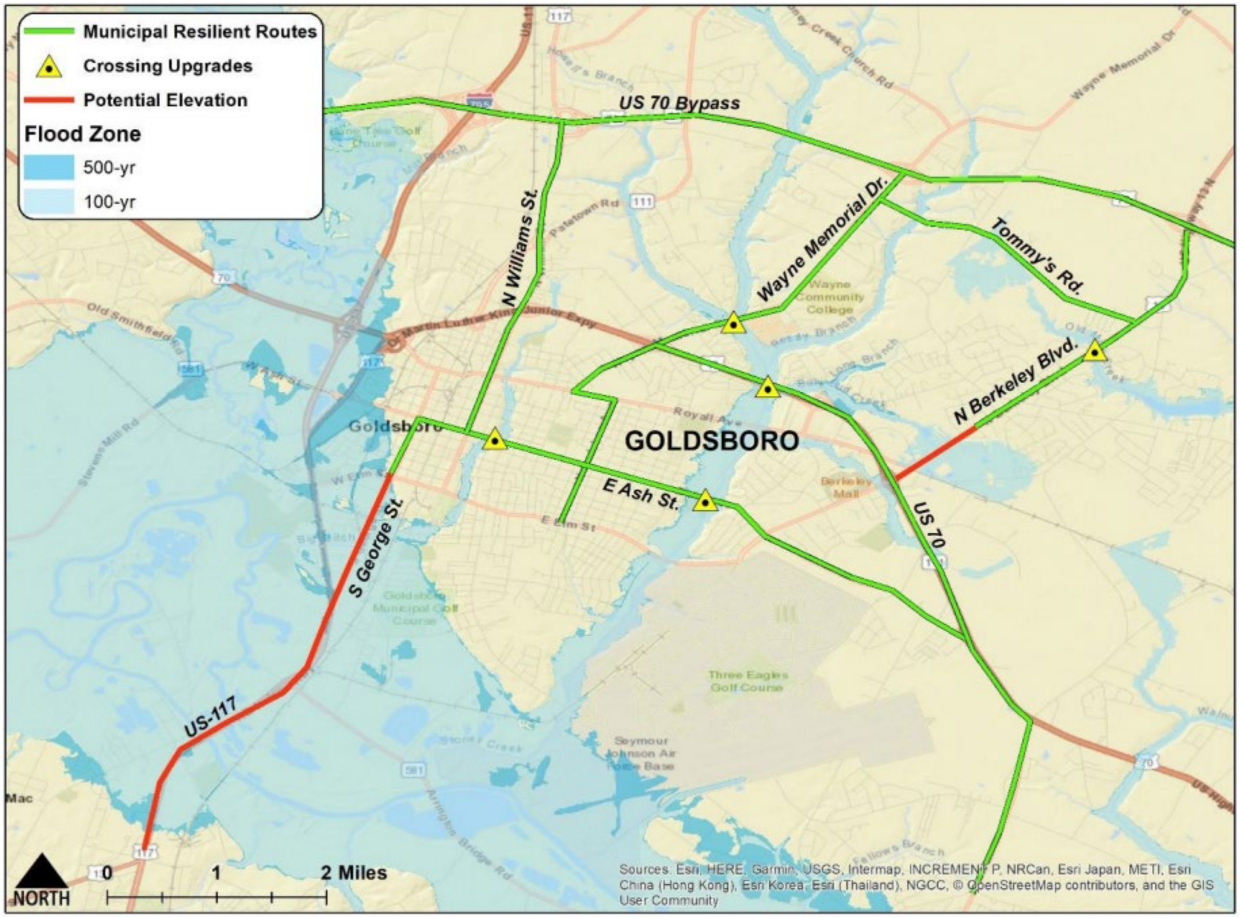

Figure 6-26: Proposed resilient routes for Goldsboro.

Figure 178. US-117 resilient route from 2020 NCDOT Report

The upstream levee terminus tied into the HWY70/US-117 interchange, east of Little River. US-117 was modeled as a 20-foot wide lateral structure elevated to the 0.002-AEP event plus 1 feet to account for hydraulic uncertainty. The lateral structure was placed on top of US-117 and traced its route south to the intersection with South George Street. The total length of elevated road embankment was approximately 4 miles. It then involved a bridge deck raise where it crossed the Neuse River. No additional modifications to the existing bridge structure were made for this alignment. As mentioned earlier, a southern levee alignment, that included portions of Arrington Bridge Rd and Westbrook Rd elevated to the 0.002-AEP plus 1-foot to account for hydraulic uncertainty was considered part of this overall alignment. Overview of this levee alignment is shown in Figure 179.

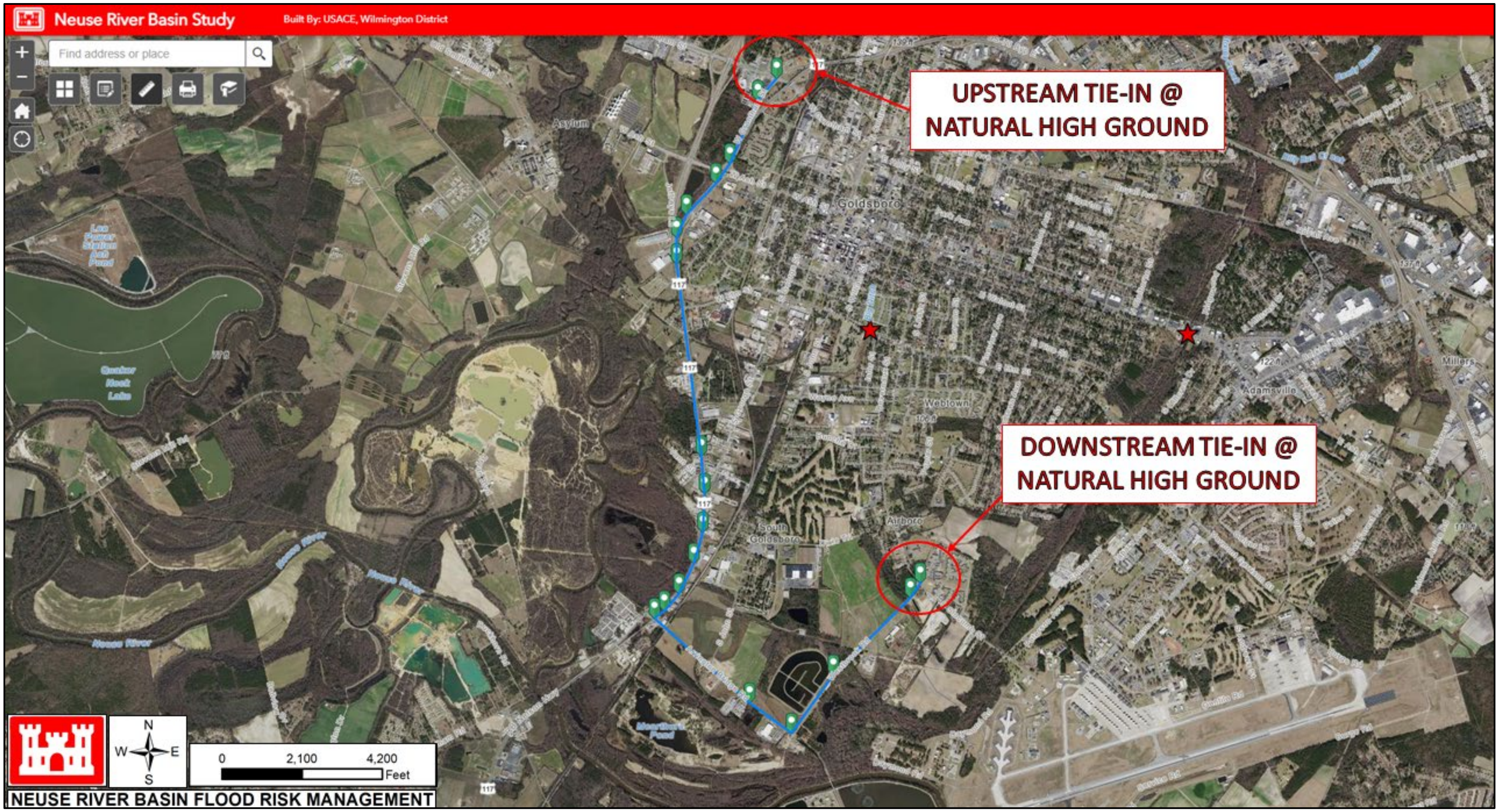

Figure 179. Goldsboro Levee Alignment (US-117)

A review of the leveed area using NCFRIS showed about 500 structures that would be removed from the existing 0.002-AEP floodplain. It was acknowledged that the US-117 route served to bisect several clusters of structures and that a number of those would be left outside of the leveed area. There were over 150 structures that would remain exposed to flooding due to their location near the riverside toe of the US-117 levee embankment.

A secondary alignment that involved an elevated portion of US-117 that transitioned to an overland earthen levee along the eastern edge of the FEMA regulatory floodway was hydraulically assessed. The intent in this alignment was to reduce the number of structures that would have remained within the floodplain between the Neuse River channel and riverside levee embankment. A 1-mile portion of US-117, which began at the HWY70 interchange at the upstream end, was elevated to the 0.002-AEP event plus 1-foot to account for hydraulic uncertainty. The alignment then transitioned off road, running parallel to a NC Railroad, just north of Elmwood Cemetery. The levee crossed over the railroad then took a nearly 90-degreen turn to the south and ran roughly parallel to the FEMA regulatory floodwall for 8,500 feet. Finally, the alignment tied back into an elevated portion US-117, about 0.4 miles south of the Vann Street intersection and ran along US-117 Southbound to the US-117 bridge Neuse River crossing. Like the previous alignment, the bridge deck was elevated to the 0.002-AEP event plus 1-foot to account for hydraulic uncertainty. No additional bridge modifications were made. Like the previous alignment, the southern Arrington Bridge Rd and Westbrook Rd road elevation was included in this proposal. Overview of this levee alignment is shown in Figure 180.

Figure 180. Goldsboro Levee Alignment (US-117/Overland)

A review of the leveed area using NCFRIS showed about 600 structures that would be removed from the existing 0.002-AEP floodplain. Unlike the previous alignment, there were approximately 50 structures in the immediate vicinity of the riverside levee embankment. Although this was not ideal, it was considered an improvement over the previous alignment by allowing the levee to follow the outline of the floodway rather than just be aligned to US-117.

Hydraulic performance of both alignments showed that over an approximate 14-mile length upstream from the Arrington Bridge Rd Neuse River crossing, there would be an average increase to WSEL during the 0.002-AEP of 2.5 feet. There didn't appear to be a significant difference in induced water levels between the two alignments. This 2.5-foot WSEL increase over existing conditions was shown to impact nearly 600 structures within the 14-mile length of the floodplain. From a cost efficiency perspective, these initial assessments revealed a substantial amount of mitigation that would be required to address the induced WSEL. Given the large floodplain footprint in this area, mitigation options were limited to nonstructural measures during this assessment. After weighing the likely potential for considerable mitigation requirements and uncertainty related to engineering assumptions made for interior drainage, a levee alignment in Goldsboro, NC was screened from further consideration.

7.3.7 New Levee Along Crabtree Creek in Raleigh, NC

This measure was not extensively assessed for Crabtree Creek due to several engineering and design implementation constraints. Overall, the highly urbanized Crabtree Creek corridor made it challenging to identify an ideal site for new levee alignments. The consistent presence of residential and commercial development on both sides of the creek banks created a concern for induced damages as a result of levee construction. The leveed area behind the structure effectively eliminates a portion of existing floodplain storage for use during an overbank flooding event.

One identified levee alignment was assessed through a simplified modeling approach. An alignment that traversed the right overbank floodplain between the Anderson Dr and Norfolk Southern railroad crossings. The alignment began at natural high ground off of Oxford Rd, about 500 feet downstream of Anderson Dr and was routed on top of the Crabtree Creek Greenway trail for 1,000 feet where the trail transitioned to the opposite creek bank. The levee continued along the right overbank, eventually being routed on top of Hodges St. It was uncertain that the proposed earthen levee embankment could be placed within the riparian corridor between the creek's top of bank and north side of Hodges St due to limited space. The downstream levee terminus tied into the existing Norfolk Southern railroad embankment. Total levee length was 1.0 miles. Conceptual levee alignment is shown in Figure 181.

Figure 181. Crabtree Creek Conceptual Levee Alignment

The levee was represented as a lateral structure in the Crabtree Creek HEC-RAS study model. The leveed area behind the levee was constructed as a storage area. The levee crest elevations were based on exceeding the 0.002-AEP flood event at the upstream extent of the measure location for screening purposes. Levee elevations were reduced from upstream to downstream to mimic the general slope of the water surface elevation. A new model geometry reflecting the above design approach was simulated for the full range of design storms. As anticipated, results showed that for the more frequent design storms that were confined by the proposed levee, a measurable WSEL increase above FWOP conditions occurred. During the 0.04- and 0.01-AEP events, a maximum WSEL increase of 0.8-ft and 1.0-ft was seen just downstream of the Wake Forest Rd crossing, respectively. This induced WSEL began above the measure footprint, near the Yadkin Rd crossing, and persisted downstream of the measure footprint through to the mouth of Crabtree Creek.

Based on the overall lack of effectiveness at improving FRM in the study area, this measure was screened from further consideration.

7.3.8 New Levee Along Hominy Swamp Creek in Wilson, NC

The numerous road crossings over Hominy Swamp Creek and the vulnerable structures dispersed throughout the floodplain area made it challenging to identify an ideal site for a levee feature. Available high ground sufficient to tie into a levee alignment was also limited. While road embankments at various crossings were elevated somewhat above the adjacent floodplain, they were often not to the height required to tie in a levee alignment designed for a severe flood event. Significant road raises were not considered for this measure implementation due to the general disproportionate benefit-to-cost characteristics of this study area. Furthermore, the narrow width of the overall floodplain meant induced damages related a levee system were a significant concern.

One identified levee alignment was assessed through a simplified modeling approach. An alignment that traversed the left overbank floodplain between the Lodge St and Phillips St crossings was chosen. This alignment was roughly 2,500 feet in length. Its downstream terminus would tie into the existing earthen embankment of the CSX railroad line. Its upstream terminus would tie into the natural high ground situated between Lodge St and Norfolk St S. The levee crest was placed on top of the existing Norris Blvd thoroughfare. It was assumed for this evaluation that the existing road would be removed to accommodate the earthen levee embankment. Conceptual levee alignment at this site is shown in Figure 182.

Figure 182. Hominy Swamp Creek Conceptual Levee Alignment

The levee was represented as a lateral structure in the Hominy Swamp Creek HEC-RAS study model. The leveed area behind the levee was constructed as a storage area. The levee crest elevations were based on exceeding the 0.002-AEP flood event at the upstream extent of the measure location for screening purposes. Levee elevations were reduced from upstream to downstream to mimic the general slope of the water surface elevation. A new model geometry reflecting this above design approach was simulated for the full range of design storms. Simulation results showed an apparent induced WSEL seen immediately at the levee site as well as upstream and downstream for multiple miles. At several bridge locations, FWP conditions were shown as overtopping bridge decks that were previously not overtopped for FWOP conditions. Due to the overall lack of effectiveness of this measure, it was screened from further consideration.

7.3.9 Crabtree Creek Bridge Modification in Raleigh, NC

This measure involved physical modification of bridge structures and/or their associated embankments. Based on a review of FWOP flood profiles, there were several bridge structures with significantly long embankments that made up their approaches. Considerations were given to structure purpose (i.e., pedestrian, vehicular, train), expected traffic volume, associated route-approach characteristics, and adjacent infrastructure. The effects of bridge modifications were analyzed with profile plots, inundation extents, spatial observation of flood elevation changes.

Two creek crossings with significantly long embankments that bisected the floodplain were identified as primary candidates for modification. The first site was the Norfolk Southern railroad bridge that crossed Crabtree Creek and Hodges St. It was located about 500 feet upstream from the Atlantic Ave bridge. The second site was Raleigh Blvd, roughly 1.5 miles downstream of the railroad bridge.

The Norfolk Southern railroad site's impact to flooding was not solely related to the structure itself but also impacted by the ~3,000-foot-long earthen embankment that led up to the crossing. Since it is a railroad bridge, the vertical alignment made it necessary to have such a long approach. The embankment is over 30 feet above adjacent floodplain at some places. An exercise was conducted to completely remove the bridge structure and associated ineffective flow areas from the geometry. Results showed a WSEL reduction of 1.5-ft that began immediately upstream of railroad's previous location and persisted upstream for about 1,000 feet before quickly returning to FWOP conditions. Effects of the bridge removal were most evident for the 0.005-AEP event. Notably, after removal, there was a 0.2-ft WSEL increase above FWOP conditions that remained through the downstream end of the HEC-RAS model, or about 7.5 miles. It was determined impractical to modify the earthen embankment and to instead focus on improving conveyance through the existing bridge opening. The existing bridge structure was such that relocation of piers would require complete bridge replacement; therefore, piers would remain in place. A simplified concrete flume design, running under the railroad bridge deck, was investigated that would accommodate the existing in-channel pier placement. The proposed rectangular concrete flume channel had a channel bottom width of 80 feet, length of 180 feet and channel wall height of 14 feet. There would be a vertical drop of 1 foot across the total flume length. Manning's roughness values within the channel were reduced to 0.015 to represent the concrete lining. There would be a transitional zone of either riprap or turf reinforcement matting that tied the concrete wall to the natural channel banks. A conceptual cross section of the flume design is shown in Figure 183.

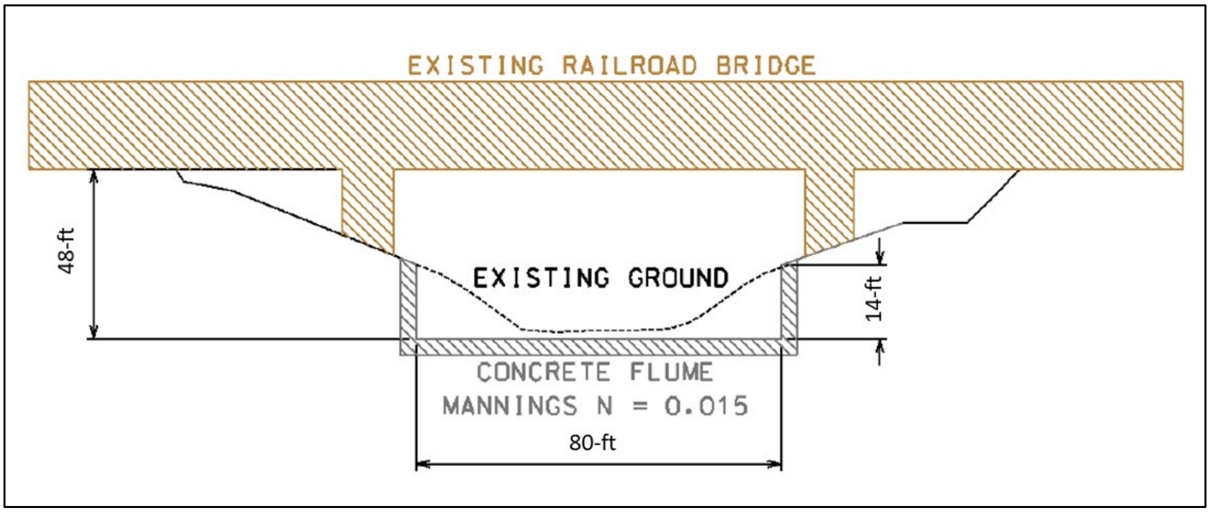

Figure 183. Concrete Flume Conceptual Design

The flume design was analyzed using the Crabtree Creek HEC-RAS model. Bridge and channel geometries were modified to reflect the concrete channel dimensions and improved conveyance efficiency. This FWP condition was simulated for the full range of design storms. Model results showed that for the 0.01-, 0.005-, and 0.002-AEP events there was a consistent maximum WSEL reduction of about 1.0-ft witnessed immediately upstream of the bridge. The effectiveness of this improved WSEL was reduced to 0.5-ft roughly 3,000 feet upstream of the railroad bridge. Over this 3,000-ft segment, average WSEL reduction was 0.6-ft. After modification, there was on average a 0.1-ft WSEL increase above FWOP conditions downstream of the railroad bridge, throughout the remaining modeled reach.

The Raleigh Blvd site was similar to the railroad site in that its crossing was associated with an earthen embankment about 2,700 feet in length that spanned the entire floodplain width. In some places the embankment extended at least 10 feet above the adjacent floodplain. The FWOP 0.002-AEP event was not able to overtop the embankment, so all overbank flow was eventually forced through the bridge span. The bridge and its ineffective flow areas were removed from the geometry to determine its potential backwater effect. Without bridge conditions resulted in a WSEL reduction of

1.8-ft for the 0.005-AEP event. This reduction was largely confined to the segment of Crabtree Creek between the Capital Blvd and the current Raleigh Blvd crossings. The model showed a 0.1-ft WSEL increase above FWOP conditions that remained through the downstream end of the HEC-RAS model. Due to the bridge's superelevation design, pier modification was not considered practical. Instead, the left overbank floodplain and road embankment were investigated for possible supplemental flow area through a triple box culvert design. Three 12-ft by 12-ft box culverts were placed approximately 200 feet to the left of the Raleigh Blvd bridge span. The culvert inverts were set roughly 10 feet above the Crabtree Creek channel bottom invert that passed through the bridge opening. The culverts would be activated for flows above the 0.5-AEP event. A conceptual cross section of the supplemental culvert design is shown in Figure 184.

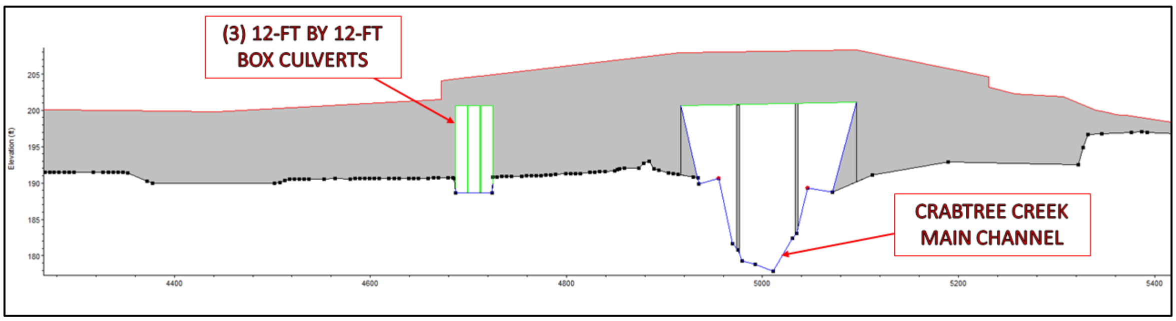

Figure 184. Raleigh Blvd Supplemental Culvert Design

The supplemental culvert design was analyzed using the Crabtree Creek HEC-RAS model. Similar methods to the railroad analysis were used to simulate this measure for the full range of design storms. Model results showed that for the 0.01-, 0.005-, and 0.002-AEP events there to be a WSEL reduction immediately upstream of Raleigh Blvd of 1.5-ft, 0.8-ft, and 0.4-ft, respectively. The averaged WSEL reduction seen within the creek segment between the Norfolk Southern railroad and Raleigh Blvd bridges was 0.3-ft. There was a maximum WSEL increase above FWOP conditions of 0.2-ft.

Based on modeling results of the two measures, the bridge modifications were successful at improving FRM for segments of Crabtree Creek immediately upstream of the assessed sites. However, implementing both measures did show an increase in WSEL above FWOP conditions, potentially creating induced damages. This induced WSEL occurred downstream of the improved conveyance. This scenario was reasonable given the dam-like effect associated with the existing structure's large embankments that spanned the full floodplain width. The two site improvements detailed above were carried forward for alternative plan formulation.

7.3.10 Hominy Swamp Creek Bridge Modification in Wilson, NC

This measure involved physical modification of bridge/culvert crossings over the Hominy Swamp Creek channel. Review of FWOP condition flood profiles helped select sites for evaluation within the study hydraulic model.

An initial exercise of completely removing bridges and ineffective areas from the model provided the magnitude of a structure's impact to overbank flooding. If the creek crossing was associated with train transportation; however, it was not removed from the model. Select crossings removed from the model included: NC-42, Raleigh Rd, Tarboro St, and Ward Blvd. While NC-42 removal resulted in a decreased WSEL of 1.5-ft immediately upstream of its crossing for the 0.01-AEP event, it resulted in a 0.5-ft increase above FWOP conditions for about 3.1 miles downstream. Furthermore, the upstream improvements were partially reduced due the watershed area above this site not meeting ER 1165-2-21 requirements. Raleigh Rd removal resulted in a decreased WSEL of 0.25-ft immediately upstream of the crossing for the 0.01-AEP event. However, few structures were impacted by flooding during FWOP conditions for this upstream segment. Tarboro St removal resulted in a WSEL reduction of 1.0-ft upstream for about 0.75 miles for the 0.01-AEP event. Due to the significant embankment size associated with this crossing, there was a 0.5-ft WSEL increase above FWOP conditions for about 1.5 miles downstream. Ward Blvd removal resulted in a negligible difference in WSEL both upstream and downstream. Due to the lack of effectiveness and disproportionate benefit-to-cost assumptions that resulted from these structures removal, they were not considered for modification.

The CSX railroad crossing over the Hominy Swamp Creek channel was selected for modification. While this crossing was not assessed by complete removal due to it being associated with train transportation, there appeared to be a significant backwater effect occurring immediately upstream of its location. This crossing consisted of an approximate 20-ft span, 14-ft rise, ellipse concrete culvert, based on the effective FEMA hydraulic model. The associated design chart is #29-Horizontal Ellipse, concrete construction with a Scale design #1-square edge with headwall. The culvert length was 67 feet. The railroad top surface elevation was approximately 113.5 feet, NAVD88, with a top width of 28 feet. There was about 13 feet of vertical fill placed between the culvert top and railroad top surface. The railroad upstream and downstream embankment side slopes were on average 1.5H:1V. Aerial imagery of this crossing is shown in Figure 185.

Figure 185. CSX railroad over Hominy Swamp Creek Aerial Imagery

The crossing's backwater flooding impacts were not solely based on the culvert opening but also by the extensive earthen embankment that spanned the full floodplain width. The embankment was oriented in an oblique angle to the main flow path and was about 3,500 feet long. It was impractical to modify this embankment without severely impacting the required vertical alignment of the railroad route. Therefore, modifications were focused on providing additional cross-sectional area of flow passing through the culvert.

The modification consisted of replacing the existing ellipse culvert with a triple box culvert design. The design consisted of three 11-ft span by 8-ft rise concrete boxes, each box separated by a 1-ft wide concrete divider. The upstream and downstream invert elevations were left unchanged from existing conditions. Likewise, the proposed box culvert length was unaltered. There would be about 18 feet of vertical fill required between the top of the box culvert headwall and the railroad top surface.

The Hominy Swamp Creek HEC-RAS study model was used to analyze the CSX culvert modification. The proposed culvert dimensions described in the preceding paragraph were incorporated into a new model geometry. This FWP condition was simulated for the full range of design storms. Results showed WSEL reductions across the full range of design storms. For the 0.1-, 0.01-, and .002-AEP events, there was a maximum WSEL decrease of 0.7-, 1.2-, and 1.0-ft, respectively. This reduction was seen immediately upstream of the CSX crossing and improved conditions continued upstream for about 1 mile to the Tarboro St crossing. An increased WSEL of 0.1-ft to 0.4-ft above FWOP conditions was seen downstream of the CSX crossing. Design storm profiles for FWOP and implemented measure conditions is shown in Figure 186.

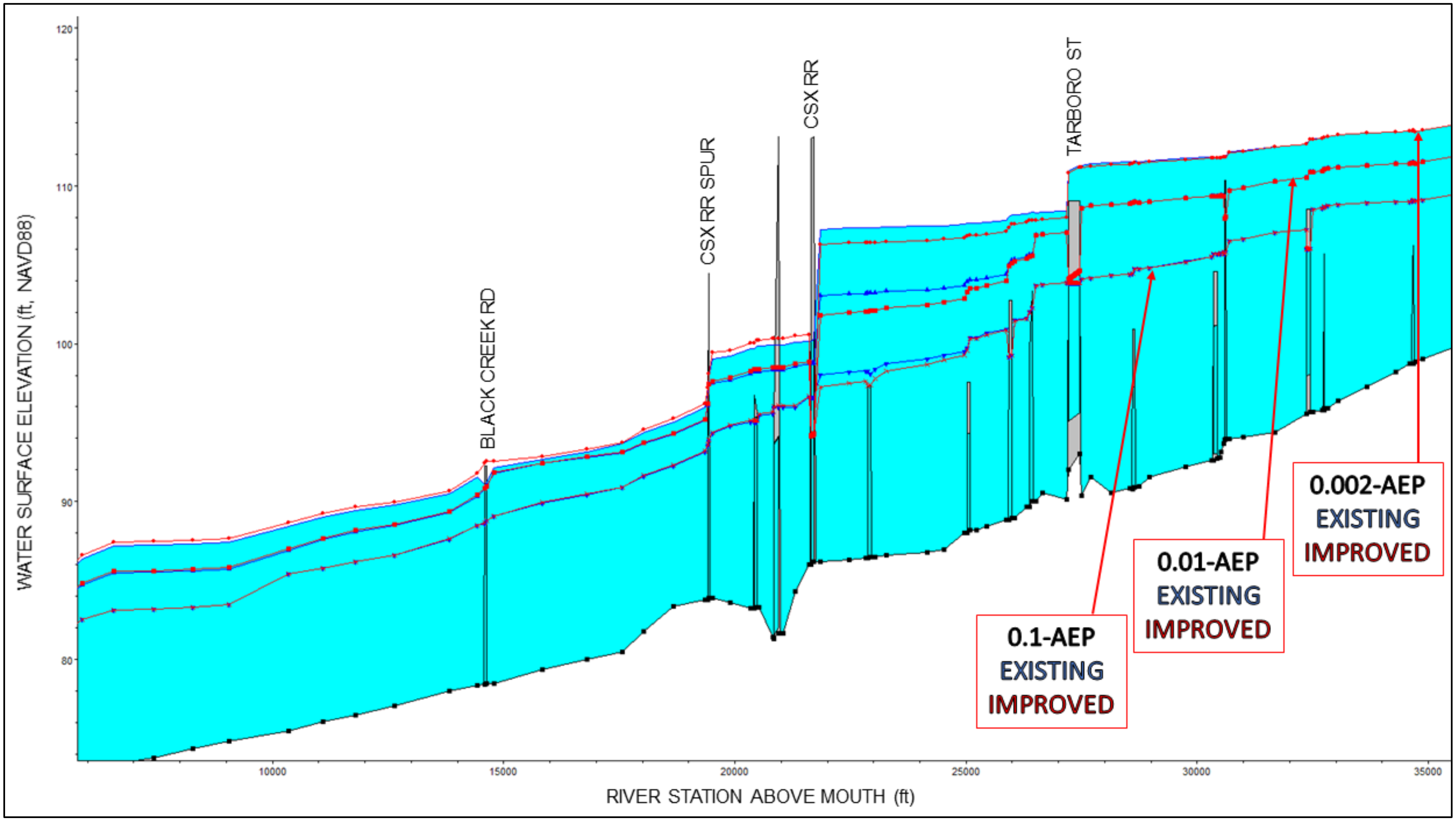

Figure 186. Select Profiles for Culvert Modification at CSX crossing in Hominy Swamp Creek

In order to mitigate for the increased WSEL downstream, 3 additional features were conditional to implementation of this measure. Three creek crossings downstream of the CSX culvert, The Ralston St culvert, CSX railroad spur bridge, and Black Creek Rd bridge were identified for conveyance improvements, as shown in Figure 187.

Improvements were modeled as reduced channel roughness values that would result from removal of sandbar and debris accumulation at the immediate upstream and downstream cross sections of the 3 locations. A site visit supplemented the FWOP conditions that were included in the original FEMA model to validate their current deteriorated state. The CSX Spur crossing had a significant amount of debris accumulated at its piers within the channel. Irregularities in the bridge channel geometry were reduced to represent its FWP shape. The reduced channel roughness values at the three sites and improved conveyance at the CSX Spur bridge were included in the same geometry as the CSX culvert modification. Results showed a negligible increase (< 0.1-ft) in WSEL above FWOP conditions for the 0.002-AEP event, immediately downstream of the CSX railroad culvert and through the Black Creek Rd crossing. However, further downstream at the US-264 crossing, there was still an increased WSEL of 0.1-ft to 0.2-ft above FWOP conditions for the 0.002-AEP event.

Reviewing modeling results showed this measure to be successful at improving FRM for identified problem areas within the Hominy Swamp Creek floodplain. This measure did increase WSEL downstream, though only slightly and in areas of land cover designated as undeveloped and woody wetlands. This condition did reduce its possibility as a standalone alternative; however, it remained a good candidate for being part of a larger array of measures within an alternative plan. Due to this possibility, it was carried forward into alternative plan formulation.

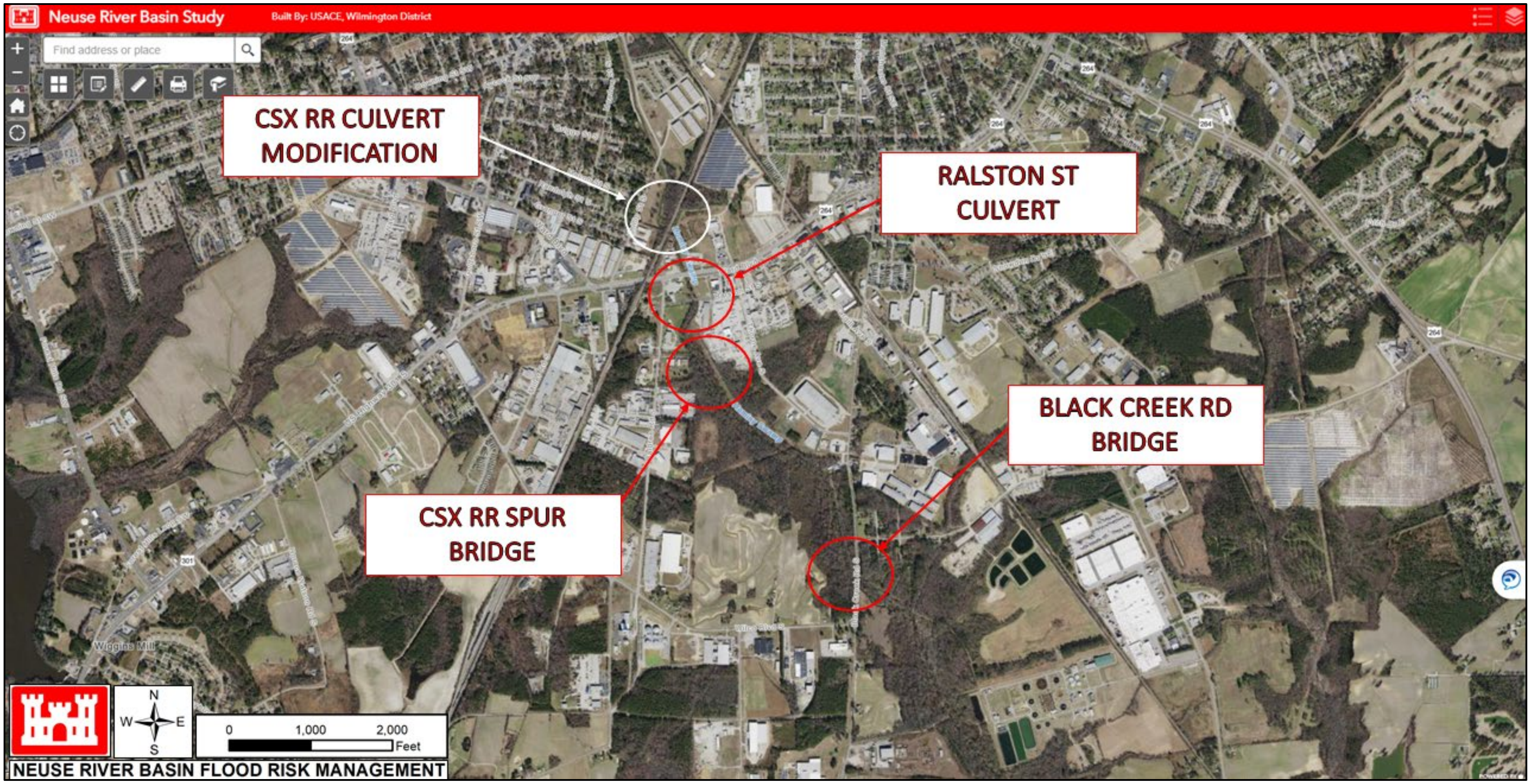

Figure 187. Stream Crossing Improvements Associated with CSX RR Culvert Modification

7.3.11 Hominy Swamp Creek Overbank Detention in Wilson, NC

The pursual of this measure was assisted by documentation from the City of Wilson (Hominy Creek Greenway and Water Quality Park Master Plan, 2016). The city's master plan For Hominy Swamp Creek included conceptual overbank detention sites within the study area. While their functional intent was focused more on providing improvements to water quality, environmental conditions, and aesthetics, the sites did allow for secondary flood risk management enhancements. In general, there were some challenges with locating ideal overbank detention sites in this study area due to the floodplain's narrow shape.

One suggested site from the master plan was located upstream of Park Ave., within the left overbank floodplain. This site was chosen for viability as a standalone measure in this feasibility study. The location of this site is shown in Figure 188.

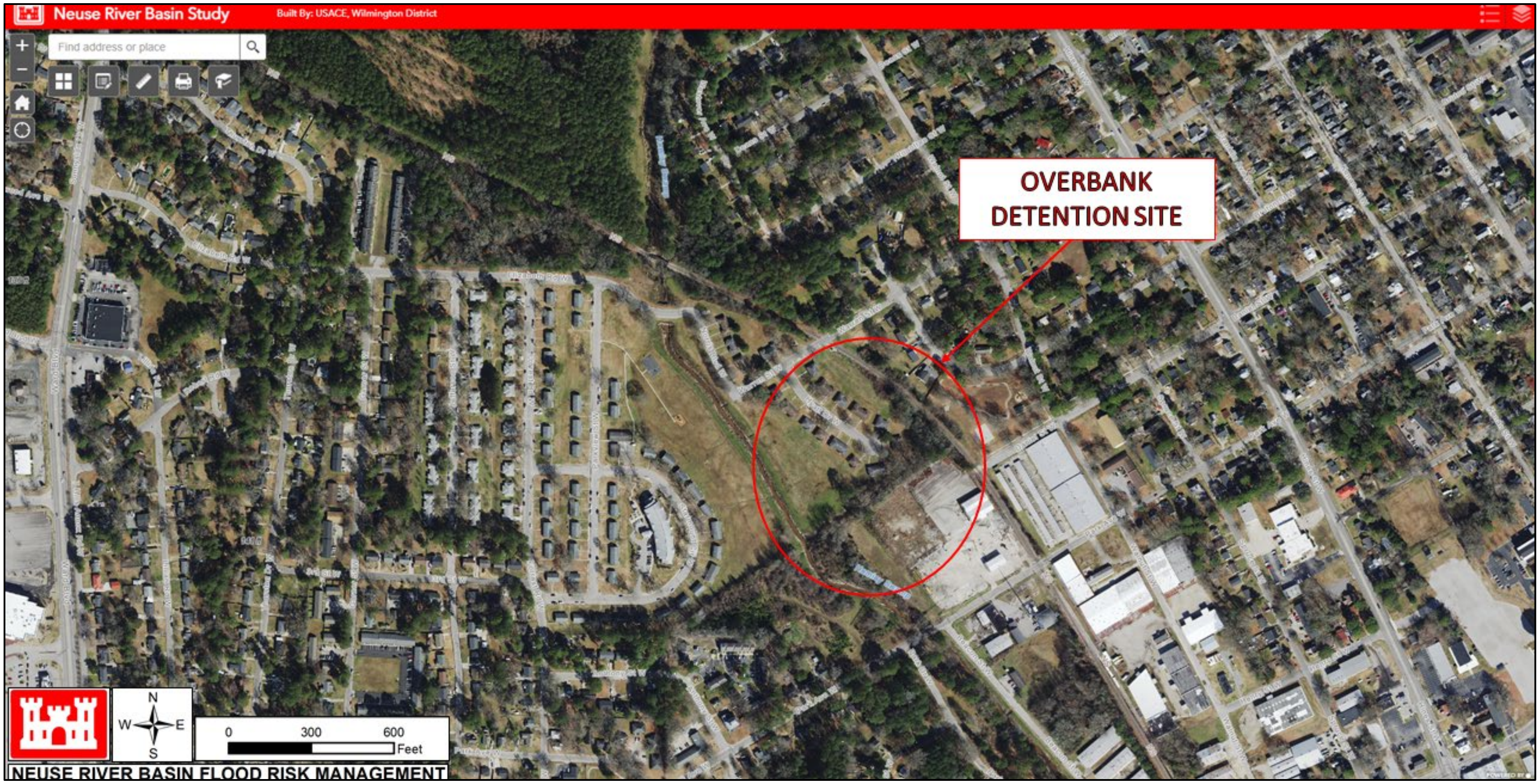

Figure 188. Hominy Swamp Creek Overbank Detention Site

The site would require real estate actions to remove a number of residential structures and commercial parcels from the floodplain. The remaining residential and commercial parcels would require excavation in order to reach design invert of the detention pond, requiring roughly 15 feet of vertical cut in some areas. The total surface area of the proposed site, slightly reduced from the master plan, was about 12 acres; total dry volume was conservatively estimated at 110 ac-ft. Due to site’s footprint on top of an existing unnamed tributary that drained to Hominy Swamp Creek, it was likely a wet detention scenario would be more successful than dry. Therefore, a portion of the total volume was considered inactive storage during flood events. Through an iterative design process, the crest elevation of its constructed berm was set to the 0.01-AEP event. The inline weir, 50-ft long, had its crest elevation set to the 0.04-AEP event.

The Hominy Swamp Creek HEC-RAS model was used to assess the detention site using a simplified method appropriate for this evaluation level. Overbank detention sites were modeled as storage areas that received overtopping flow via lateral structures. The conceptual design template assumed a constructed berm with an inflow weir would be activated to fill the storage area. The storage area elevation per acre-foot volume curve was developed by projecting the pond’s surface area as determined by site-specific characteristics, from a design invert elevation up to the top of control berm. A 30% reduction in capacity was applied to account for side slopes and site grading. To account for the inactive storage as part of the wet detention design, an initial elevation was set within the storage area flow data. The FWP geometry was simulated for the full range of design storms. Results showed that for the 0.01-AEP event, a maximum WSEL reduction of 0.2-ft was seen just upstream of the Tarboro St culvert. For design storms more frequent than 0.04-AEP, when flows were unable to activate the inline weir, there was an overall WSEL increase of >=0.1-ft above FWOP conditions, seen both upstream and downstream. Due to the relative minor impact to FWOP conditions and its decreased and potentially adverse efficiency for the more frequent design storms, this measure was screened from further consideration. Additionally, there were also underlying engineering considerations related to the site’s ability to meet requirements as a federally authorized levee.

7.3.12 Crabtree Creek Overbank Detention in Raleigh, NC

There was limited applicability of this measure to the Crabtree Creek corridor due to the extensive footprint of existing development near the creek channel. In most cases, the trade-off of sizing this measure upstream of areas of significant flooding resulted in disproportionate cost-to-benefit due to assumed real estate impacts. Furthermore, it was assumed removal of structures that were within a proposed overbank detention site would directly hurt realized benefits in the immediate area.

One site was identified along Crabtree Creek for overbank detention assessment. This location is shown in Figure 189. The site was located within the left overbank floodplain, immediately downstream of the Atlantic Ave. crossing. It appeared to be a good candidate site due to the presence of woody wetlands and lack of development. A cursory review of aerial imagery showed the site to regularly have standing water. An approximate 10-acre pond was proposed at this location. Its detention volume was conservatively assumed to have a design invert set to the adjacent Crabtree Creek channel invert. The crest elevation of its constructed berm was set to the 0.01-AEP event with the inline weir crest set to the 0.04-AEP event. This crest design was based on the overall lack of damageable structures for AEP events more frequent than the 0.01-AEP event. It was assumed that the berm would also serve as the Crabtree Creek Greenway trail as there was limited distance between the banks of the creek and detention site. A rough volume was estimated for pond WSEL at berm crest to be 90 ac-ft.

The Crabtree Creek HEC-RAS model was used to assess the detention site using the same simplified method used for the Hominy Swamp Creek study area. The FWP geometry was simulated for the full range of design storms. Results showed that for the 0.01-AEP event, a roughly equivalent decrease downstream and increase upstream in WSEL was seen at 0.1- to 0.2-ft. This change in WSEL was determined to be negligible in reducing flood impacts downstream while potentially requiring mitigation upstream. Due to the overall lack of measure effectiveness and potentially disproportionate benefit-to-cost, this measure was screened from further consideration.

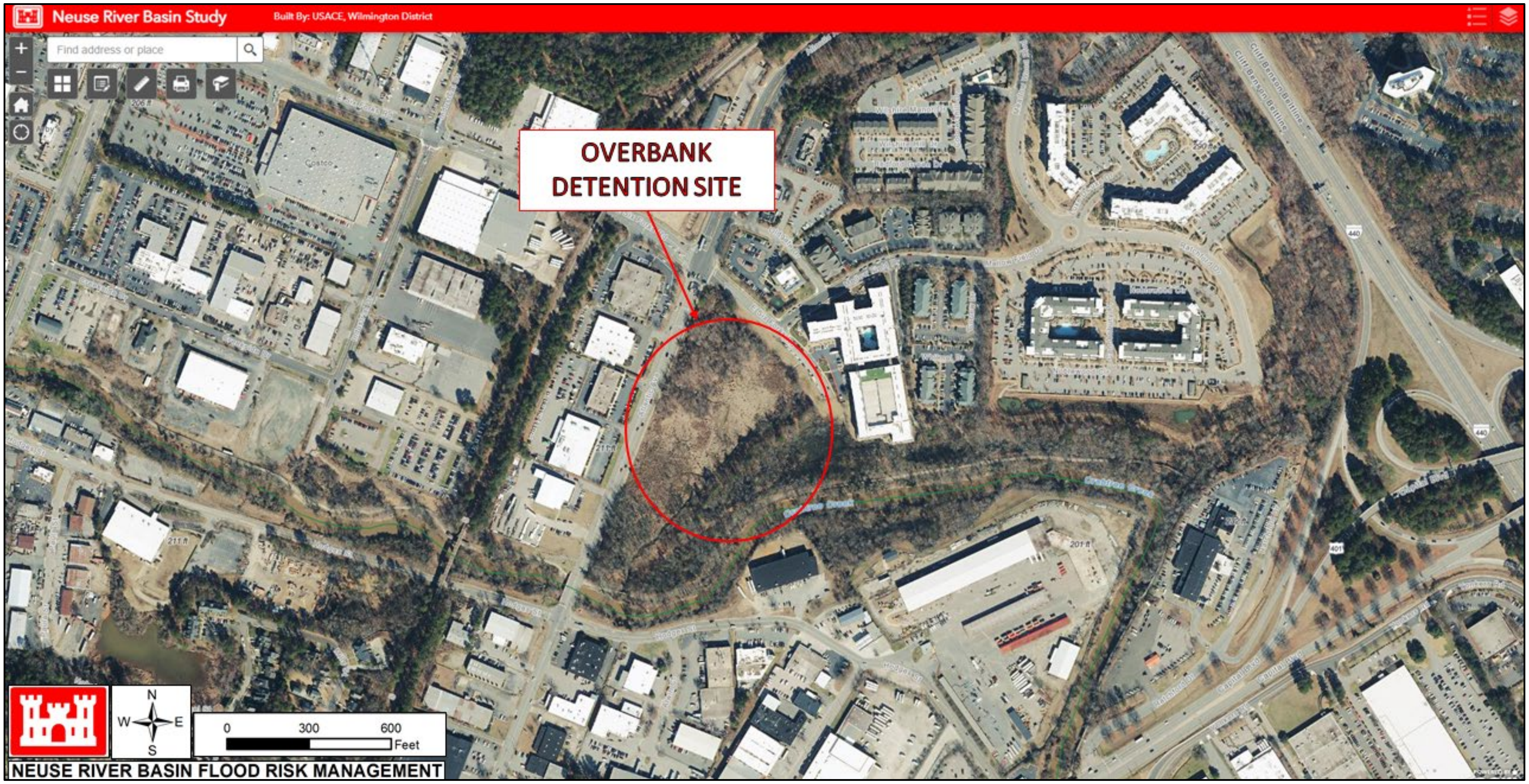

Figure 189. Crabtree Creek Overbank Detention Site

7.3.13 Modification of Existing Detention Structures

This measure was proposed upon initial investigation of several existing Natural Resources Conservation Service (NRCS) detention structures within the Crabtree Creek watershed. There was also interest expressed during coordination with the City of Raleigh to assess the potential for additional reservoir storage capacity to address flooding concerns along Crabtree Creek. The NRCS structures were originally proposed in the 1960s as part of a watershed masterplan (Crabtree Creek Watershed Work Plan, SCS, 1963). During the 1970s and 1980s, a number of these structures were constructed and are currently operated and maintained at a municipality level. A list of detention structures constructed following the 1963 report is listed in Table 79. A general location map of select NRCS supplied by the City of Raleigh is shown in Figure 190.

Table 79. Select NRCS Detention Structures in Wake County

Name	Location		
FCS #1 Sorrell's Grove Reservoir	207 Sorrell Grove Church Rd.	Morrisville	NC
FCS #11A Richland Creek Lake Reservoir	5124 Richland Dr.	Raleigh	NC
FCS #13 Shelly Lake Reservoir	1400 W Millbrook Rd.	Raleigh	NC
FCS #18 Cole's Branch Reservoir	690 Crabtree Crossing Pkwy.	Cary	NC
FCS #2 Hatchers Grove Reservoir	1776 Morrisville Pkwy.	Morrisville	NC
FCS #20A Brier Creek Reservoir	Pleasant Grove Rd.	Raleigh	NC
FCS #22 Lake Lynn Reservoir	Lynn Rd.	Raleigh	NC
FCS #23 Lake Crabtree Reservoir	2139 Old Reedy Creek Rd.	Cary	NC
FCS #3 Bond Lake Reservoir	801 High House Rd.	Cary	NC
FCS #5A Page Lake Reservoir	Triple Oak Dr.	Morrisville	NC

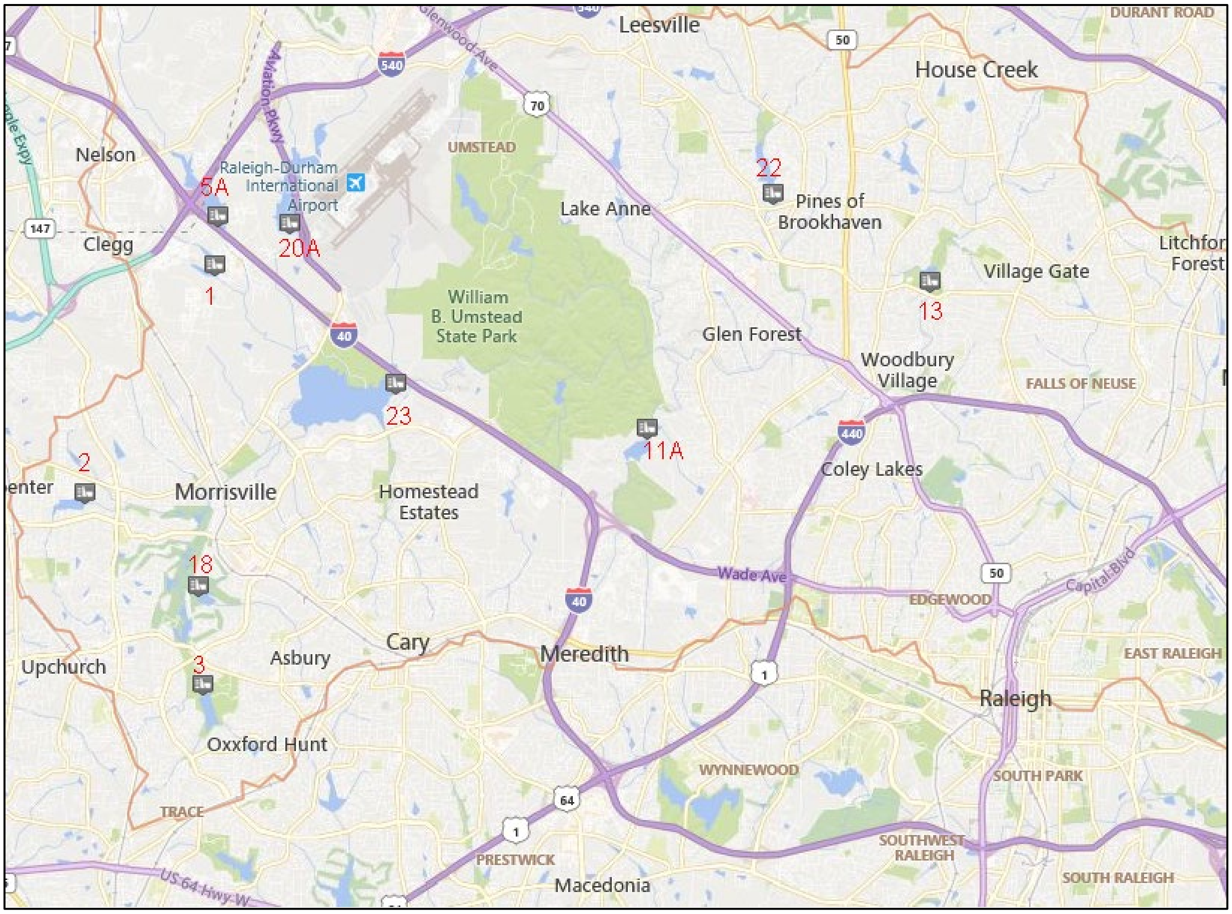

Figure 190. General Location of Select NRCS Detention Structures in Wake County

Notably, not all structures detailed in the 1963 report were eventually constructed. There appeared to be some re-design of select detention structures following this report and some sites were combined or re-configured. Availability of historical documentation following the 1963 report was sparse, so details of this re-scoping process are largely unknown.

For the purposes of the Neuse River basin study, a number of these NRCS detention structures were initially screened from consideration partially due to their relatively small size and distance along a tributary upstream from Crabtree Creek, which was determined to be the primary flooding source, based on historical documentation and sponsor feedback. Furthermore, upon examining the site configuration of the upper Crabtree Creek watershed, improvements were initially limited to a single detention structure (#23 in Figure 190). This limitation was based on the presence of Lake Crabtree in Cary, NC, the largest detention structure, which functioned to capture flow from 6 smaller NRCS sites as well as local rainfall runoff. It was impractical in improving these 6 upland sites without also including the larger Lake Crabtree, so the viability of a Lake Crabtree modification was critical to preliminary screening of this measure.

Overall, the structural design of these detention sites was similar in nature, which involved a two-stage principal spillway designed for floodwaters that were temporarily detained in an upload storage area to be automatically released through conduits at a predetermined rate. These sites were designed such that during a flood on-site management was typically not required thus reducing the complexity of operations. However, this passive design would potentially require significant structural modification in order to increase the outflow capacity, especially if more active regulation is desired.

At this evaluation level, improvements to the Lake Crabtree site were limited to increasing the available flood storage pool by excavating material within the established lake footprint and not modifying the existing outlet works. This excavation would allow for additional acre-feet of floodwaters to be temporarily detained with the target of reducing the severity of the flood hydrograph peak as it made its way downstream into the more populated areas of the Crabtree Creek watershed. An elevation-surface area relationship for water levels between the assumed normal pool capacity (elevation 275.26 ft, NAVD88) and the maximum pool capacity (elevation ~300.0 ft, NAVD88) was developed using the ArcMap surface volume tool. Terrain values were based on QL2 LiDAR. A multiplier was applied to the existing conditions surface area at top of dam to determine a range of total reservoir capacity increase. The new capacity was then distributed to the reservoir area between normal pool and top of dam based on the shape of the existing elevation-surface area curve. This range would provide a general idea of expected reductions in downstream discharge. The existing conditions and range of proposed conditions elevation-surface area curves are shown in Figure 191.

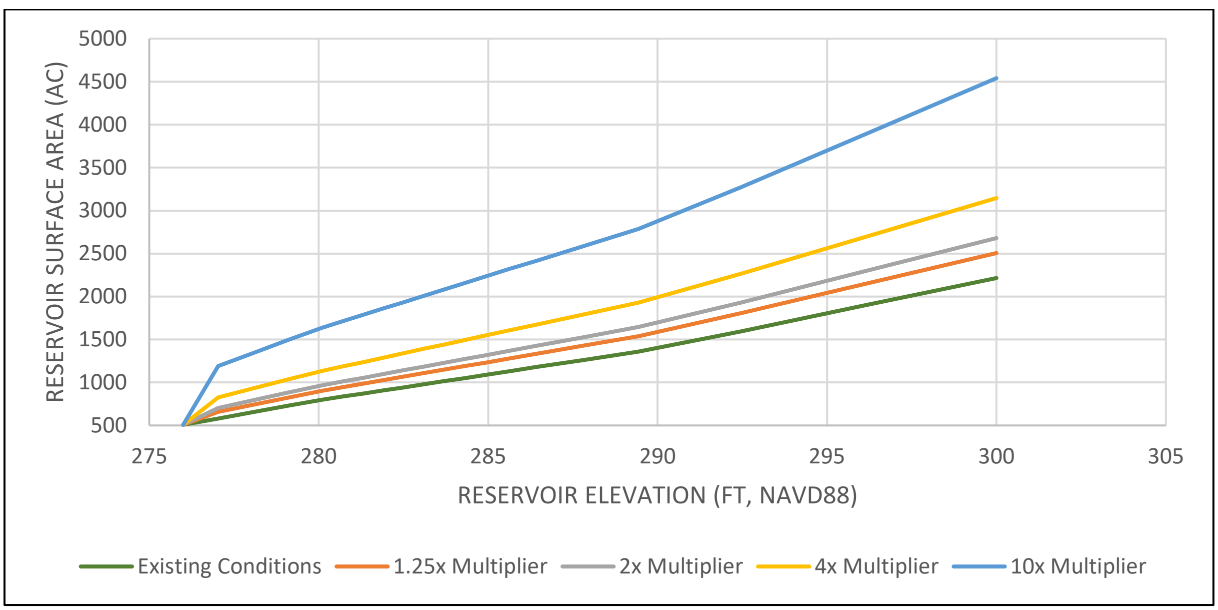

Figure 191. Existing and Proposed Elevation-Surface Area Curves for Lake Crabtree

The Crabtree Creek HEC-HMS study model was used for this evaluation. Elevation-area functions were created for the proposed multiplier curves using the paired data manager. Simulations were run with each curve in place for Lake Crabtree over the suite of design storms. Event hydrographs were reviewed immediately downstream of Lake Crabtree Dam as well as further downstream at Ebenezer Church Rd. This road crossing represented the first portion of Crabtree Creek floodplain that contained damageable structures downstream of William B. Umstead State Park. The locations of these assessed hydrographs are shown in Figure 192.

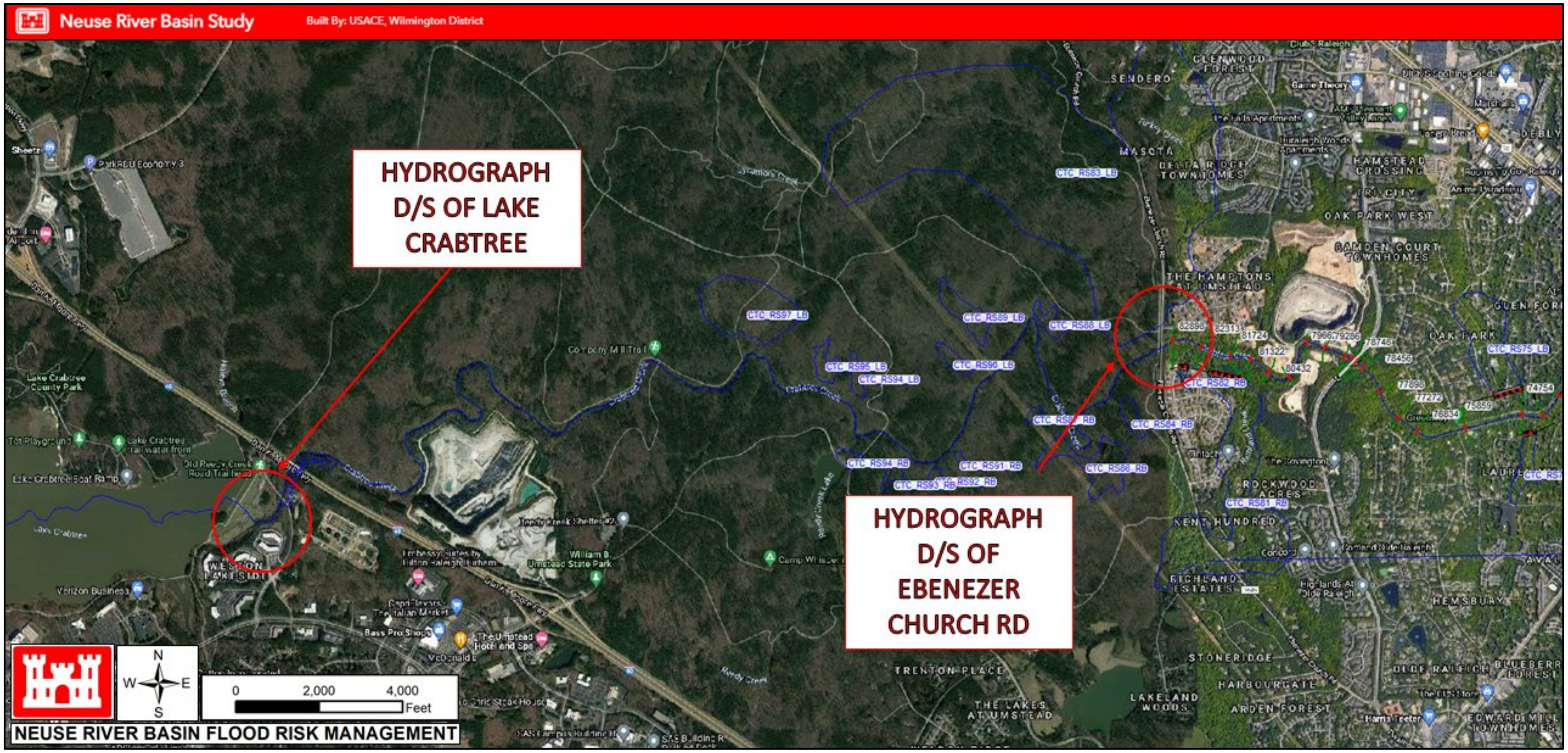

Figure 192. Lake Crabtree – assessed hydrograph locations

HEC-HMS results for increased reservoir capacity scenarios showed a modest reduction in the hydrograph peak discharge as flows exited the dam. The 0.01-, 0.005-, and 0.002-AEP peak discharges were reduced by 73-, 56-, and 40-percent, respectively. However, hydrograph attenuation over the 5.5-mile distance downstream of the dam resulted in a minor reduction in peak flows by the time it reached Ebenezer Church Rd. At this location, peak discharges were only reduced by 20 cfs for the 0.002-AEP event. A comparison of design storm discharges for existing and improved reservoir capacity conditions is shown in Figure 193.

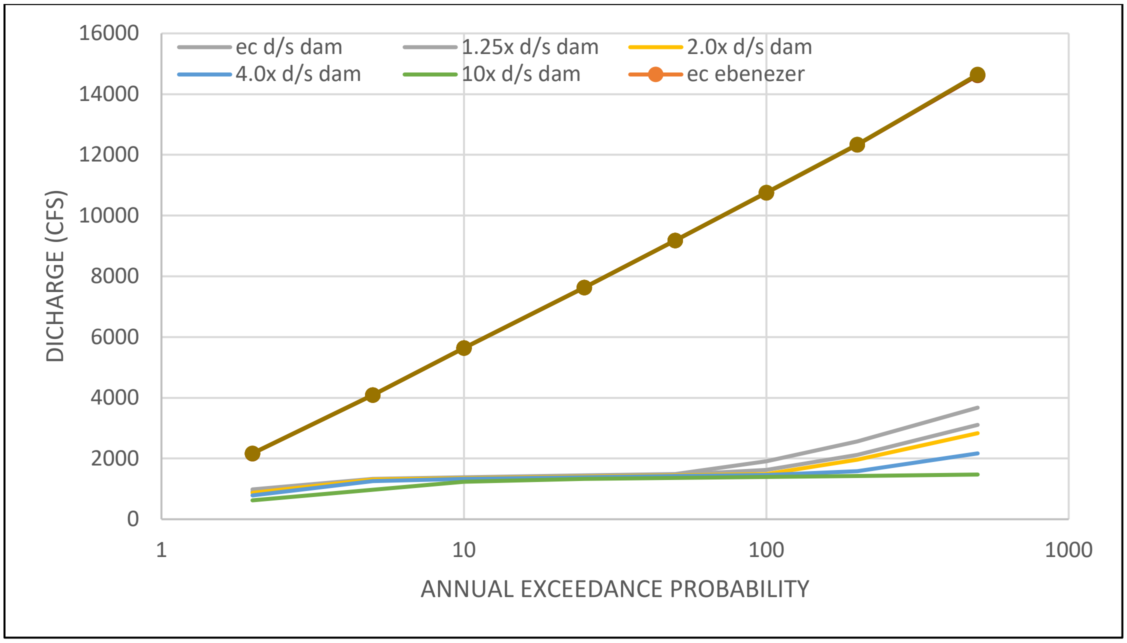

Figure 193. Discharge Comparison – Existing and Improved Reservoir Capacity Conditions

Upon determining that improved Lake Crabtree reservoir capacities had a negligible impact to the existing downstream flooding, discharge over the total hydrograph duration was reviewed. The 0.002-AEP event hydrograph at Ebenezer Church Rd. is shown in Figure 194.

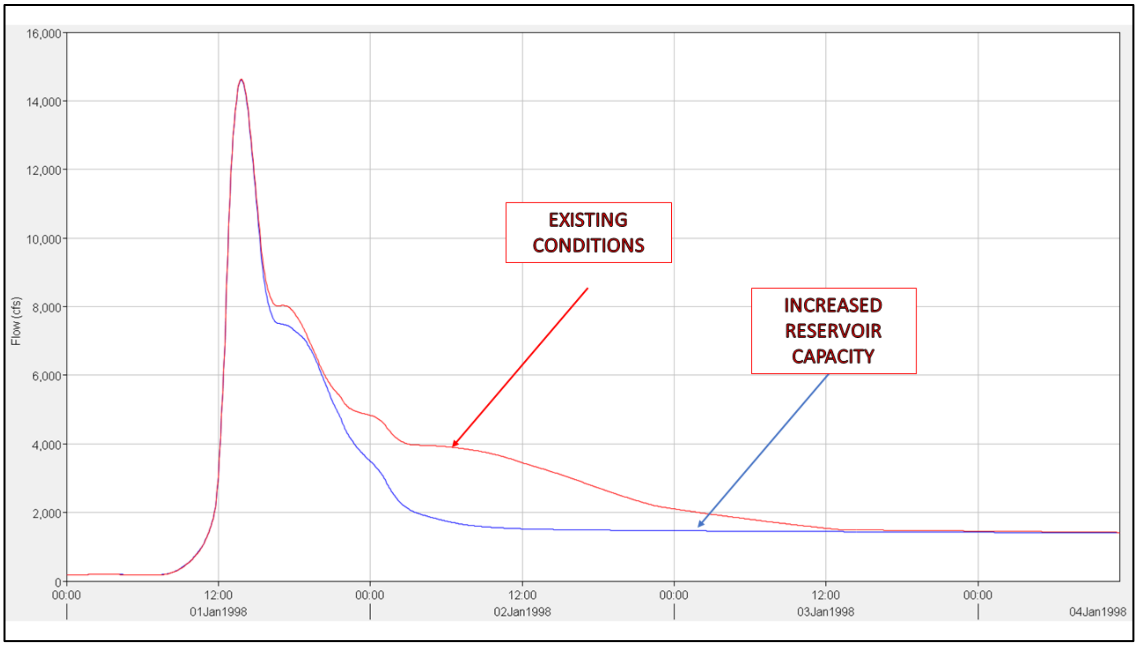

Figure 194. Existing Conditions vs. Increased Reservoir Capacity @ Ebenezer Church Rd

The improved conditions hydrograph did show a reduction in discharge for the receding limb and a quicker return to baseflow. It was concluded that additional reservoir capacity was not effective at reducing the peak discharge, which was identified as the primary driver for event damages. Based on this engineering assessment, modification to the existing NCRS detention structures in the Crabtree Creek watershed was screened from further consideration.

7.3.14 Clearing and Snagging Along Crabtree Creek In Raleigh, NC

This measure involved removal of vegetation along the bank and selective removal of snags, drifts, and other obstructions from the Crabtree Creek channel. Historically, there have been challenges with preventing woody debris and other dislodged material from creating blockages at the numerous crossings throughout the creek’s length, given a significant flood event. At this feasibility planning-level, and without a recent physical survey of the creek, a conservative approach was taken to establish the length over which this measure would take place. It was determined that clearing and snagging would be done for approximately 15.7 miles of Crabtree Creek, beginning at its mouth and stopping at Ebenezer Church Rd.

This measure was assessed using the Crabtree Creek HEC-RAS model. Manning’s roughness values for the channel geometry were reduced to about 90-percent of FWOP values, on average. In general, FWP values were not significantly lower than FWOP due to uncertainty without a physical survey to confirm existing conditions. The largest difference in n-values between FWOP and FWP was 0.004. The FWP condition was simulated under the suite of design storms. Across all design storms, there was an average reduction in WSEL of 0.2-ft. While this reduction did not have a significant impact to the FWOP flooding, it had potential as a component of a larger alternative plan. Due to this potential, it was carried forward to alternative plan formulation.

8 Preliminary Structural Alternatives

Despite the overall large study area of the Neuse River basin, the hydraulically separated measure locations that were carried forward in the evaluation process made for efficient plan formulation to identify structural alternatives. An overview of specific study areas and their measures that were considered for alternative plan formulation is listed in Table 80.

Table 80. Measures Carried Forward to Alternative Plan Formulation

Location in Basin	Measure Type		
	Channel Modification	Bridge/Culvert Modification	Clearing and Snagging
Wilson, NC - Hominy Swamp Creek	✓	✓	
Raleigh, NC - Crabtree Creek	✓	✓	✓
Kinston, NC - Neuse River	✓		

A number of tributary-specific alternatives were identified, predominately based on an increasing level of design complexity and magnitude of potential FRM improvement.

8.1.1 Alternative HS-S1

This alternative was comprised of the channel modification measure evaluated for Hominy Swamp Creek in Wilson, NC. The measure included all nine segments of channel bench modifications along Hominy Swamp Creek. As this was the only measure included in this structural alternative, the WSEL reductions detailed in Section 7.3.2 were still applicable to alternative evaluation.

8.1.2 Alternative HS-S2

This alternative was comprised of the channel modification measure, as described in Alternative 1, plus the Hominy Swamp Creek CSX railroad culvert improvement that was detailed in Section 7.3.10. The intent in this alternative was to combine the improved conveyance offered by the channel bench measure with the larger culvert opening through the CSX railroad. The overall WSEL reduction related to the channel bench design would also alleviate the downstream impacts associated with the CSX measure. Maximum WSEL reductions within the Hominy Swamp Creek floodplain between the Tarboro St and CSX railroad crossings for the 0.04-, 0.01-, and 0.002-AEP events were 2.3-ft, 1.8-ft, and 1.2-ft, respectively. Select design storm profiles of FWOP and alternative 2 conditions are shown in Figure 195.

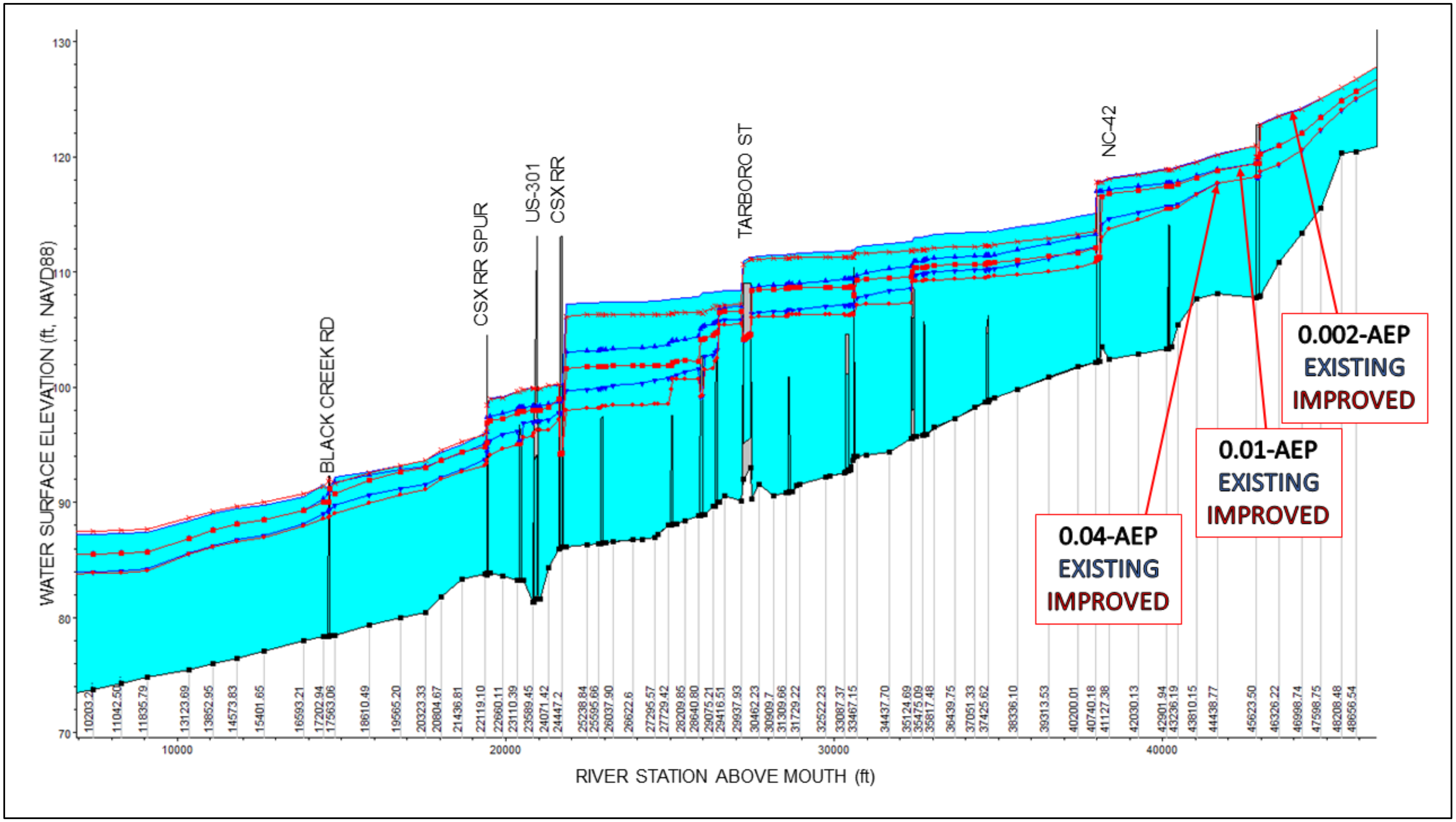

Figure 195. Select Design Storm Profiles for FWOP and Alternative 2 Conditions (Hominy Swamp Creek)

8.1.3 Alternative CTC-S3

This alternative was comprised of the channel modification measure evaluated for Crabtree Creek in Raleigh, NC. The measure included all seven segments of channel bench modifications along Crabtree Creek, as detailed in Section 7.3.3. The alternative also included the clearing and snagging measure, as describe in Section 7.3.14. The intent in this alternative was to combine the two measures that represented simplified engineering methods to improve FRM. These two measures were not structurally complex in their design, which primarily involved excavation and debris removal. Furthermore, the measures carried negligible mitigation requirements (no measurable increase in WSEL above FWOP conditions. Maximum WSEL reductions within the Crabtree Creek floodplain between the Lassiter Mill Rd and Norfolk Southern railroad crossings for the 0.1-, 0.01-, and 0.002-AEP events were 1.8-ft, 1.5-ft, and 1.3-ft, respectively. Select design storm profiles for FWOP and alternative 3 conditions are shown in Figure 196.

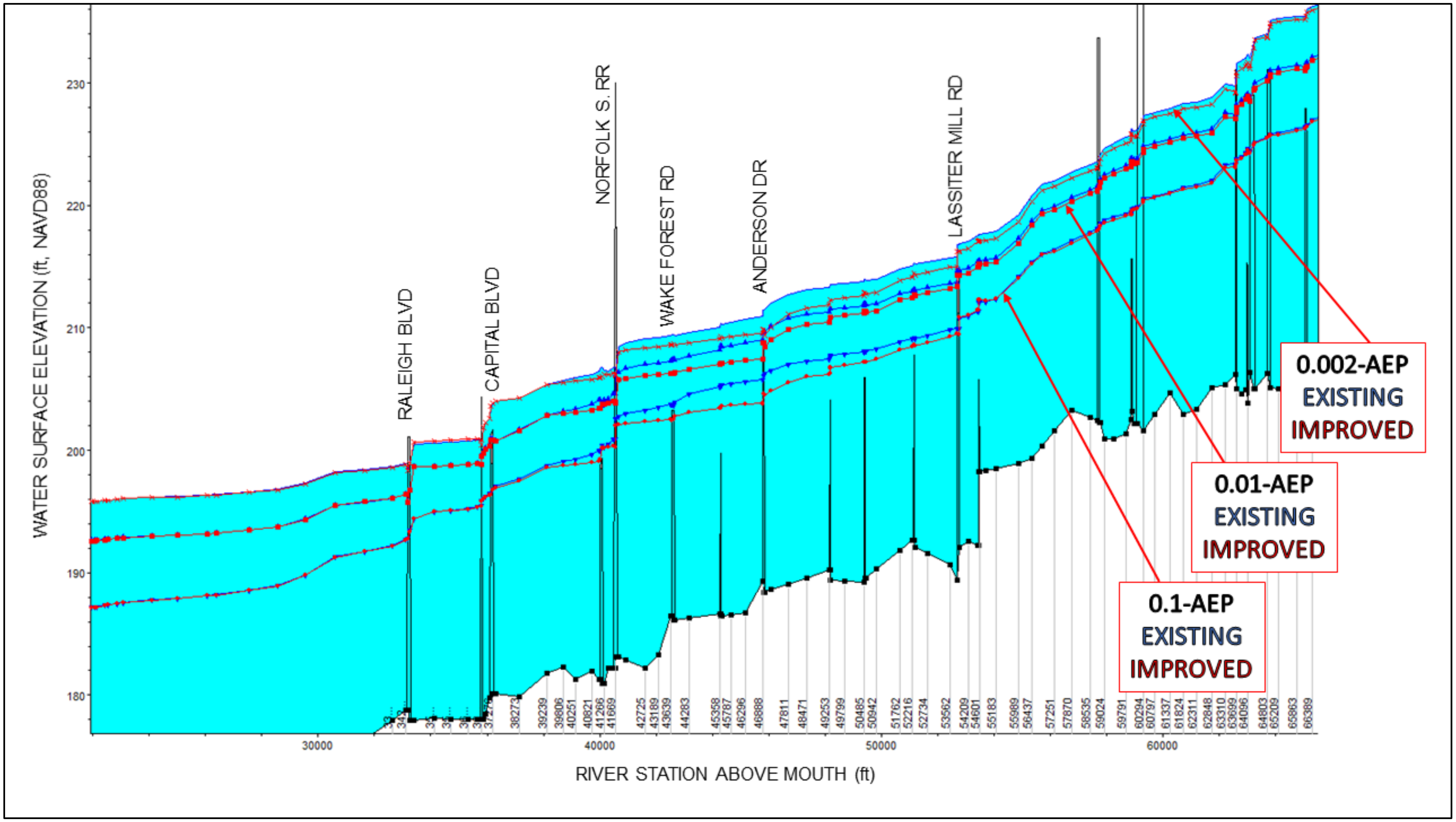

Figure 196. Select Design Storm Profiles for FWOP and Alternative 3 Conditions (Crabtree Creek)

8.1.4 Alternative CTC-S4

This alternative was comprised of the channel modification and clearing and snagging measures in Alternative 3, plus the bridge modification measure at the Norfolk Southern railroad crossing. The bridge modification involved construction of a rectangular concrete flume within the Crabtree Creek channel as it passed under the railroad bridge, as described in Section 7.3.9. The intent of this alternative was to reduce potential mitigation requirements related to increased WSEL above FWOP conditions by combining the bridge modification measure with the alternative 3 measures. The WSEL reductions associated with the channel modification and clearing and snagging measures would offset the increases directly related to the concrete flume. Maximum WSEL reductions within the Crabtree Creek floodplain between the Lassiter Mill Rd and Norfolk Southern railroad crossings for the 0.1-, 0.01-, and 0.002-AEP events were 2.2-ft, 2.0-ft, and 1.9-ft, respectively. Select design storm profiles for FWOP and alternative 4 conditions are shown in Figure 197.

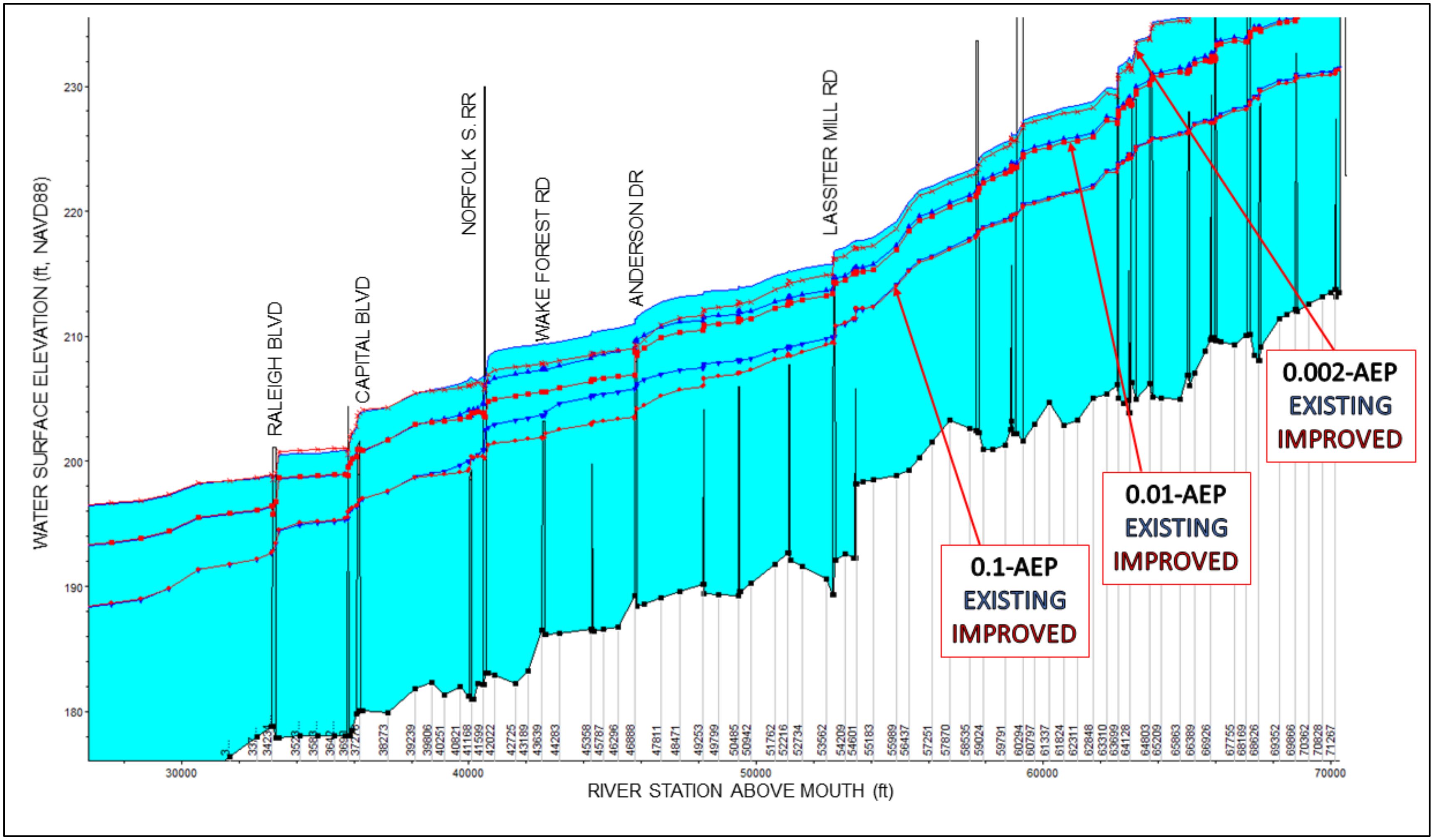

Figure 197. Select Design Storm Profiles for FWOP and Alternative 4 Conditions (Crabtree Creek)

8.1.5 Alternative CTC-S5

This alternative was comprised of the channel modification, clearing and snagging, and bridge modification at the Norfolk Southern railroad crossing in alternative 4, plus the bridge modification measure at the Raleigh Blvd crossing. The Raleigh Blvd bridge modification involved construction of a triple box culvert within the left overbank, through the existing Raleigh Blvd embankment, as described in Section 7.3.9. The intent in this alternative was similar to Alternative 4. The inclusion of the Raleigh Blvd bridge modification would provide for the greatest WSEL reduction, relative to the other standalone measures evaluated for the Crabtree Creek study area. Maximum WSEL reductions within the Crabtree Creek floodplain between the Lassiter Mill Rd and Norfolk Southern railroad crossings for the 0.1-, 0.01-, and 0.002-AEP events were 2.3-ft, 2.1-ft, and 2.3-ft, respectively. Select design storms profiles for FWOP and alternative 5 conditions are shown in Figure 198.

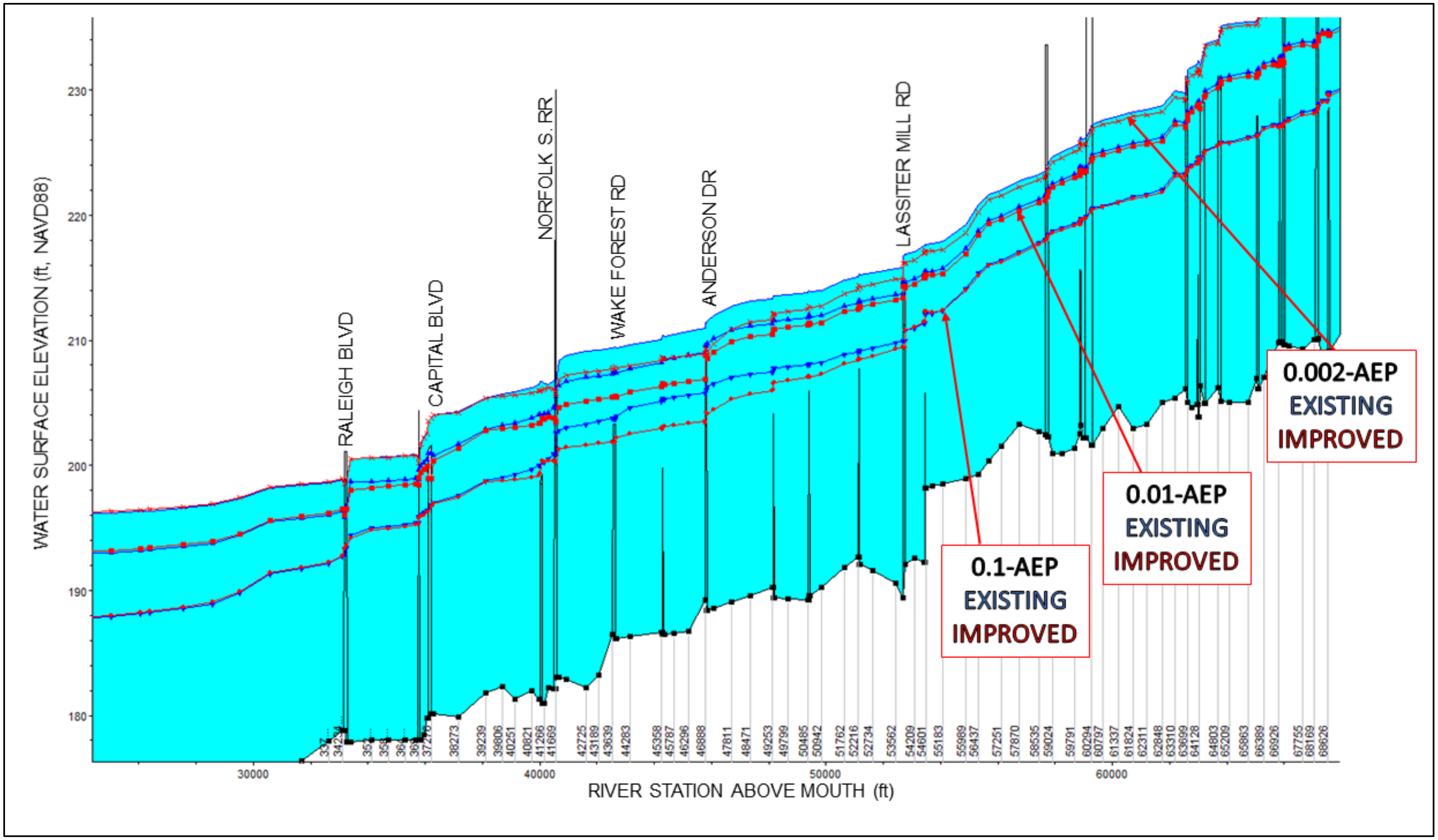

Figure 198. Select Design Storm Profiles for FWOP and Alternative 5 Conditions (Crabtree Creek)

8.1.6 Alternative MS-S1

This alternative was comprised of the channel modification measure evaluated for the Neuse River mainstem in Kinston, NC. The measure involved two channel bench segments within the overbank floodplain of the Neuse River. As this was the only measure included in this structural alternative, the WSEL reductions detailed in Section 7.3.1 were still applicable to alternative evaluation.

9 Refined Structural Alternatives

Upon completion of FWP economic analysis for the preliminary alternatives, it was determined that no structural alternative produced a benefit-to-cost ratio above 1.0. Specifically, overall perceived damages under FWOP conditions revealed significant challenges in the ability for structural measure refinement to cause an alternative plan to reach a benefit-to-cost ratio of 1.0. Based on the unlikelihood for any evaluated structural measure to be economically viable, all alternative plans that moved forward from this evaluation were comprised of nonstructural measures only.

10 Flood Risk Management Uncertainty

10.1 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). While their study area was significantly smaller than that of the Neuse River FRM study, the primary drivers of uncertainty are similar.

> 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.

10.2 Frequency and Stage-Discharge Uncertainty

In accordance with EM 1110-2-1619, Risk-Based Analysis for Flood Damage Reduction Studies, uncertainties pertaining to frequency-discharge and stage-discharge were described using methodologies provided in Chapters 4 and 5 of the referenced EM.

Estimation of frequency-discharge uncertainty was based on equivalent record lengths, as provided in Table 4-5 of EM 1110-2-1619. Due to the large study area and the presence of regulated flow from Falls Lake, there was a wide range of available gage records. Each of the hydraulically assessed subbasins, Crabtree Creek, Hominy Swamp Creek, Adkins Branch, and Big Ditch, as well as the Neuse River mainstem were assigned equivalent records lengths associated with available gage records in their specific basin as listed in Table 81.

Table 81. Equivalent Record Lengths

Hydrologic Study Model	Equivalent Record Length (yr)
Neuse River Mainstem	30
Crabtree Creek	30
Hominy Swamp Creek	25
Big Ditch	25
Adkins Branch	25

As mentioned above, regulated flows from Falls Lake Dam affected the period of record for a majority of streamflow gages in the Neuse River mainstem study area and is reflected in an equivalent record length of approximately 30 years. While some streamflow gages had recorded data as far back as the 1930s, conversion of flow from unregulated to regulated was not conducted as part of this study and the regulated period from early 1980's to current year was deemed appropriate. With a similar equivalent record length of 30 years, and while unaffected by Falls Lake Dam regulated flows, the Crabtree Creek subbasin lacked usable streamflow records prior to the mid-1980's. For the remaining smaller subbasin study areas, an equivalent record length of 25 years was based on developed of their respective hydrologic models through the use of regional model parameters, and also assisted by calibration of the larger Neuse River mainstem hydrologic model.

Stage-discharge uncertainty was assessed by methods provided in Chapter 5 of EM 1110-2-1619. Standard deviations of hydraulic roughness coefficients used in the study models were determined from reference Figure 5-4 in Figure 199 below.

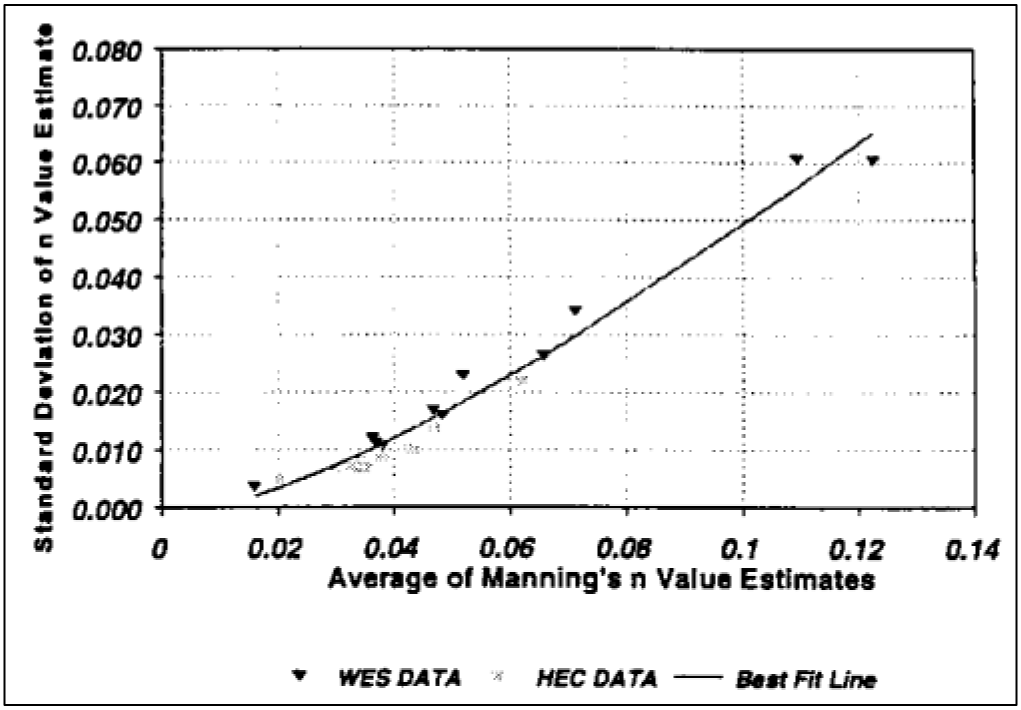

Figure 199. Figure 5-4 from EM 1110-2-1619

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 5-4. This resulted in up to roughly 30 unique standard deviation values for the larger Neuse 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 cross section and then averaged such that each HEC-FDA reach was assigned a specific model uncertainty value (Smodel) in feet. The calculated Smodel was then compared against the minimum standard deviation of error in stage within EM 1110-2-1619, Table 5-2.

Natural uncertainty (Snatural) was calculated partially based on the presence of representative streamflow gages within specific HEC-FDA reaches. The general standard deviation equation was used with data from USGS field measurements plotted against a fitted trendline in Microsoft Excel. Due the broad scale and number of separable study models, not all reaches possessed useable streamflow gages, therefore, Snatural was also based on Equation 5-5 of EM 1110-2-1619 for study model reaches that lacked said gages. Final total uncertainty (Stotal) was the summation of model uncertainty (Smodel) plus natural uncertainty (Snatural). A total uncertainty value was calculated for each HEC-FDA reach, represented by the 0.01-AEP event. For design events more frequent than 0.01, total uncertainty was based on the ratio of peak discharge to the 0.01-AEP. For design events less frequent than 0.01, total uncertainty was held constant. Total uncertainty values per HEC-FDA reach for the 0.01-AEP event across all five study models are listed in Table 82.

Table 82. Total Uncertainty per Study Model for 0.01-AEP event

	Total Uncertainty for Baseline 0.01-AEP Event (ft)				
HEC-FDA Reach	Neuse River Mainstem	Crabtree Creek	Hominy Swamp Creek	Big Ditch	Adkins Branch
1	0.8	0.9	0.8	0.8	0.8
2	0.9	0.8	0.8	0.8	0.8
3	1.0	0.8	0.8	0.8	0.8
4	0.9	0.9	0.8	0.8	1.0
5	0.8	0.9	0.8	0.8	0.8
6	1.1	0.8	0.8	--	--
7	1.1	1.1	0.9	--	--
8	1.1	--	1.0	--	--
9	1.1	--	--	--	--

11 Climate Change Assessment

11.1 Introduction and Background

This assessment of climate change impacts is required by U.S. Army Corps of Engineers (USACE, “the Corps”) Engineering and Construction Bulletin (ECB) 2018-14, “Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects.” This assessment documents the qualitative effects of climate change on the hydrology in the region. The ECB 2018-14 analysis is targeted at identifying potential impacts and risks to the Neuse Basin Feasibility Analysis Study due to climate change.

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

11.2 Neuse River Basin Description

The Neuse River Basin is located in Water Resource Region (i.e., HUC-2 watershed) number 03, the South Atlantic-Gulf Region. The Neuse River is the longest river contained entirely in North Carolina. The Neuse River originates in Wake County North Carolina at Falls Lake and flows southeasterly until it reaches tidal waters near New Bern North Carolina. The river empties into the Pamlico Sound. Major tributaries of the Neuse River include Eno River, Flat River, Little River, Stoney Creek, Crabtree Creek, Walnut Creek, Contentnea Creek and the Trent River. Based on the 2011 National Land Cover Data, the Neuse River Basin's estimated developed area is ~12%, agriculture ~25%, wetlands ~19% grassland/scrub ~10% and forest ~22%.

The Neuse River Basin begins in the Piedmont of North Carolina and extends 275 miles southeast through the Coastal Plan and flows to the Pamlico Sound estuary. The total basin area considered for this climate change assessment covered about 5,630 square miles. The basin encompasses all or part of seven counties. Major population centers in the study area include the cities of Raleigh, Smithfield, Goldsboro, Kinston and New Bern, NC.

11.3 Neuse River Gage Data

The Neuse Basin has 17 stream gage sites, of which 5 are located along the Neuse River mainstem. Listed below in Table 83 are the USGS gages that are within the Neuse Basin.

Table 83. Summary of Available USGS gages located in the Neuse Basin

USGS NO.	Gage Name and Location	DA, mi^2	Latitude	Longitude	Water Quality Data	Start of Record	Latest Record
02085070	Eno River Near Durham, NC	141	36.072	78.908	Y	1963	Present
0208524975	Little River at Farintosh, NC	98.9	36.113	78.859	Y	1995	Present
02086500	Flat River at Dam near Bahama, NC	168	36.148	78.829	Y	1927	Present
02086624	Knap of Reeds Creek near Butner, NC	43	36.128	78.789	Y	1982	Present
02086849	Ellerbe Creek near Gorman, NC	21.9	36.059	78.833	Y	1982	Present
02087183	Neuse River near Falls. NC	771	35.940	78.581	Y	1970	Present
02087324	Crabtree Creek at US 1 at Raleigh, NC	121	35.811	78.611	Y	1990	Present
02087359	Walnut Creek at Sunnybrook Drive near Raleigh, NC	29.8	35.758	78.583	Y	1996	Present
02087500	Neuse River near Clayton, NC	1150	35.647	78.405	Y	1927	Present
02087580	Swift Creek near Apex, NC	21	35.719	78.752	Y	2002	Present
02088000	Middle Creek near Clayton, NC	83.5	35.571	78.591	Y	1939	Present
02088500	Little River near Princeton, NC	232	35.511	78.160	Y	1930	Present
02089000	Neuse River near Goldsboro, NC	2399	35.337	77.998	Y	1930	Present
02089500	Neuse River at Kinston, NC	2692	35.208	77.585	Y	1930	Present
02090380	Contentnea Creek near Lucama, NC	161	35.691	78.109	Y	1964	Present
02091500	Contentnea Creek at Hookerton, NC	733	35.429	77.583	Y	1928	Present
02091814	Neuse River near Fort Barnwell, NC	3900	35.314	77.303	Y	1996	Present

11.4 Literature Review

11.4.1 Observed Trends

11.4.1.1 Temperature

A number of studies focusing on observed trends in historical temperatures were reviewed for this report. These include both national scale studies inclusive of results relevant to Water Resources Region 03 and regional studies focusing more specifically and exclusively on the area. Results from both types of studies are discussed below.

A 2009 study by Wang et al. examined historical climate trends across the continental United States. Gridded (0.5 degrees x 0.5 degrees) mean monthly climate data for the period 1950 – 2000 were used. The focus of this work was on the link between observed seasonality and regionality of trends and sea surface temperature variability. The authors identified positive statistically significant trends in recent observed mean air temperature for most of the U.S. (Figure 200). For the South Atlantic-Gulf Region, mixed results are presented. A positive, but mild, warming trend is identified for most of the area in the spring and summer. For the fall months, the southern portion of the area is shown to be warming while mild cooling is shown in the northern portion of the area. For the winter months, the divide appears to be more east-west, with warming in the east and cooling in the western portion of the area. A later study by Westby et al. (2013), using data from the period 1949 – 2011, moderately contradicted these findings, presenting a general winter cooling trend for the entire region for this time period. The third NCA report (Carter et al., 2014) presents historical annual average temperatures for the southeast region. Their southeast study region is larger than, but inclusive of the South Atlantic-Gulf Region. For this area, historical data generally shows mild warming of average annual temperatures in the early part of the 20th century, followed by a few decades of cooling, and is now showing indications of warming. However, though a seasonal breakdown is not presented, the NCA report cites an overall lack of trend in mean annual temperature in the region for the past century. Details on statistical significance are not provided.

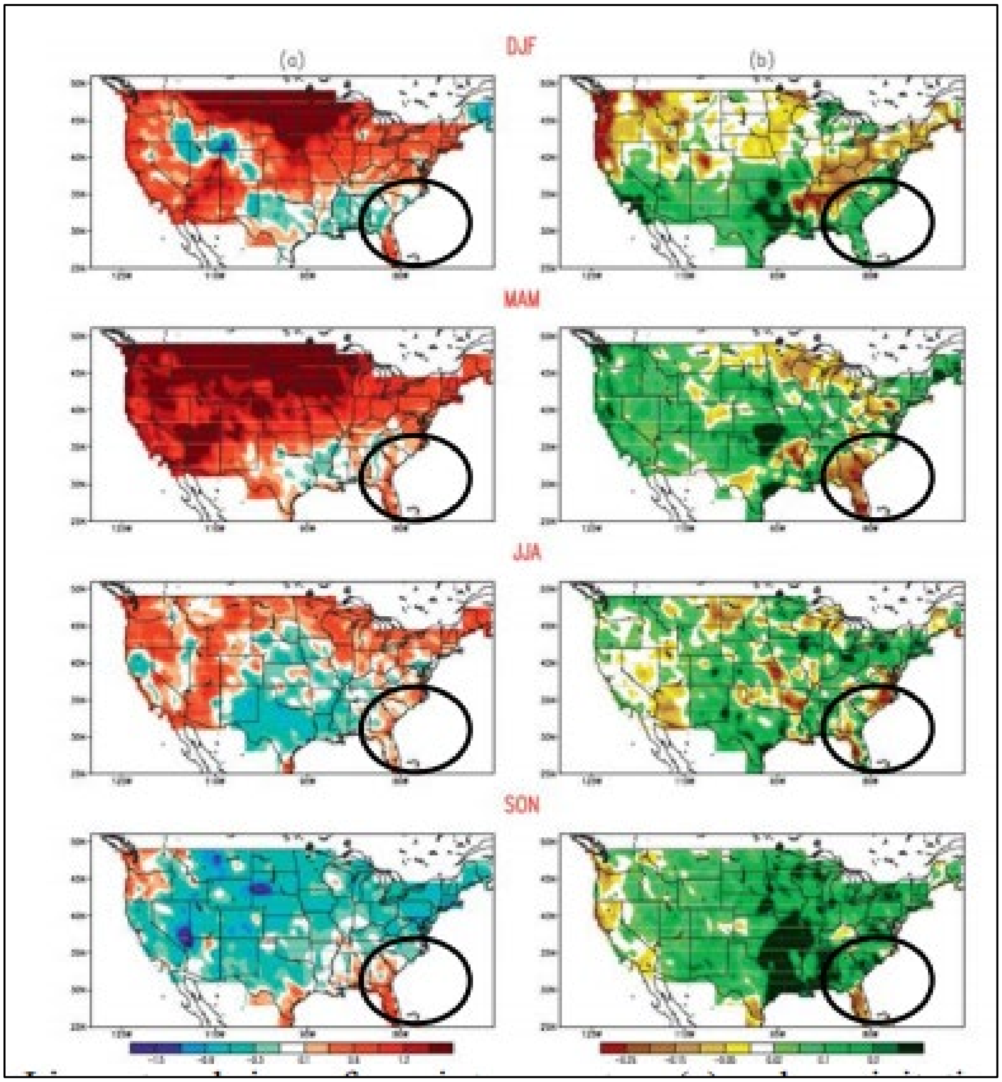

Figure 200. Linear trends in surface air temperature (a) and precipitation (b) over the United States, 1950 – 2000. The South Atlantic-Gulf Region is within the black oval (Wang et al., 2009)

A 2012 study by Patterson et al. focused exclusively on historical climate and streamflow trends in the South Atlantic region. Monthly and annual trends were analyzed for a number of stations distributed throughout the South Atlantic-Gulf Region for the period 1934 – 2005. Results (Figure 201) identified a largely cooling trend for the first half of the historical period and the period as a whole. However, the second half of the study period (1970 – 2005) exhibits a clear warming trend with nearly half of the stations showing statistically significant warming over the period (average increase of 0.7 ºC). The circa 1970 “transition” point for climate and streamflow in the U.S. has been noted elsewhere, including Carter et al. (2014). Trends in overnight minimum temperatures (Tmin) and daily maximum (Tmax) temperatures for the southeast U.S. were the subject of a study by Misra et al. (2012). Their study region encompasses nearly the full extent of the South Atlantic-Gulf Region and used data from 1948 to

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

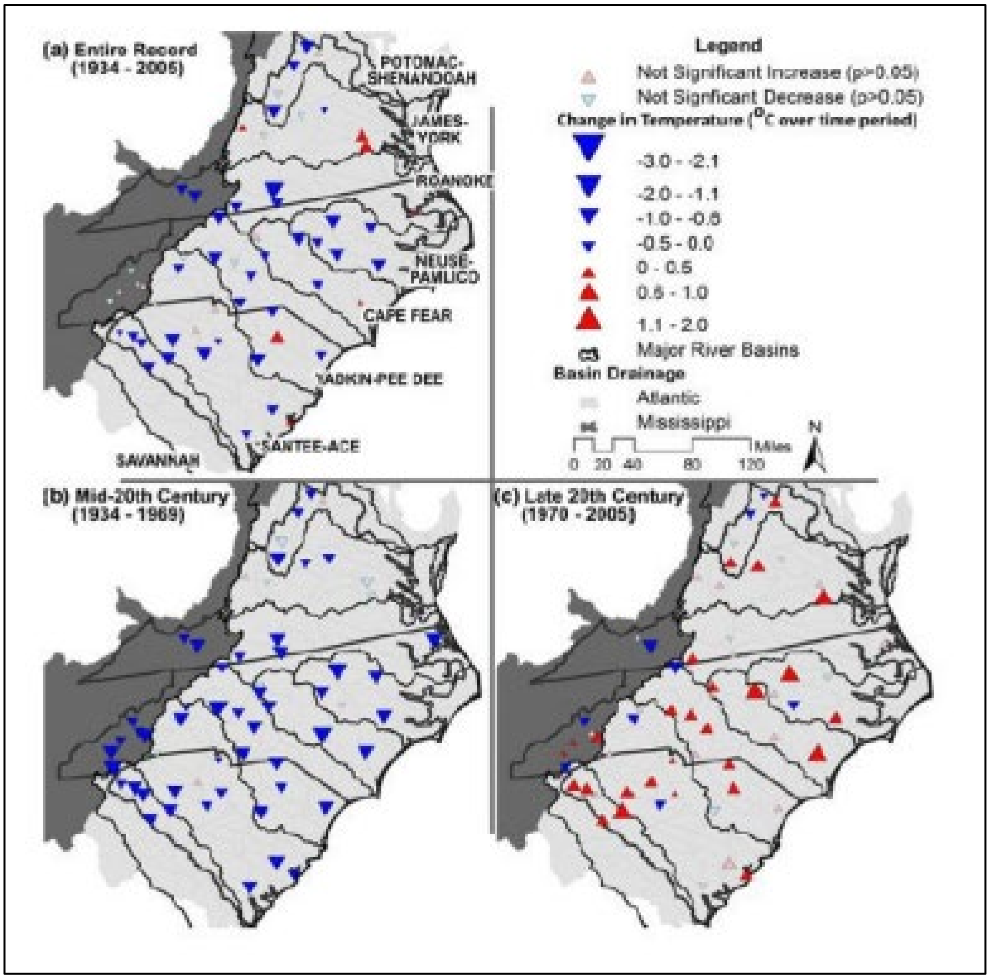

Figure 201. Historical annual temperature trends for the South Atlantic Region, 1934 – 2005. Triangles point in the direction of the trend, size reflects the magnitude of the change. Blue indicates a decreasing temperature trend. Red indicates an increasing temperature trend (Patterson et al., 2012)

In North Carolina specifically the temperatures have risen more than 1°C since the beginning of the 20th century (NCEI, 2022). Winter average temperatures have been increasing with the 2015-2020 period exceeding the levels of the 1930's and 1950's. Summer average temperatures in the 2005-2020 period have been the warmest on record.

Greenhouse gases come from a variety of human activities including: burning fossil fuels for transportation, heat and energy, clearing forests, fertilizing crops, storing waste in landfills, raising livestock, and producing some kinds of industrial products (https://www.epa.gov/ghgemissions/overview-greenhouse-gases). The most common gases referred to as greenhouse gases (GHG's) are: 79% Carbon dioxide (CO_2); 11%

Methane (CH_4); 7% Nitrous Oxide (N_2O); and 3% Fluorinated Gases (which are synthetic, such as: hydrofluorocarbons, perfluorocarbons, sulfur hexafluoride, and nitrogen trifluoride). CO_2 is the most abundant of GHG's being emitted to the atmosphere, and the primary source of this emission is from human activities such as: combustion of fossil fuels for transportation, electricity generation, and industrial processes. Carbon dioxide emissions can be reduced through energy conservation, more energy efficient products and transpiration, carbon capture and sequestration, and more conservative land use practices. Methane emissions are emitted from a mix of energy, industry/mining, agriculture, waste management/landfills, and land use. Reduction in waste and upgrades/modifications to equipment and practices are the best ways to reduce methane emissions currently. Nitrous Oxide emissions are primarily from agricultural soil management practices, but can also occur with wastewater treatment, production of some chemicals (nitric acid and adipic acid), and some fuel combustion. Nitrous Oxide emissions can be reduced by reducing the frequency and amount of fertilizers used in agriculture, reducing fuel used for vehicles, and upgrading technology in chemical production. Fluorinated Gases are mostly emitted through their use as refrigerants, aerosol propellants, solvents, fire retardants, and some industrial manufacturing processes. Fluorinated Gases can be reduced through better handling methods for refrigerants, gas recycling, leak reduction/prevention, and alternative refrigerants (EPA, 2022). A review of the U.S. Environmental Protection Agency's analysis for climate change for North Carolina titled, "What Climate Change Means for North Carolina," (https://19january2017snapshot.epa.gov/sites/production/files/2016-09/documents/climate-change-nc.pdf) states:

- Most of North Carolina has warmed 0.5-1.0 degrees Fahrenheit in the last 100 years. The southeastern United States has warmed less than most of the nation.
- Tropical storms and hurricanes have become more intense during the past 20 years. Hurricane wind speeds and rainfall rates are likely to increase as the climate continues to warm.
- Increased rainfall may further exacerbate flooding in some coastal areas. Since 1958, the amount of precipitation during heavy rainstorms has increased by 27 percent in the southeast, and the trend toward increasingly heavy rainstorms is likely to continue

11.4.1.2 Precipitation

Palecki et al. (2005) examined historical precipitation data from across the continental United States. They quantified trends in precipitation for the period 1972 – 2002 using NCDC 15- minute rainfall data. For the South Atlantic-Gulf Region, statistically significant increases in winter storm intensity (mm per hour) and fall storm totals were identified for the southernmost portion of South Atlantic-Gulf Region. Additionally, a statistically significant decrease in summer storm intensity was identified for the northern portion of the area.

A 2011 study by McRoberts and Nielsen-Gammon used a new continuous and homogenous data set to perform precipitation trend analyses for sub-basins across the United States. The extended data period used for the analysis was 1895 – 2009. Linear positive trends in annual precipitation were identified for most of the U.S (Figure 202). For the South Atlantic-Gulf Region, results were mixed with some areas showing mild decreases in precipitation and others showing mild increases. No clear trend for the area is evident from these results.

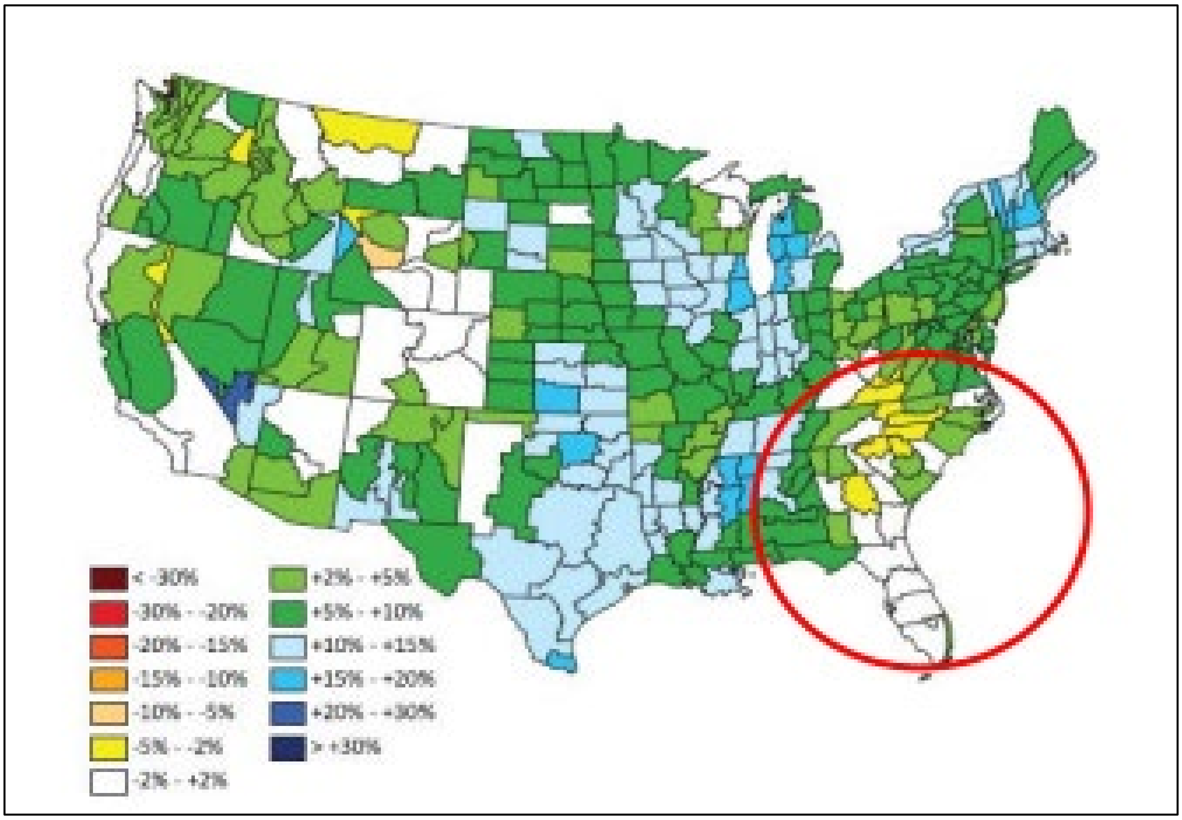

Figure 202. Linear trends in annual precipitation, 1895 – 2009, percent change per century. The South Atlantic-Gulf Region is within the red oval (McRoberts and Nielsen-Gammon, 2011).

Changes in extreme precipitation events observed in recent historical data have been the focus of a number of studies. Studies of extreme events have focused on intensity, frequency, and/or duration of such events. Wang and Zhang (2008) used recent historical data and downscaled Global Climate Models (GCMs) to investigate changes in extreme precipitation across North America. They focused specifically on the changes in the frequency of the 20-year maximum daily precipitation event. The authors looked at both historical trends in observed data and trends in future projections. Statistically significant increases in the frequency of the 20-year storm event were quantified across the southern and central U.S., in both the recent historical data and the long-term future projections (described below). For the South Atlantic-Gulf Region, significant changes in the recurrence of this storm were identified for the period 1977 – 1999 compared to the period 1949 – 1976. An increase in frequency of approximately 25 to 50% was quantified.

In North Carolina (at the Coweeta Laboratory), changes in precipitation variability have been observed (Laseter et al., 2012) (Figure 203). These changes include wetter wet years and dryer dry years compared to the middle of the 20th century. As an example, the wettest year on record occurred in 2009 at Coweeta, and only two years earlier (2007) the driest year on record was observed. This pattern of change is supported by the NCA report (Carter et al., 2014), which states that, "summers have been either increasingly dry or extremely wet" in the southeast region. This assessment is based on analysis of data dating back to the turn of the 20th century.

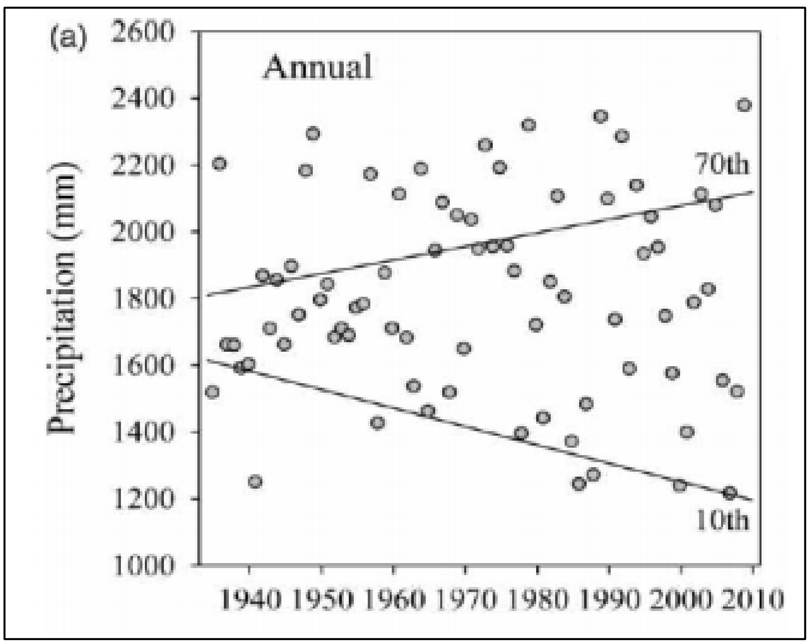

Figure 203. Total annual precipitation at Coweeta Laboratory (North Carolina). Lines show modeled 10th and 90th quantiles as a function of time, 1940 – 2010. (Laseter et al., 2012).

A 2012 study by Patterson et al. focused exclusively on the South Atlantic Region, investigating historical climate and streamflow trends. Monthly and annual trends were analyzed for a number of stations distributed throughout the South Atlantic-Gulf Region for the period 1934 – 2005. Results identified little, if any, patterns of precipitation change in the area over this period. Some sites showed increasing trends, others showed decreasing trends. Overall, and for the full period of record, more sites exhibited mild increases in precipitation than decreases.

In North Carolina there is no overall trend in annual precipitation, but precipitation is generally higher in the summer months (NCEI, 2022).

11.4.1.3 Hydrology

Kalra et al. (2008) found statistically negative trends in annual and seasonal streamflow for a large number of stream gages in the South Atlantic-Gulf Region, analyzed in aggregate, for the historical period 1952 – 2001. This study also identified a statistically significant stepwise change occurring in the mid-1970s, concurrent with the warming climate “transition” period previously noted in Section 2.1, Temperature. These findings are supported by a regional study by Small et al. (2006). This study, using HCDN data for the period 1948 – 1997, identified statistically significant negative trends in annual low flow for multiple stations distributed throughout the South Atlantic-Gulf Region (but even more stations exhibited no significant trend at all).

The Patterson et al. (2012) study also observed a “transition” period occurring around 1970, as well as identified significant decreasing trends in streamflow in the South Atlantic-Gulf Region for the period 1970 – 2005 (Figure 204). Results were mixed for an earlier time period (1934 – 1969), with some decreasing and some increasing trends. These results again highlight the noted transition period of the 1970s.

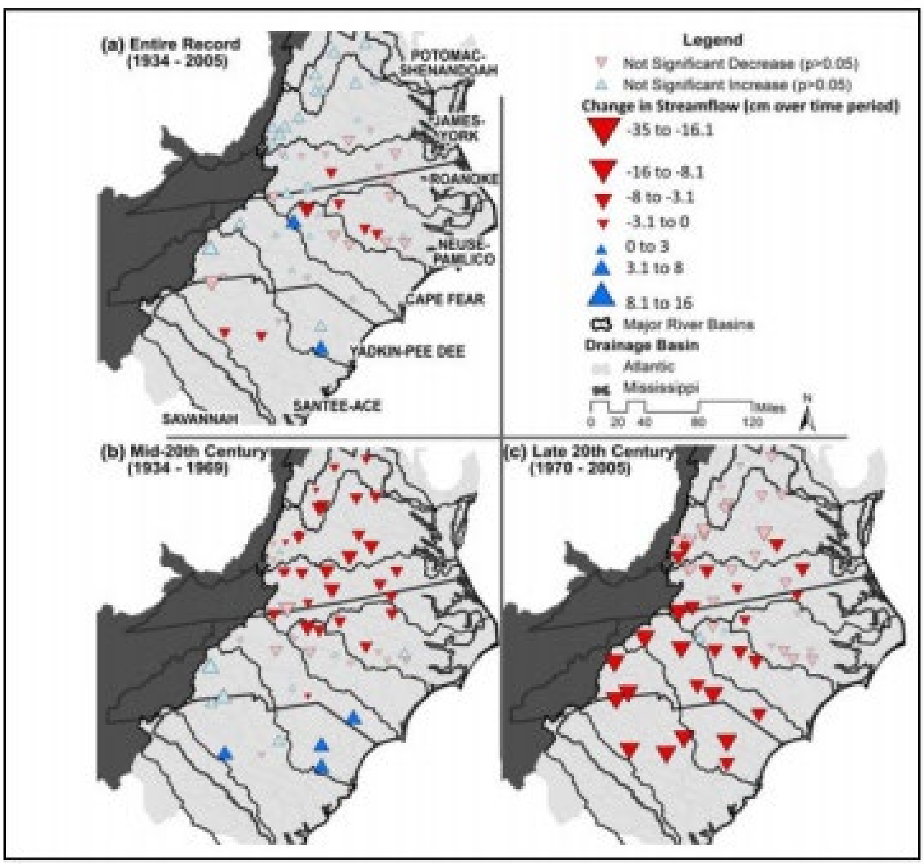

Figure 204. Observed changes in annual streamflow, South Atlantic Region, 1934 – 2005. Triangles point in the direction of the trend, size reflects the magnitude of the change. Blue indicates a decreasing streamflow trend. Red indicates and increasing streamflow trend. (Patterson et al., 2012).

11.4.2 Future Trends

11.4.2.1 Literature Review of Projected Climate Changes

While historical data is essential to understanding current and future climate, non-stationarity in the data (i.e., a changing climate) dictates the use of supplemental information in long-term planning studies. In other words, the past may no longer be a good predictor of the future (Milly et al., 2008). Consequently, the scientific and engineering communities are actively using computer models of the Earth's atmosphere and associated thermodynamics to projected future climate trends for use in water resources planning efforts. Although significant uncertainties are inherent in these model projections, the models, termed global climate models (GCMs), are widely accepted as representing the best available science on the subject, and have proven highly useful in planning as a supplement to historical data. A wealth of literature now exists on the use of GCMs across the globe.

11.4.2.2 Temperature

Elguindi and Grundstein (2013) present results of regional climate modeling of the U.S. focused on the Thornthwaite climate type – a measure of the combination of relative temperature and precipitation projections. For the South Atlantic-Gulf Region, results show a shift from primarily warm wet or warm moist climate type in the latter decades of the 20th century to a much larger proportion of hot moist or hot dry climate type areas by the period 2041 – 2070 (Figure 205).

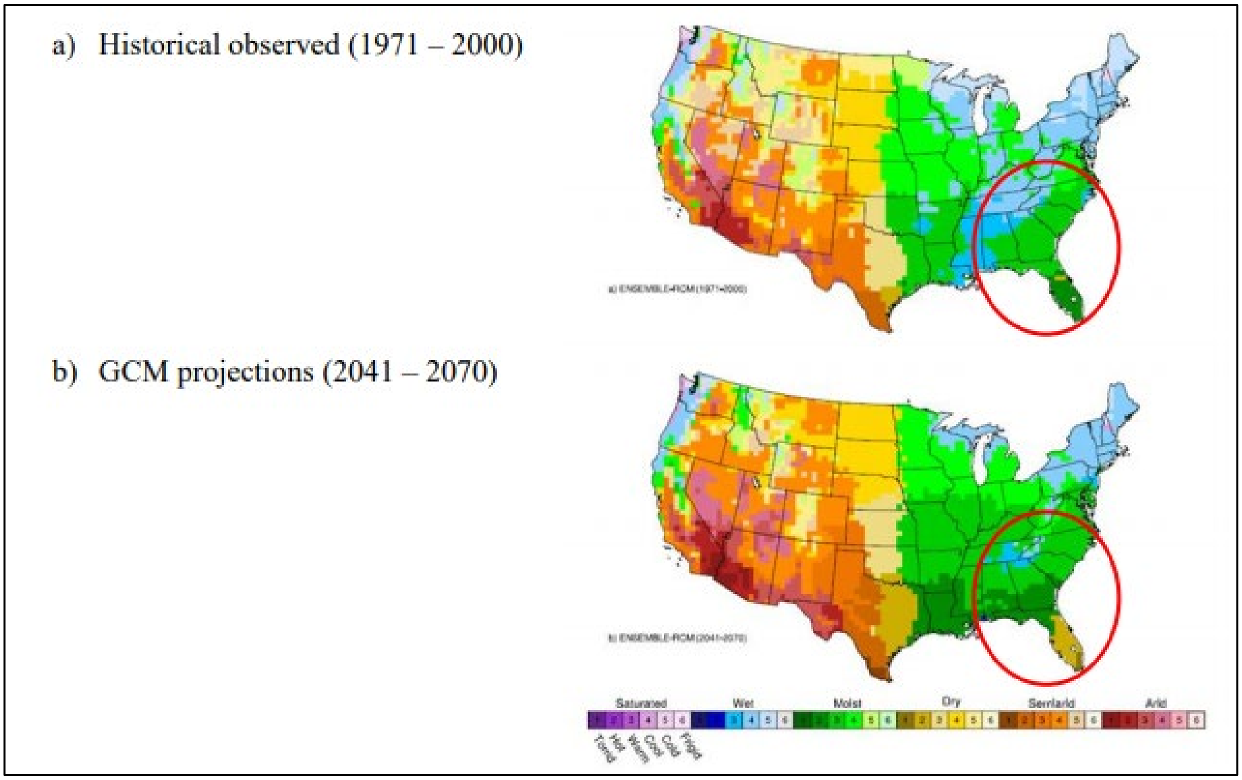

Figure 205. Revised Thornthwaite climate types projected by regional climate models. The South Atlantic-Gulf Region is within the red oval (Elguindi and Grundstein, 2013)

Projections of changes in temperature extremes have been the subject of many recent studies performed at a national scale. A 2006 study by Tebaldi et al. applied nine GCMs at a global scale focused on extreme precipitation and temperature projections. Model projections of climate at the end of the century (2080 – 2099) were compared to historical data for the period 1980 – 1999. For the general southeastern U.S., inclusive of the South Atlantic-Gulf Region, the authors identified small increases in the projected extreme temperature range (annual high minus annual low temperature), a moderate increase in a heat wave duration index (increase of 3 to 4 days per year that temperatures continuously exceeds the historical norm by at least 5 ºC), and a moderate increase in the number of warm nights (6 to 7% increase in the percentage of times in the year when minimum temperature is above the 90th percentile of the climatological distribution for the given calendar year), compared to the baseline period.

At a regional scale, Qi et al. (2009) used two GCMs (CGC1 and HadCMSul2) in combination with hydrologic modeling to project streamflow changes in the Trent River (North Carolina). Temperature projections from these two climate models (Figure 206) show increases of approximately 2 to 4 ºC by the end of the 21st century for their study area.

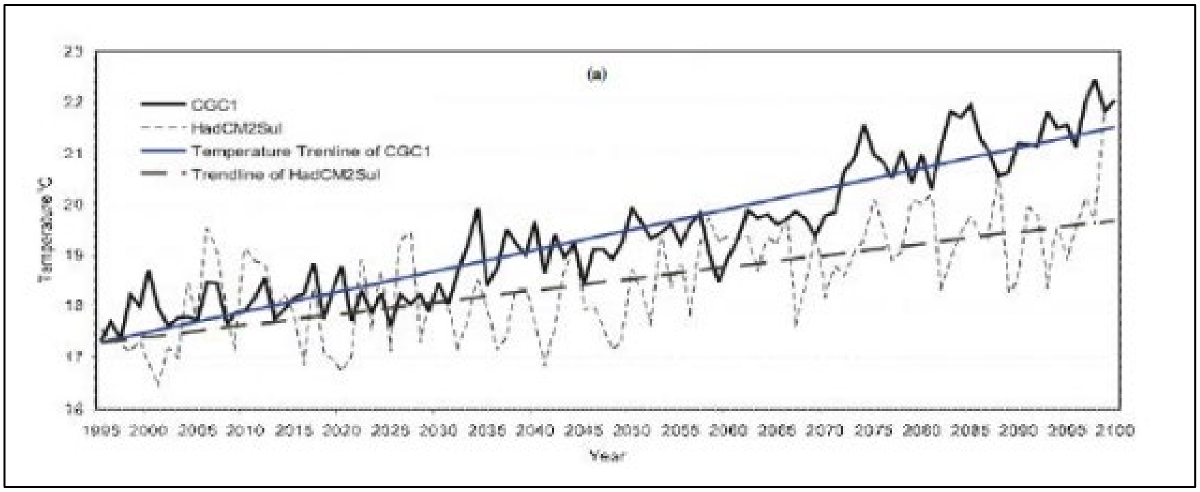

Figure 206. Projected annual average air temperature, Trent River basin, North Carolina, 1995–2100. (Qi et al., 2009)

11.4.2.3 Precipitation

Future projections of extreme events, including storm events and droughts, are the subject of studies by Tebaldi et al. (2006), Wang and Zhang (2008), Gao et al. (2012), and Wang et al. (2013a). The first authors, as part of a global study, compared an ensemble of GCM projections for the southeast U.S. and a 2090 planning horizon with historical baseline data (1980 – 1999). They report small increases in the number of high (> 10 mm) precipitation days for the region, the number of storm events greater than the 95th percentile of the historical record, and the daily precipitation intensity index (annual total precipitation divided by number of wet days). In other words, the projections forecast small increases in the occurrence and intensity of storm events by the end of the 21st century for the general study region. In addition to the historical data trend analyses by Wang and Zhang (2008) described above, these authors also used downscaled GCMs to look at potential future changes in precipitation events across North America. They used an ensemble of GCMs and a single high emissions scenario (A2) to quantify a significant increase (c. 30 to 50%) in the recurrence of the current 20-year 24-hour storm event for their future planning horizon (2075) and the general South Atlantic-Gulf Region (Figure 207). The projected increases in storm frequency presented by Wang and Zhang appear to be more significant than those projected by Tebaldi et al. (2006), but there is agreement on the general trend.

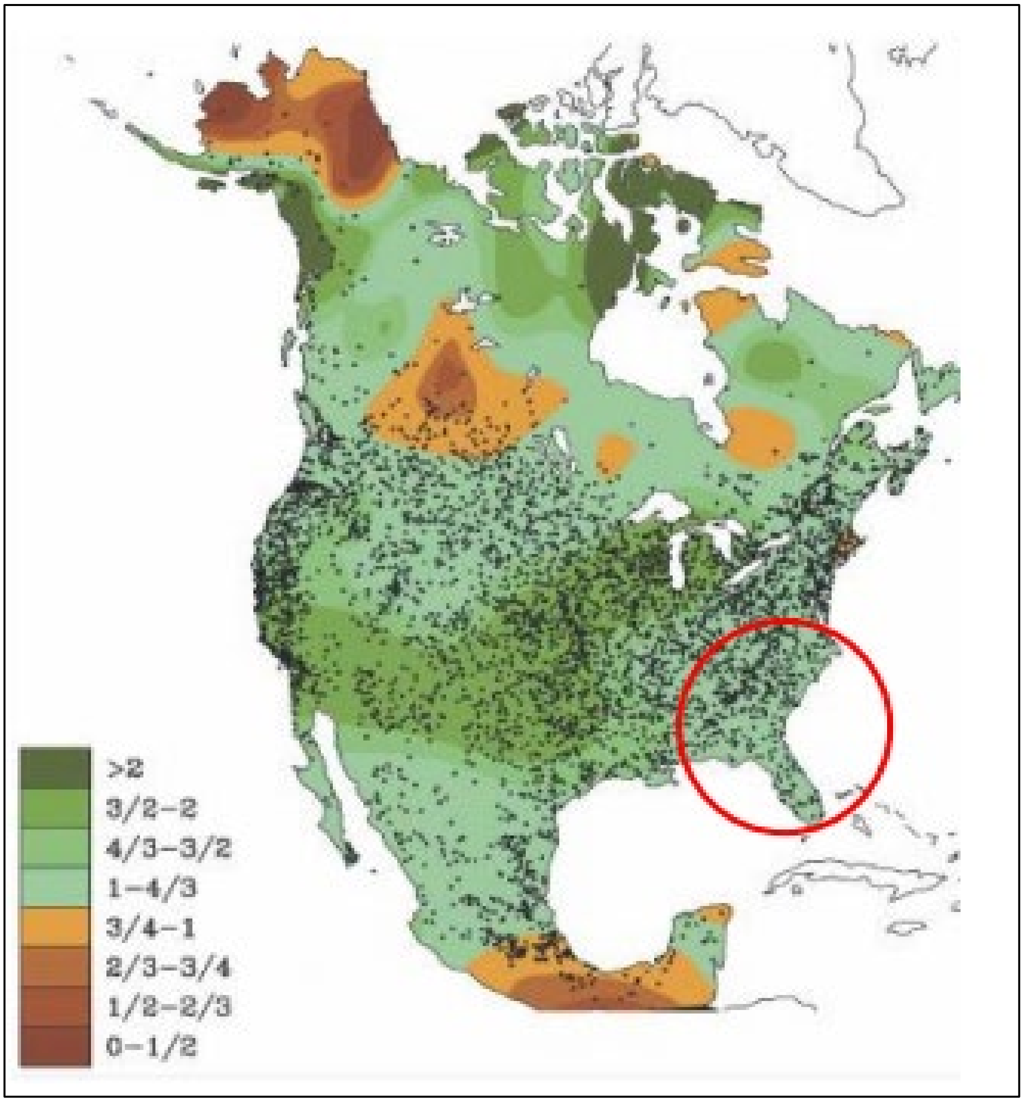

Figure 207. Projected risk of current 20-year 24-hour precipitation event occurring in 2070 compared to historical (1974). A value of 2 indicates this storm will be twice as likely in the future compared to the past. Black dots show the locations of stations. The South Atlantic Gulf Region is within the red oval (Wang and Zhang, 2008).

NCEI 2022 projects an increase in precipitation in North Carolina, primarily in the winter and spring, as well as an increase in hurricane-associated storm intensity and rainfall rates.

11.4.2.4 Hydrology

Study projections from Hagemann et al. (2013) for the general South Atlantic-Gulf Region show an overall decrease in runoff by approximately 200 mm per year for their future planning horizon (2071 – 2100) compared to the recent historical baseline (1971 – 2000) (Figure 208), assuming an A2 emissions scenario.

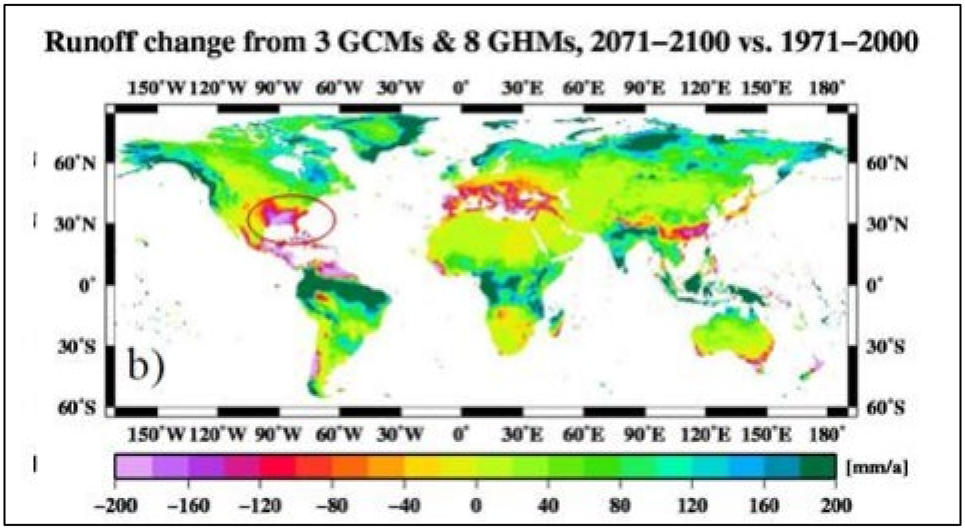

Figure 208. Ensemble mean runoff projections (mm/year) for A2 greenhouse gas emissions scenario, changes in annual runoff, 2085 vs. 1985. The South Atlantic-Gulf Region is within the red oval (Hagemann et al., 2013).

Wu et al. (2014), used the full suite of CMIP3 GCM projections in combination with a lumped rainfall-runoff model to projects future streamflow changes for Coweeta Laboratory, a watershed in North Carolina. The results suggest a likely increase in winter streamflow, however it shows mixed results for other seasons.

No clear consensus was found in projected streamflow changes in the South Atlantic-Gulf Region. Some studies point toward mild increases in flow, others point toward mild decreases in flow.

11.4.3 Summary of Literature Review

A January 2015 report conducted by the USACE Institute for Water Resources (USACE 2015b) summarizes the available climate change literature for this region, covering both observed and projected changes. This summary is represented in Figure 209 below.

The results presented in this review indicate a small upward trending in temperature and a small downward trending in streamflow in the South Atlantic-Gulf Region, particularly since the 1970s. Both temperature and streamflow show majority consensus within the literature. Studies on precipitation show mixed results but with more findings showing an upward, rather than downward, pattern over the past 50 to 100 years. There is a high consensus that future average and maximum temperatures are forecasted to have a large increase. There is no consensus on precipitation averages and streamflow trends in the future, with contradicting predictions. Precipitation extremes however are predicted to have a small increase in the future based on a majority consensus.

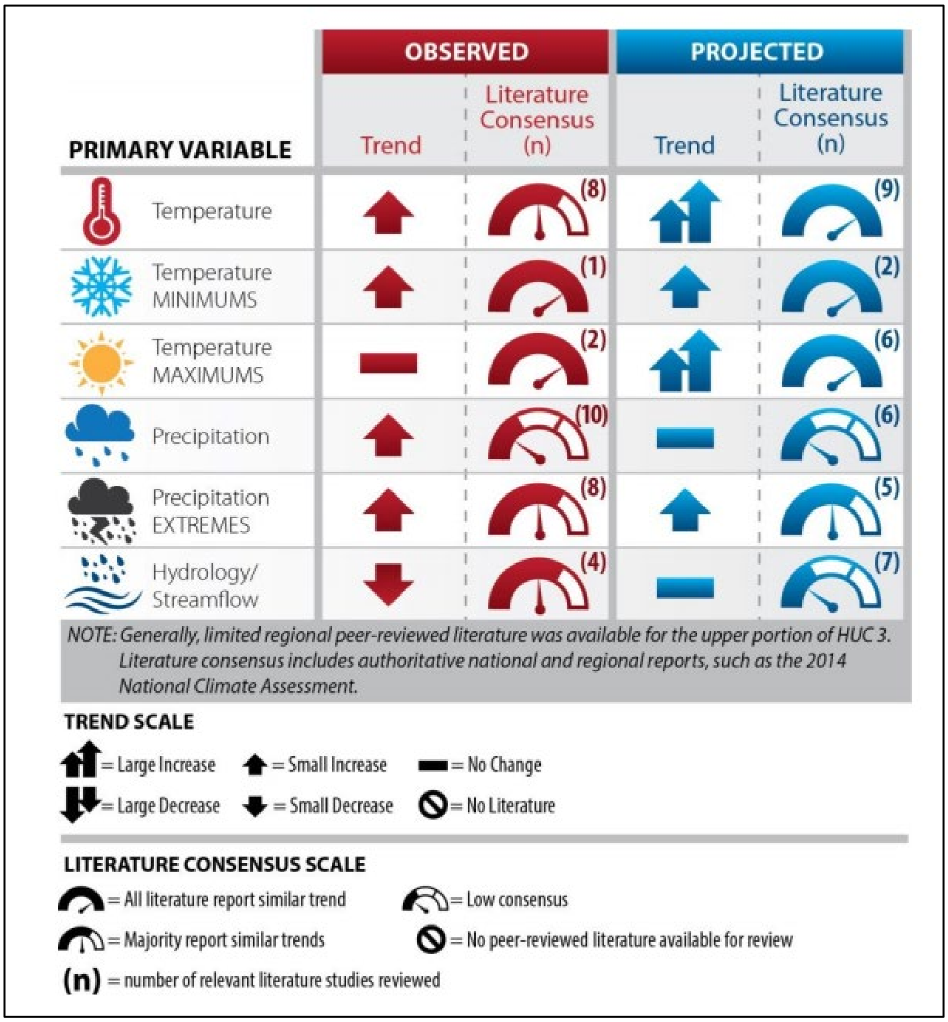

Figure 209. Summary Matrix of Observed and Project Climate Trends

The general consensus in the recent literature points toward mild increases in annual temperature in the South Atlantic-Gulf Region over the past century, particularly over the past 40 years. While much of the area is located within the so-called “warming hole” identified by various researchers (including Carter et al., 2014), recent studies have demonstrated significant warming for other parts of the area (particularly northern portions) since the 1970s. Annual precipitation totals have become more variable in recent years compared to earlier in the 20th century. Evidence has also been presented, but with limited consensus, of mildly increasing trends in the magnitude of annual and seasonal precipitation for parts of the study area. These results are seemingly contradicted by a number of studies that have shown decreasing trends in streamflow throughout the area, particularly since the 1970s. This paradox is discussed by Small et al. (2006), who attribute it largely to seasonal differences in the timing of the changes in precipitation vs. streamflow. The study authors evaluated watersheds that experienced minimal water withdrawals and/or transfers. Results presented here also suggest that increasing temperatures may also play a role in decreasing streamflows, despite the lack of corresponding precipitation decline.

There is strong consensus in the literature that air temperatures will increase in the study area, and throughout the country, over the next century. The studies reviewed here generally agree on an increase in mean annual air temperature of approximately 2 to 4 ºC by the latter half of the 21st century for the South Atlantic-Gulf Region. The largest increases are projected for the summer months. Reasonable consensus is also seen in the literature with respect to projected increases in extreme temperature events, including more frequent, longer, and more intense summer heat waves in the long-term future compared to the recent past. Projections of precipitation in the study area are less certain than those associated with air temperature. Results of the studies reviewed here are roughly evenly split with respect to projected increases vs. decreases in future annual precipitation. This is not unexpected as, according to the recently released NCA (Carter et al., 2014); the southeast region of the country (inclusive of the South Atlantic-Gulf Region) appears to be located in a “transition zone” between the projected wetter conditions to the north and dryer conditions to the west. There is, however, moderate consensus among the reviewed studies that future storm events in the region will be more intense and more frequent compared to the recent past. Similarly, clear consensus is lacking in the hydrologic projection literature. Projections generated by coupling GCMs with macro-scale hydrologic models in some cases indicate a reduction in future streamflows but in other cases indicate a potential increase in streamflows in the study region. Of the limited number of studies reviewed here, results are approximately evenly split between the two.

11.5 Observed Trends in Current Climate and Climate Change

11.5.1 Climate Hydrology Assessment Tool

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

A summary of the regression trends and their statistical significance is listed in Table 84 below. Individual graphical output for all gages and period of record data analyzed is shown in Figure 210 through Figure 226. The gage stations along the Neuse River near Falls, Clayton, Goldsboro, and Kinston were only analyzed for period after the Falls Dam was built and began operations. The Neuse River near Falls gage showed a statistically significant downward trend in observed peak annual flows but would be expected as the flow at this station is regulated by dam operations with one purpose being flood reduction. Little River tributary at Fairntosh also showed a statistically significant downward trend, however the results are highly driven by the observed peak flow in 1996. When that data point is removed the site no longer shows a statistically significant trend.

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

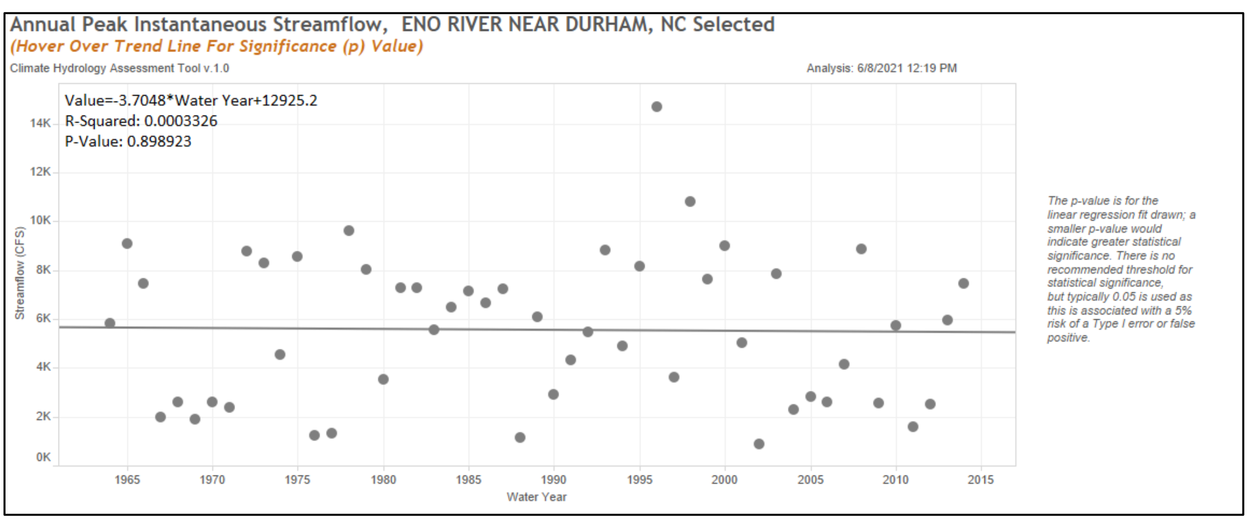

Figure 210. CHAT Results for Gage 02088070 Eno River near Durham, NC

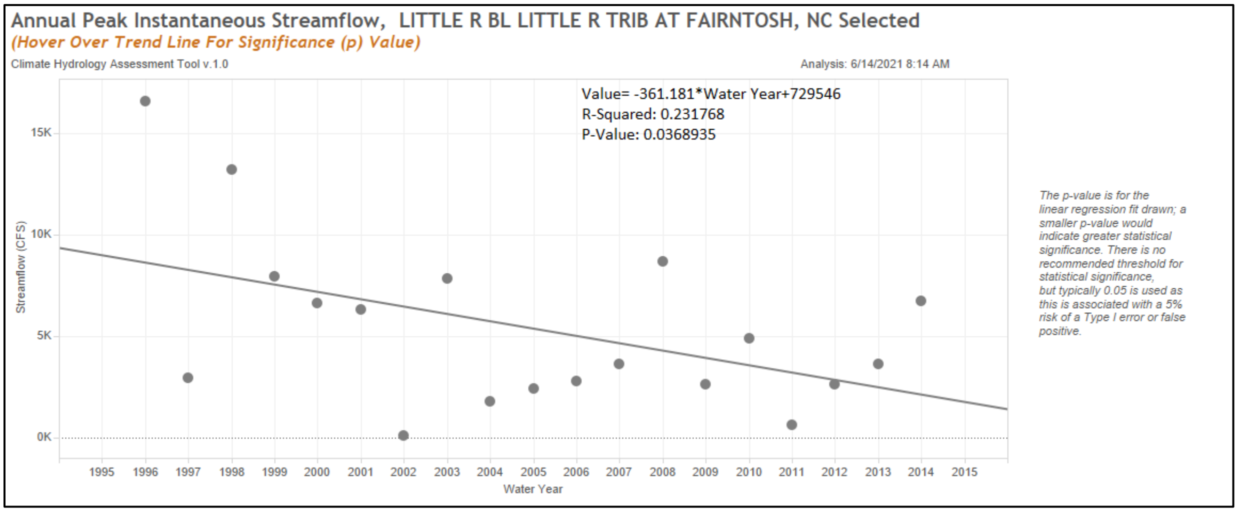

Figure 211. CHAT Results for Gage 0208524975 Little River Tributary near Fairntosh, NC

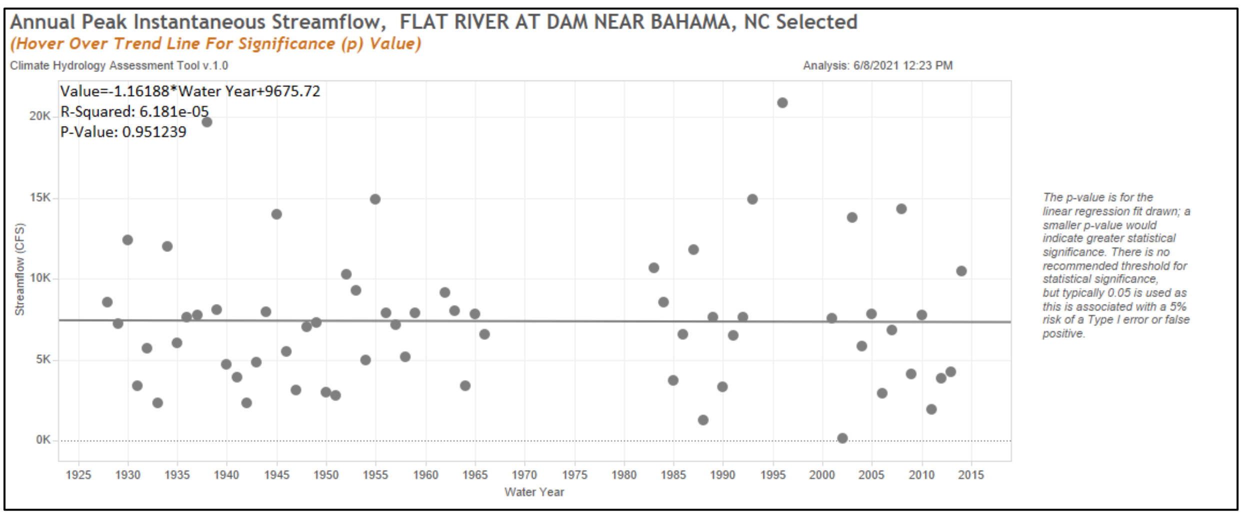

Figure 212. CHAT Results for Gage 02086500 Flat River at Dam near Bahama, NC

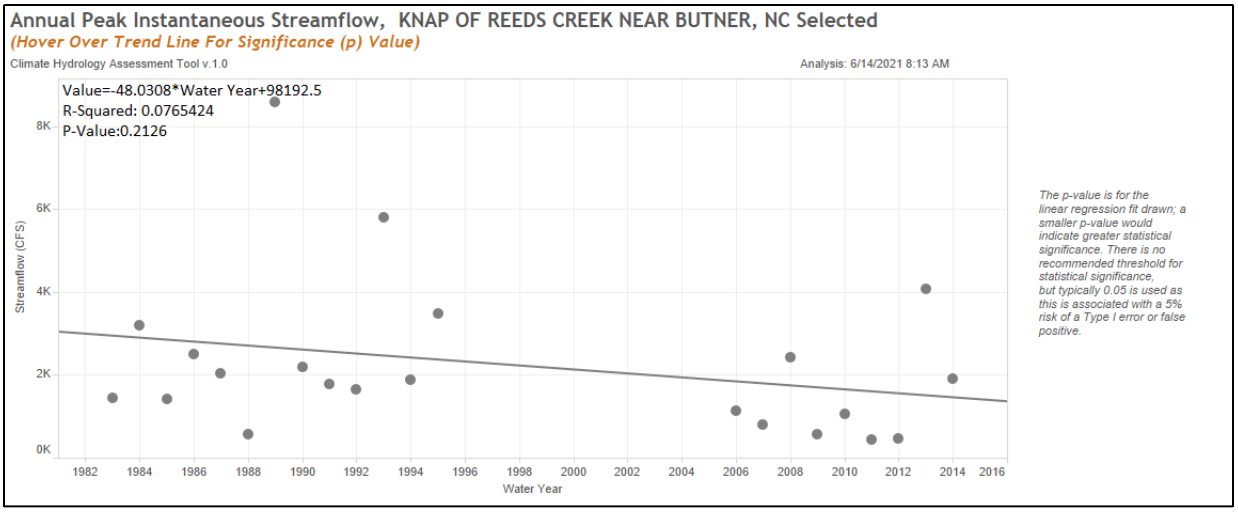

Figure 213. CHAT Results for Gage 02086624 Knap of Reeds Creek near Butner, NC

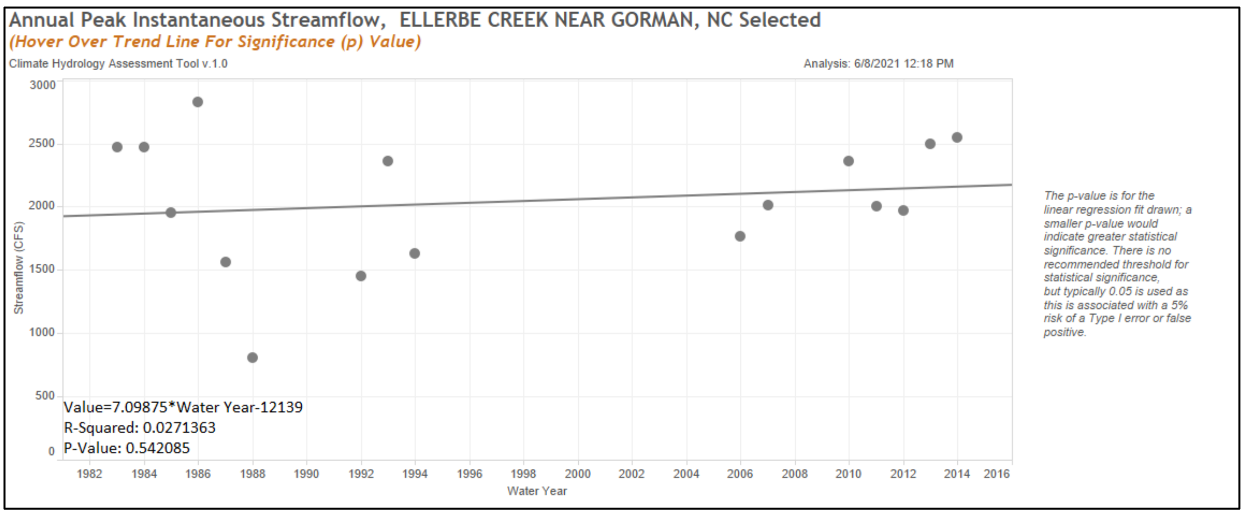

Figure 214. CHAT Results for Gage 02086849 Ellerbe Creek near Gorman, NC

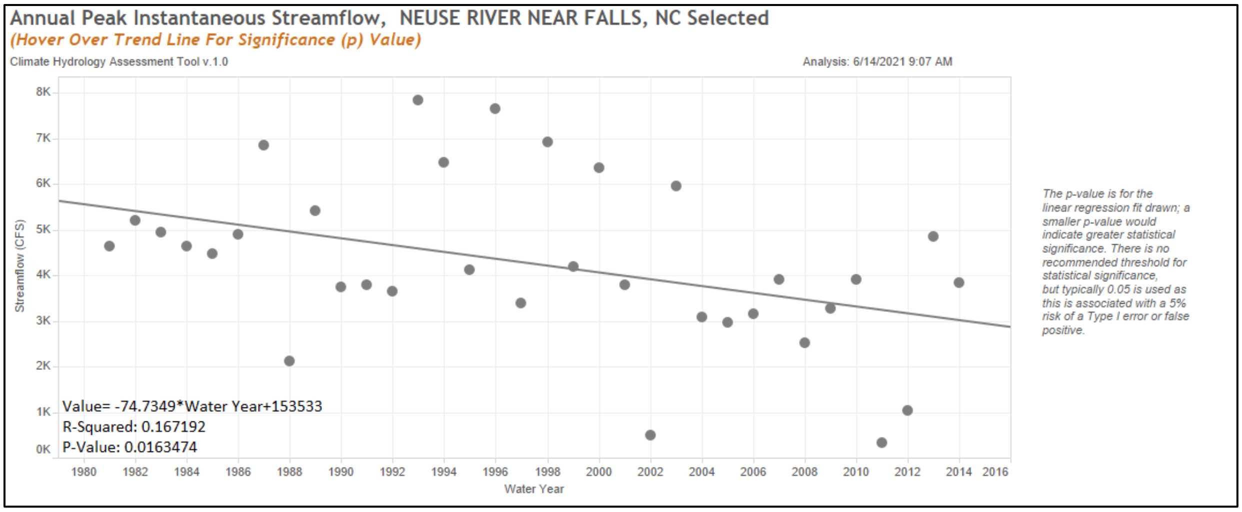

Figure 215. CHAT Results for Gage 02087183 Neuse River near Falls, NC

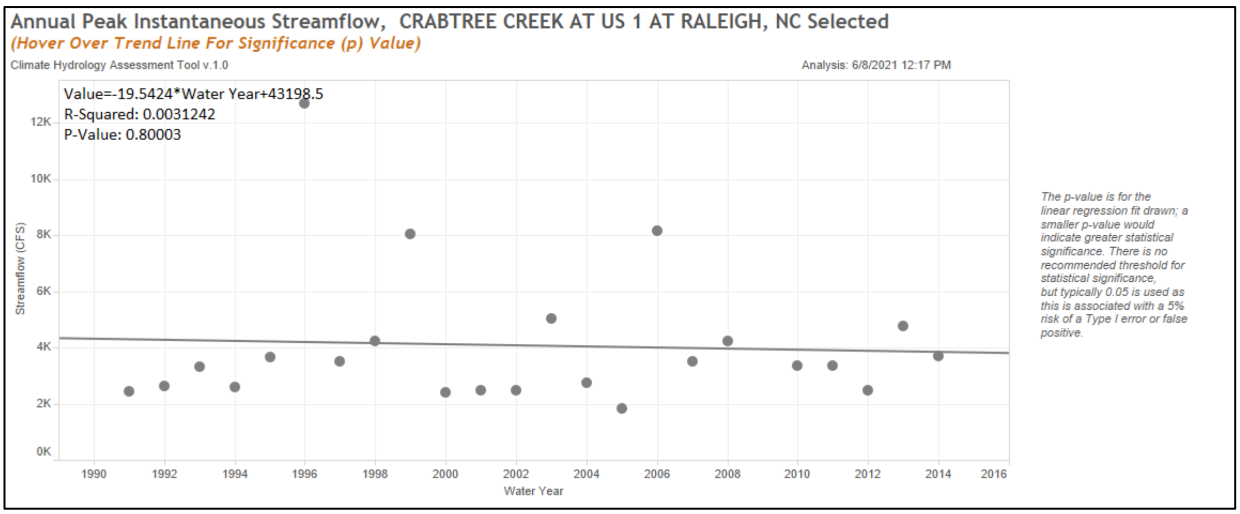

Figure 216. CHAT Results for Gage 02087324 Crabtree Creek at US 1 at Raleigh, NC

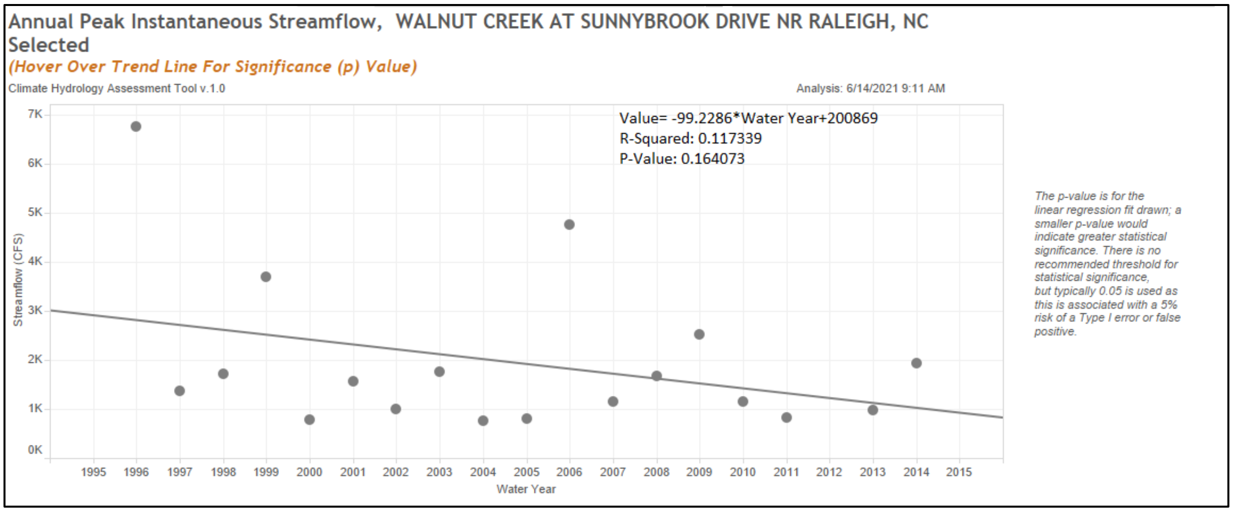

Figure 217. CHAT Results for Gage 02087359 Walnut Creek at Sunnybrook Drive near Raleigh, NC

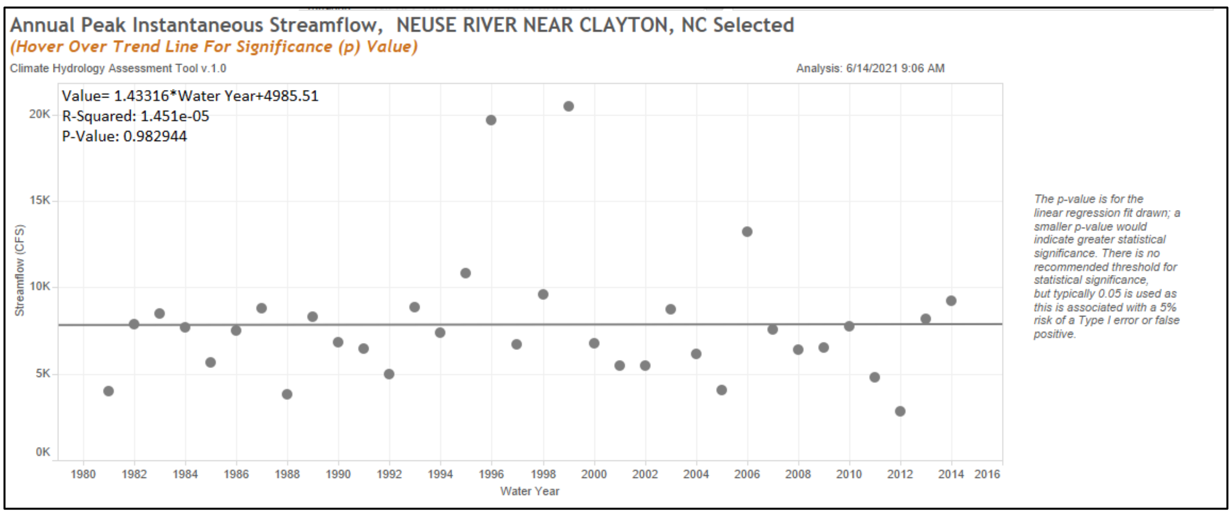

Figure 218. CHAT Results for Gage 02087580 Neuse River near Clayton, NC

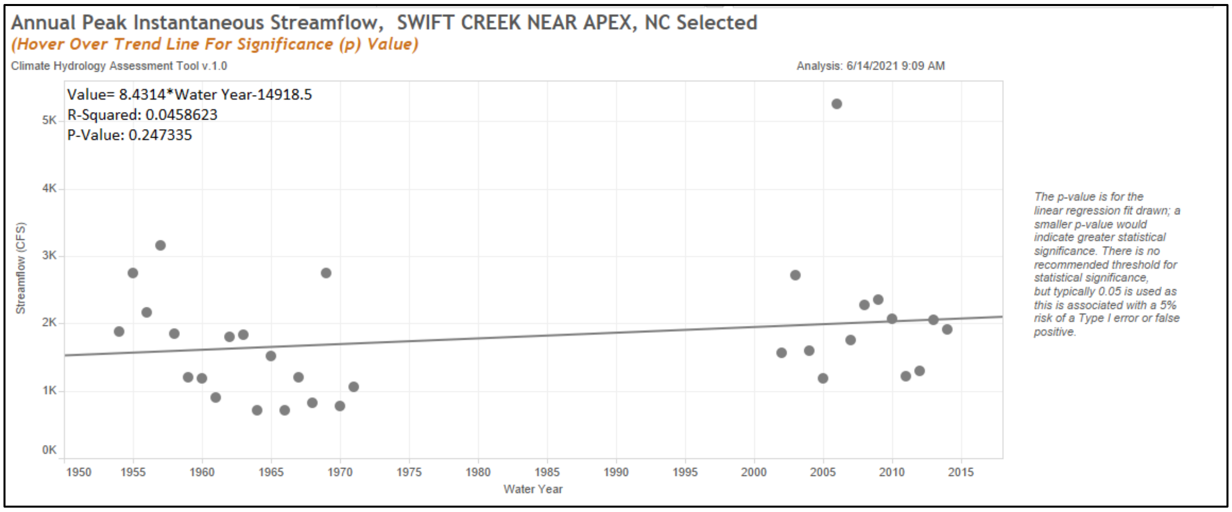

Figure 219. CHAT Results for Gage 02087580 Swift Creek near Apex, NC

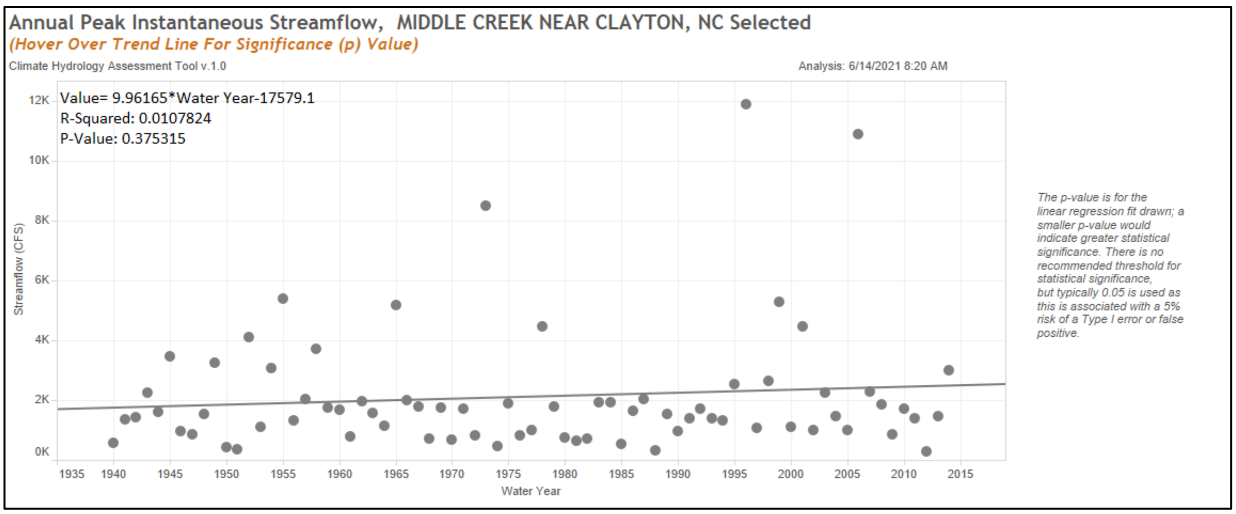

Figure 220. CHAT Results for Gage 02088000 Middle Creek near Clayton, NC

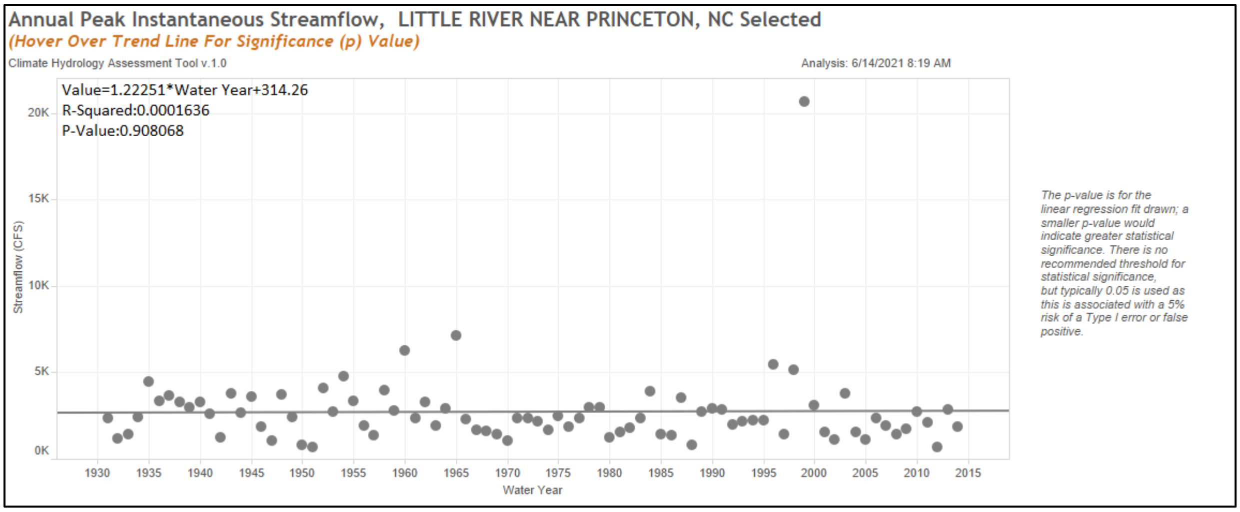

Figure 221. CHAT Results for Gage 02088500 Little River near Princeton, NC

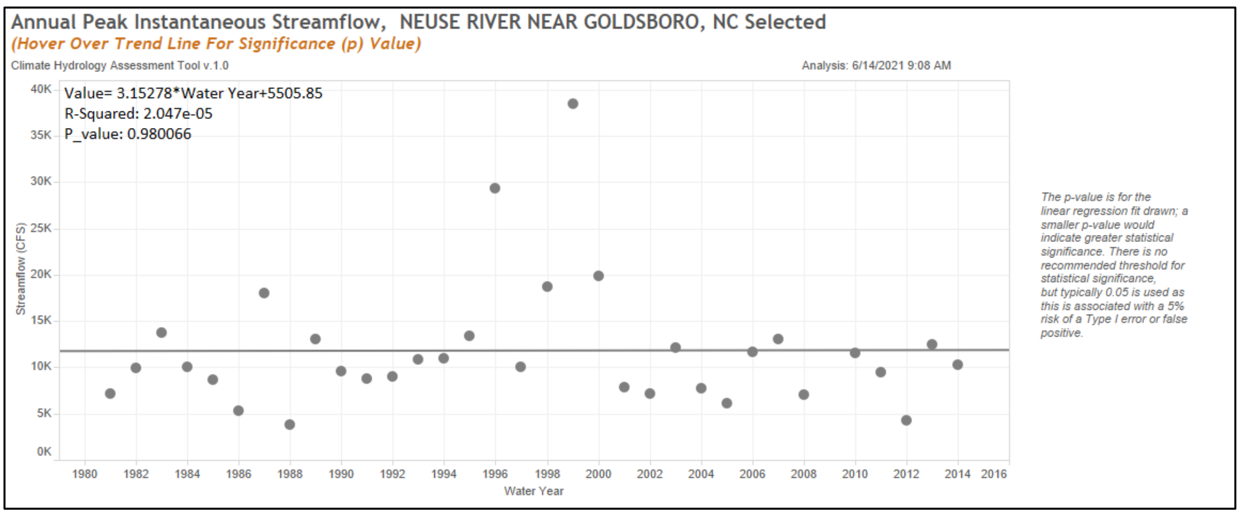

Figure 222. CHAT Results for Gage 02089000 Neuse River near Goldsboro, NC

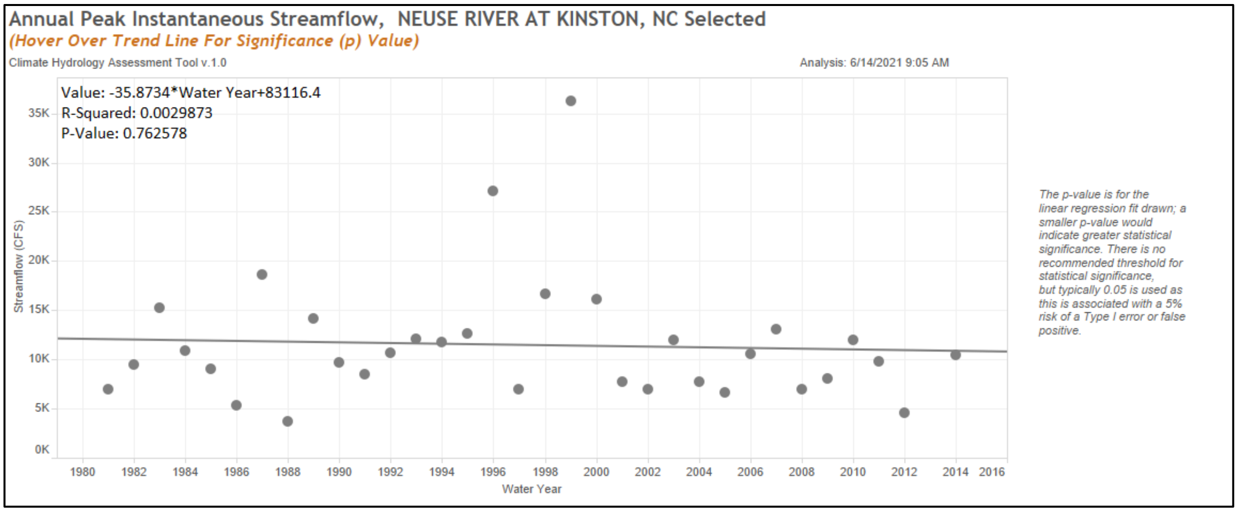

Figure 223. CHAT Results for Gage 02089500 Neuse River at Kinston, NC

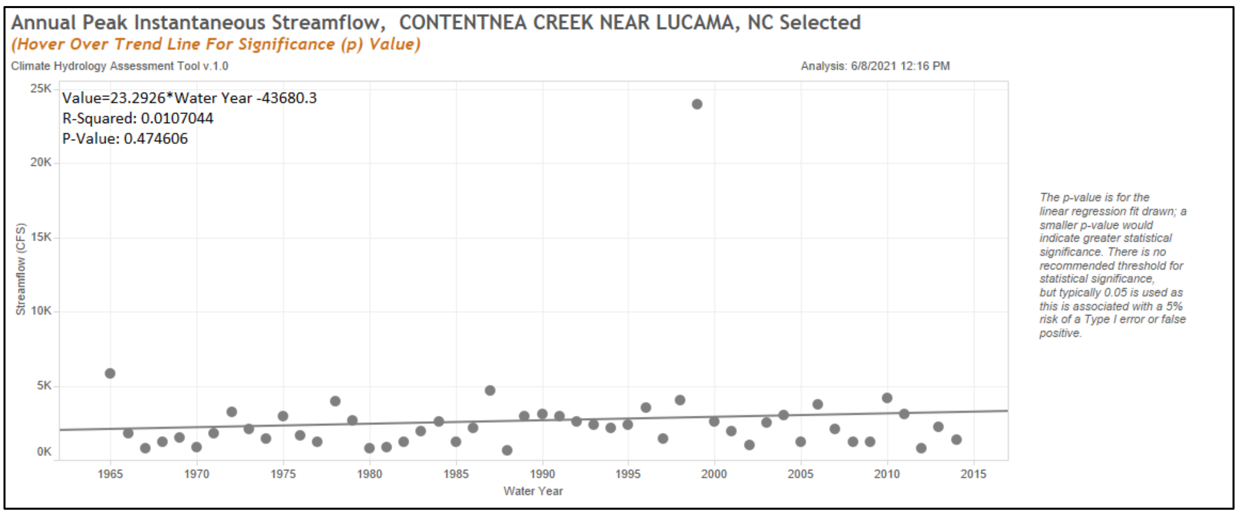

Figure 224. CHAT Results for Gage 02090380 Contentnea Creek near Lucama, NC

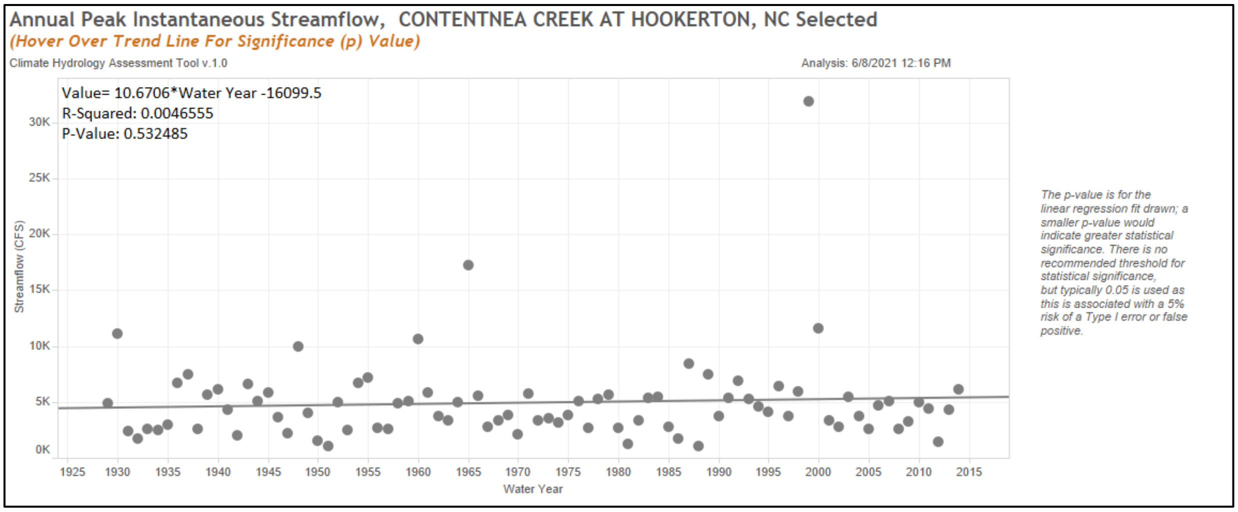

Figure 225. CHAT Results for Gage 02091500 Contentnea Creek at Hookerton, NC

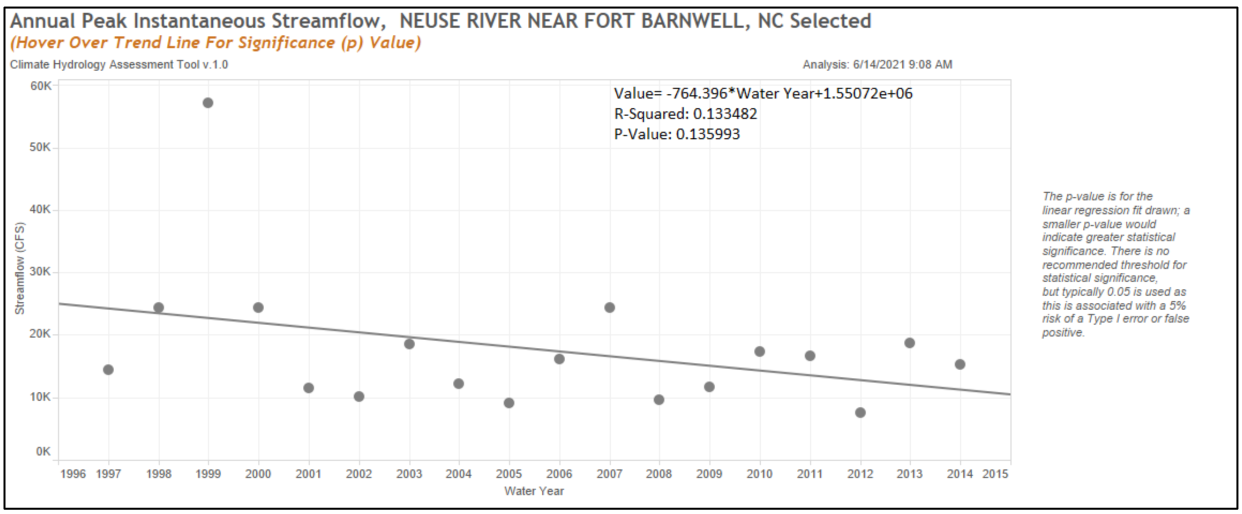

Figure 226. CHAT Results for Gage 02091814 Neuse River near Fort Barnwell, NC

Table 84. Summary of Observed Streamflow Trends in Annual Peak Streamflow using CHAT

Gage Number	Gage Name and Location	POR for CHAT	POR for NSD	POR Note	Regression Slope	P-Value	Trend Direction	Significance
02085070	Eno River Near Durham, NC	1985-2014	1964-2014		-3.7048	0.898923	Downwards	Not Significant
0208524975	Little River Tributary at Fairntosh, NC	1996-2014	NA		-361.181	0.0368935	Downwards	Significant
02086500	Flat River at Dam near Bahama, NC	1985-2014	1928-2014	Gap 1959-1962, 1966-1983, 1994-1995, 1997-2000	-1.16188	0.951239	Downwards	Not Significant
02086624	Knap of Reeds Creek near Butner, NC	1985-2014	NA	Gap 1996-2005	-48.0308	0.2126	Downwards	Not Significant
02086849	Ellerbe Creek near Gorman, NC	1985-2014	NA	Gap 1989-1991, 1994-2006, 2008-2009	7.09875	0.542085	Upwards	Not Significant
02087183	Neuse River near Falls. NC	1981-2014	1971-2019		-74.7349	0.0163474	Downwards	Significant
02087324	Crabtree Creek at US 1 at Raleigh, NC	1991-2014	NA		-19.5424	0.80003	Downwards	Not Significant

02087359	Walnut Creek at Sunnybrook Drive near Raleigh, NC	1996-2014	NA		-99.2286	0.164073	Downwards	Not Significant
02087500	Neuse River near Clayton, NC	1981-2014	1928-2014		1.43316	0.982944	Upwards	Not Significant
02087580	Swift Creek near Apex, NC	2002-2014	1954-2014	Gap 1972-2001	8.4314	0.247335	Upwards	Not Significant
02088000	Middle Creek near Clayton, NC	1985-2014	1940-2014		9.96165	0.375315	Upwards	Not Significant
02088500	Little River near Princeton, NC	1985-2014	1931-2014		1.22251	0.908068	Upwards	Not Significant
02089000	Neuse River near Goldsboro, NC	1981-2014	1930-2014	NSD gap 2009	3.15278	0.980066	Upwards	Not Significant
02089500	Neuse River at Kinston, NC	1981-2014	1928-2012		-35.8734	0.762578	Downwards	Not Significant
02090380	Contentnea Creek near Lucama, NC	1965-2014	1965-2014		23.2926	0.474606	Upwards	Not Significant
02091500	Contentnea Creek at Hookerton, NC	1929-2014	1929-2014		10.6706	0.532485	Upwards	Not Significant
02091814	Neuse River near Fort Barnwell, NC	1997-2014	NA		-764.396	0.135993	Downwards	Not Significant

11.5.2 Nonstationarity Detection Tool

The USACE Nonstationarity Detection (NSD) Tool was used to assess whether the assumption of stationarity, which is the assumption that the statistical characteristics of a time-series dataset are constant over the period of record, is valid for a given hydrologic time-series dataset. Nonstationarities are detected through the use of 12 different statistical tests which examine how the statistical characteristics of the dataset change with time (Engineering Technical Letter (ETL) 1100-2-3, Guidance for Detection of Nonstationarities in Annual Maximum Discharges; Nonstationarity Detection Tool User Manual, version 1.2). Abbreviations of the 12 statistical tests are listed in Table 85 below.

Table 85. NSD Statistical Test Abbreviations

Nonstationarity Detection Method Abbreviation	Statistical Test Name
CVM	Cramer-Von-Mises (CPM)
KS	Kolmogorov-Smirnov (CPM)
LP	LePage (CPM)
END	Energy Divisive Method
LW	Lombard Wilcoxon
PT	Pettitt
MW	Mann-Whitney (CPM)
BAY	Bayesian
LM	Lombard Mood
MD	Mood (CPM)
SLW	Smooth Lombard Wilcoxon
SLM	Smooth Lombard Mood

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

A summary of the NSD results can be found in Table 86 below. Two stream gages produced nonstationarities. The gage at 02087183 Neuse River near Falls, NC produced nonstationarity consensus in 2000 with the Cramer-Von-Mises, LePage, Pettitt, and Man-Whitney methods. The CVM and KS methods detect changes in the underlying distribution, while the PT and WM methods detect changes in the mean. This nonstationarity is strong as it is detected by multiple methods and has a change in magnitude of 1,200 cfs, a nearly 25% decrease of the instantaneous peak streamflow. The nonstationarity detected is within the timeframe of the statistically significant (p=0.03) downward trend detected by the CHAT. A monotonic trend analysis on the entire POR (1971-2020) detected a statistically significant decreasing slope of 95 cfs using the t-Test (p=0.0002), the Mann-Kendall Test (p=0.001) and the Spearman Rank-Order Test (p=0.001). Analyzing only the period before the non-stationarity (1971-1999) detected a statistically significant decrease of 159 cfs in the slope using the t-Test (p=0.011), the Mann-Kendall Test (p=0.012), and the Spearman Rank-Order Test (p=0.012). Analyzing only the period after the nonstationarity (2000-2020) detected no statistically significant trend in the slope. This gage is directly downstream of Falls Lake Dam which is a USACE operated dam. Beginning in 2000 the guide curve, top of conservation pool, and controlled flood pool elevations were changed. In addition, after public held meetings in the late 1990’s flood control releases considerations were changed, reducing public complaints so it is not unexpected to detect a nonstationarity of the mean and underlying distribution during this time frame.

The second gage the detected a nonstationarity is downstream from the Neuse River at Falls gage, 02087500 Neuse River near Clayton, NC. A consensus was detected in 1966 using the Cramer-Von Mises, Kolmogorov-Smirnov, LePage, Pettitt, and Mann-Whitney methods. The CVM, KS, and LP methods detect change in the underlying distribution while the PT and MW detect changes in the mean. The nonstationarity detected is considered strong due to its robustness and change in magnitude of 2,500 cfs or 23% of the instantaneous peak streamflow, however there was no coincidental trend detected using the CHAT. Falls Lake Dam began construction in 1978 and was completed in 1980 with the lake reaching its permanent impoundment level in 1983. If the analysis of Neuse River Near Clayton is restricted to the period after Falls Lake Dam began normal operations no nonstationarities are detected. A monotonic trend analysis on the entire POR (1928-2020) detected a statistically significant decrease in the trend using the t-Test (p=0.008), the Mann-Kendall Test (p=0.0006) and the Spearman Rank-Order Test (p=0.0006). The traditional slope method calculated a negative trend of 41 cfs. Analyzing only the period before the nonstationarity (1928-1966) also showed a statistically significant trend, but only by the t-Test (p-value=0.038). The traditional slope method calculated a negative slope of 111 cfs. Analyzing only the period after the nonstationary (1967-2020) did not show any statistically significant trend.

All other gages (Figure 227 through Figure 238) either did not produce nonstationarities, did not have enough data to perform an analysis or the data that was found on USGS was not recent enough to be feasible for the analysis.

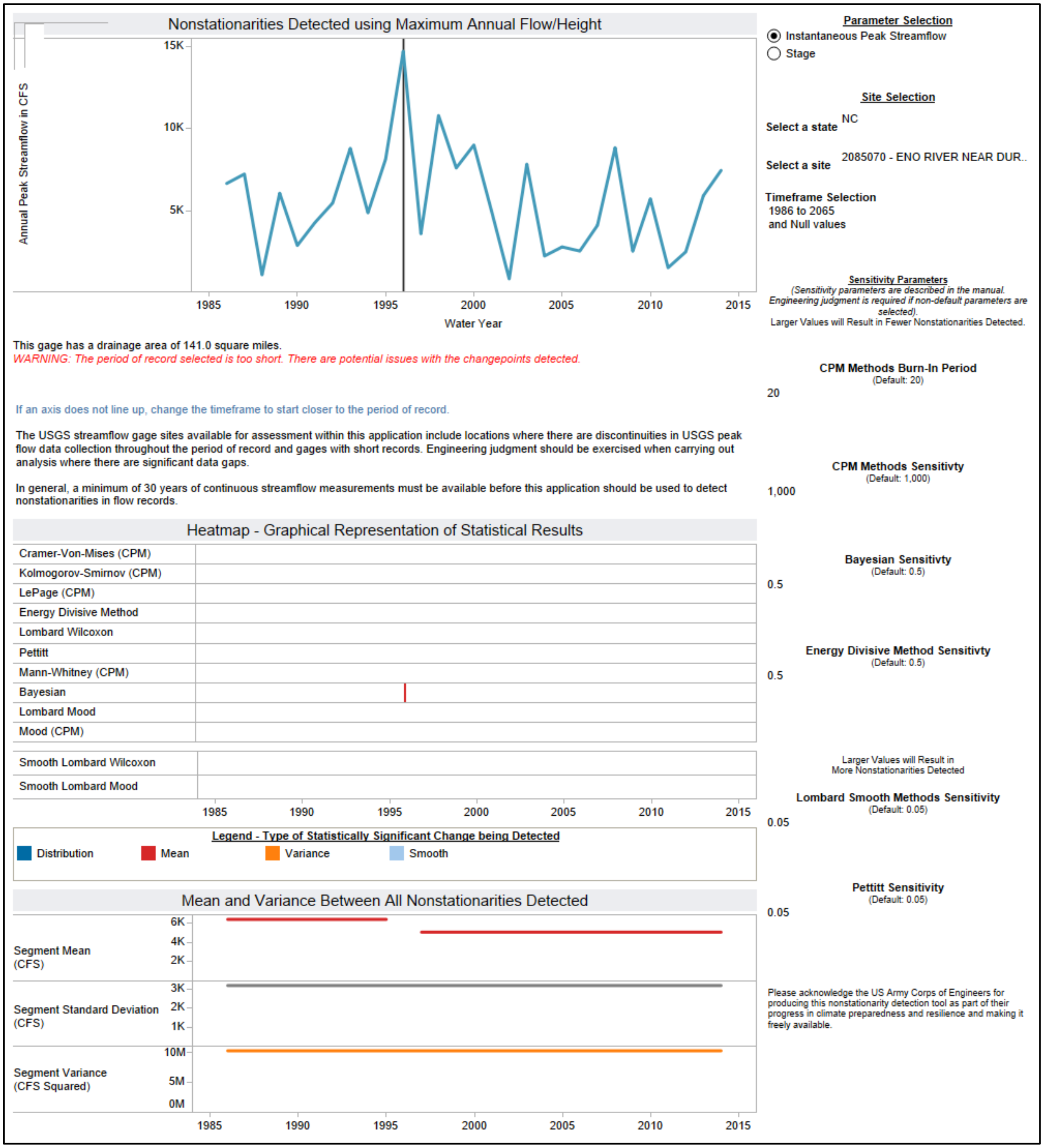

Figure 227. Nonstationarity Detection Results for Gage 02088070 Eno River near Durham, NC

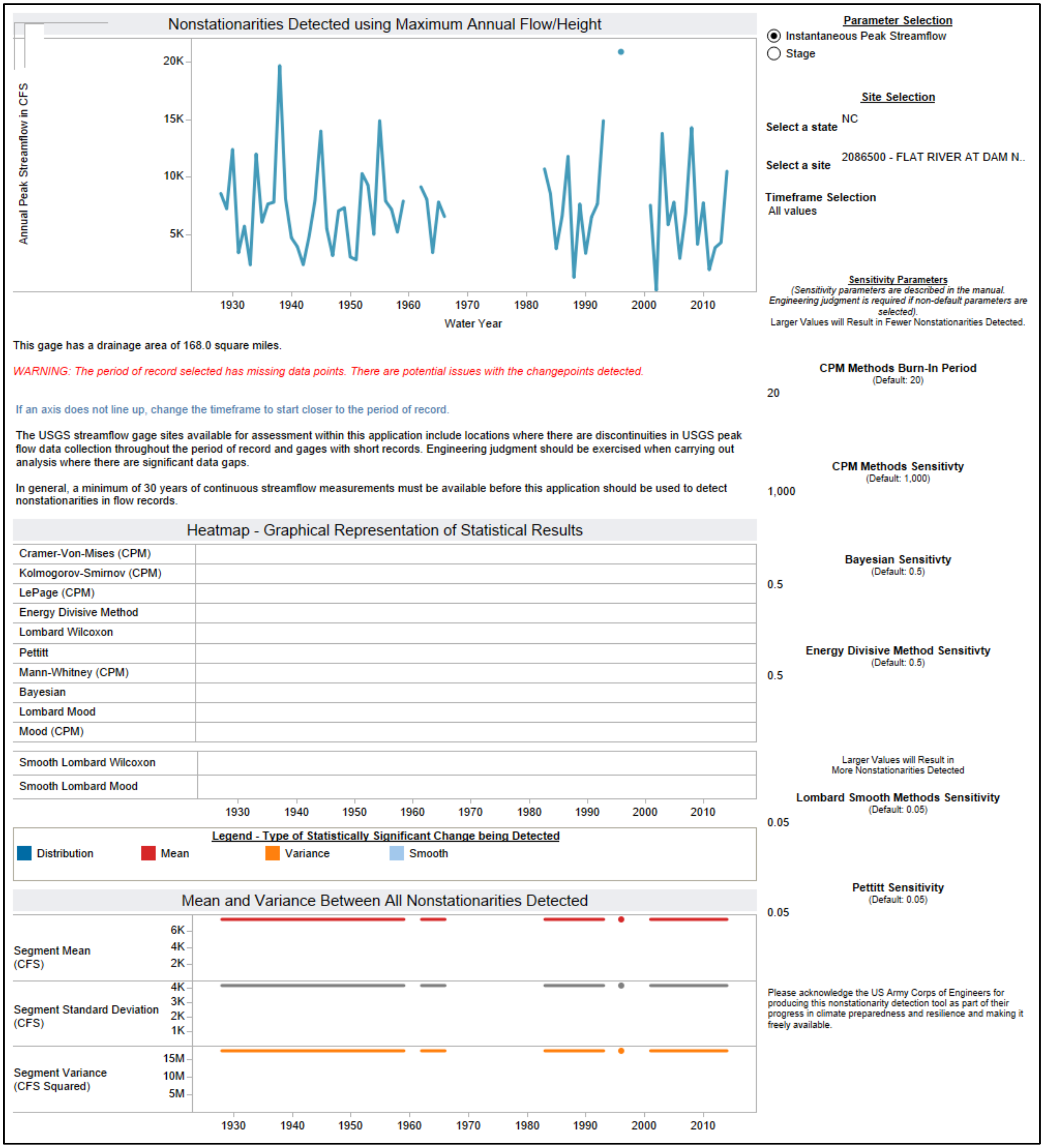

Figure 228. Nonstationarity Detection Results for Gage 02086500 Flat River at Dam at Bahama, NC

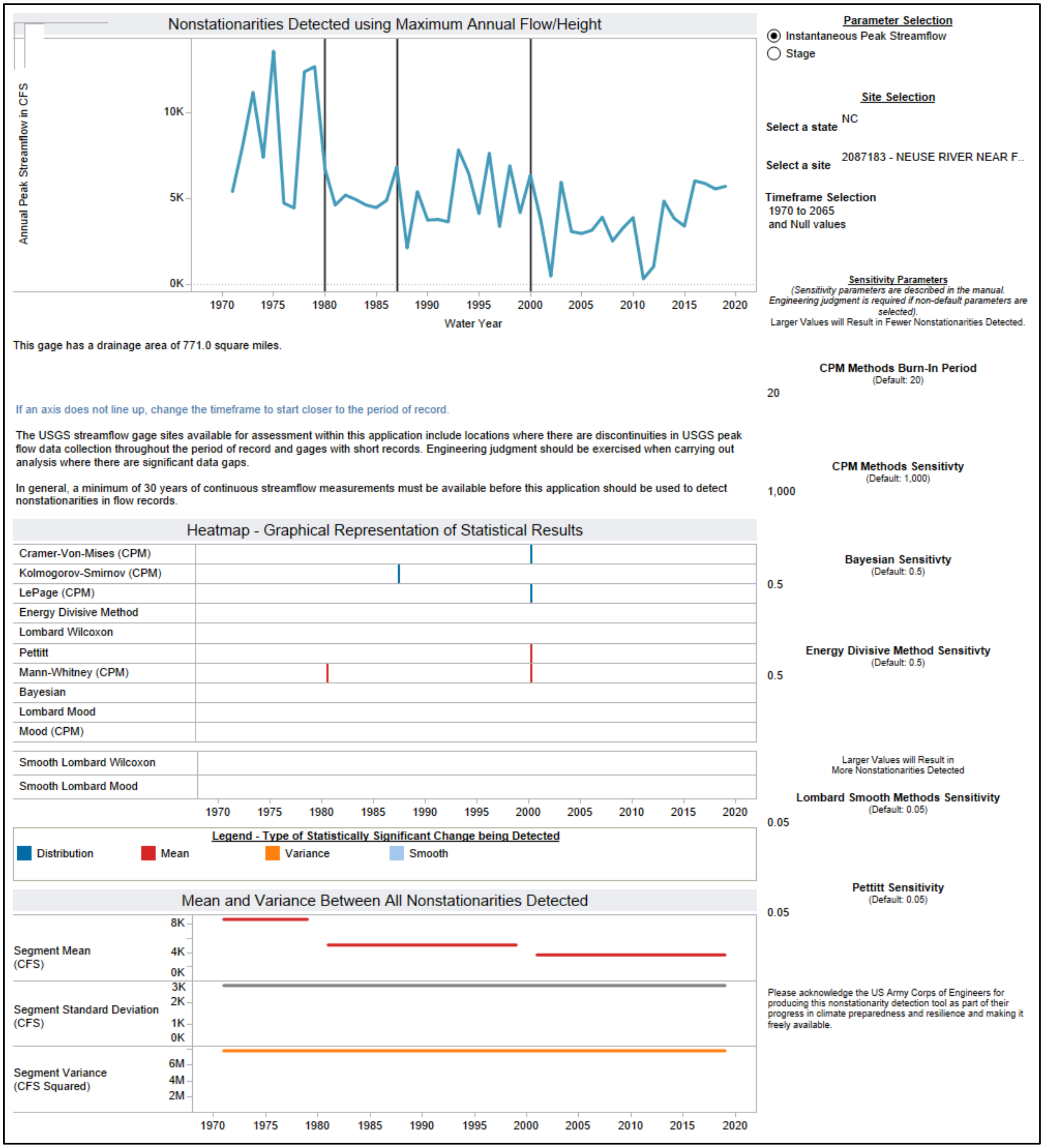

Figure 229. Nonstationarity Detection Results for Gage 02087183 Neuse River near Falls, NC

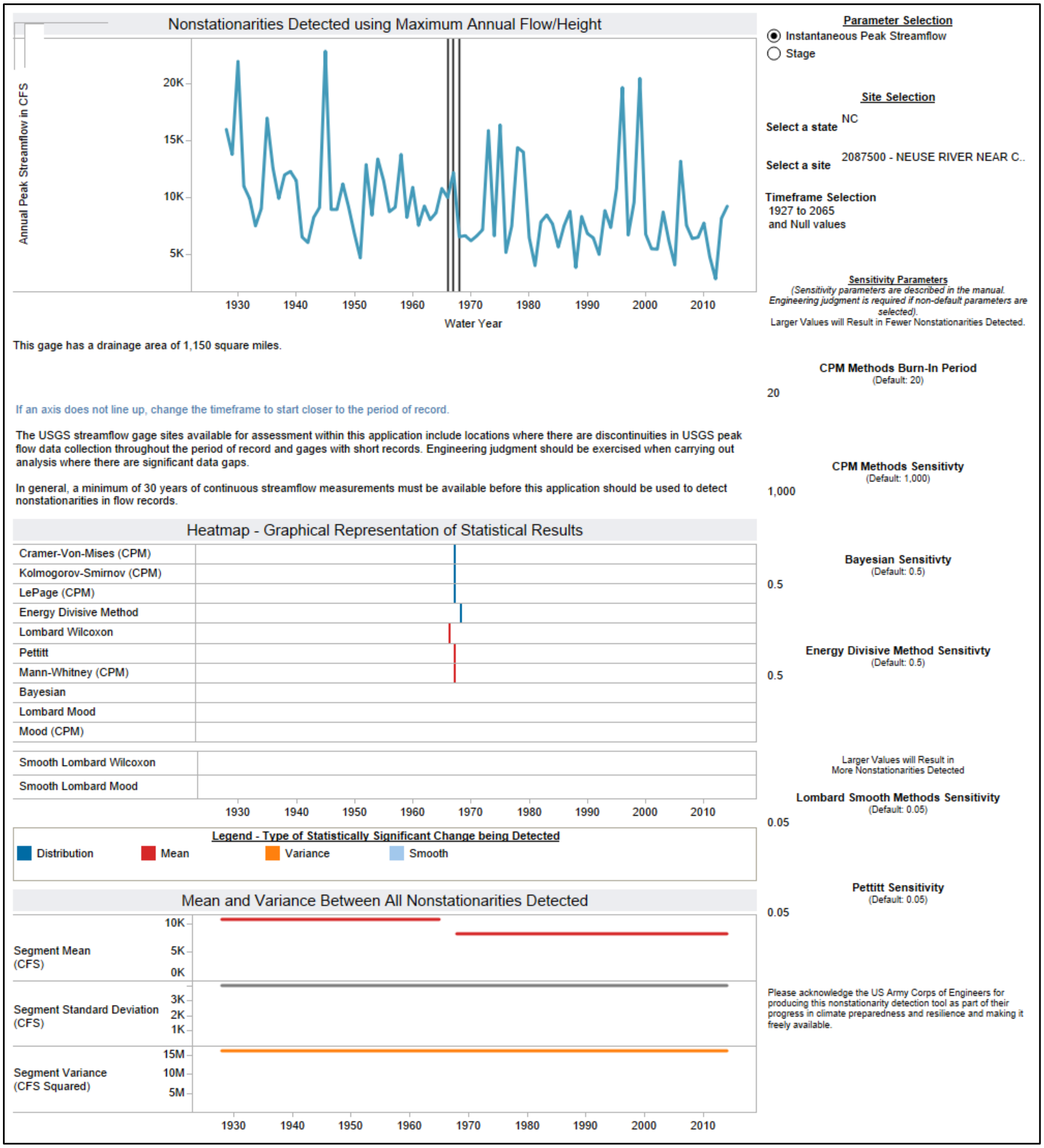

Figure 230. Nonstationarity Detection Results for Gage 02087500 Neuse River near Clayton, NC

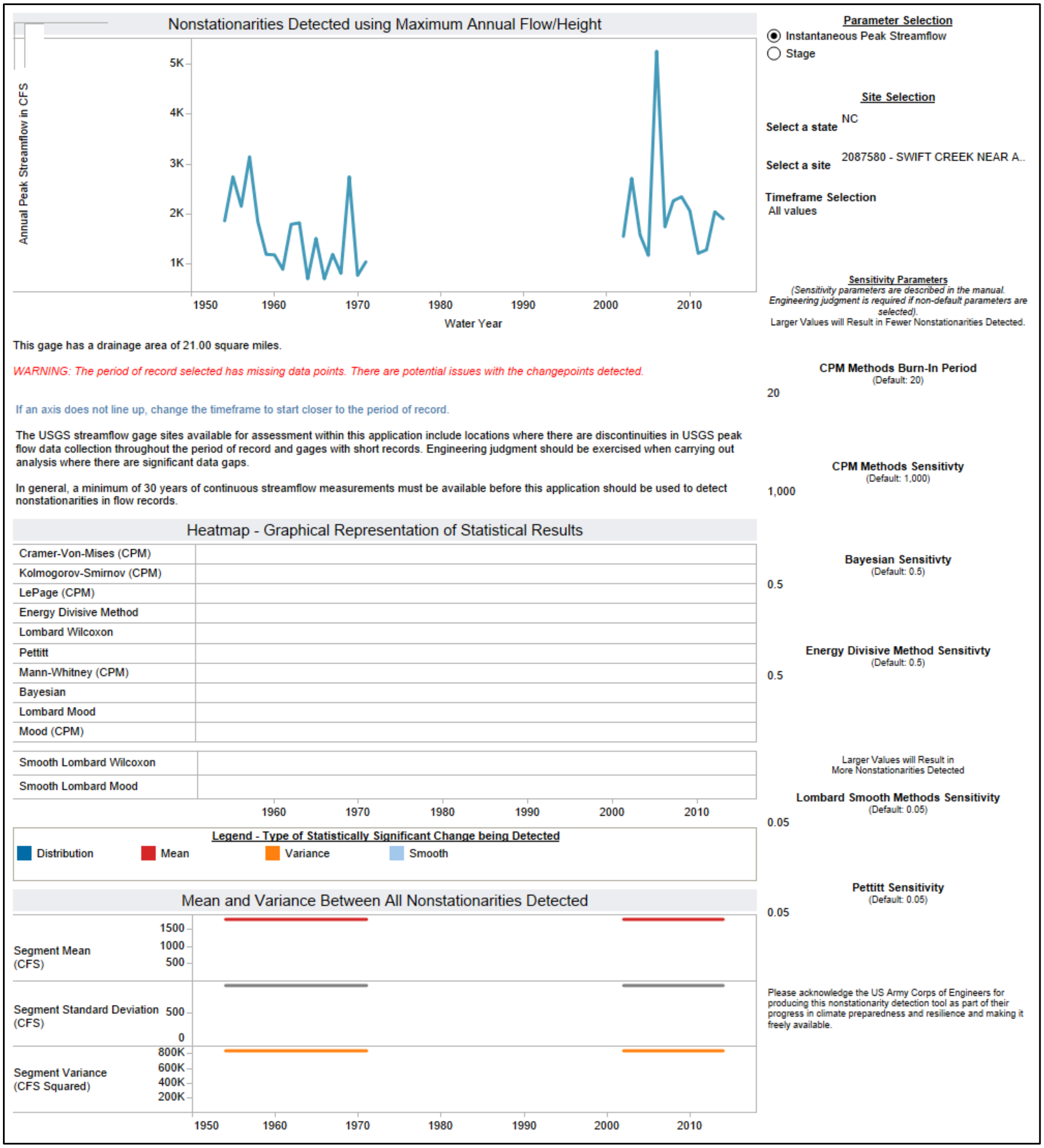

Figure 231. Nonstationarity Detection Results for Gage 02087580 Swift Creek near Apex, NC

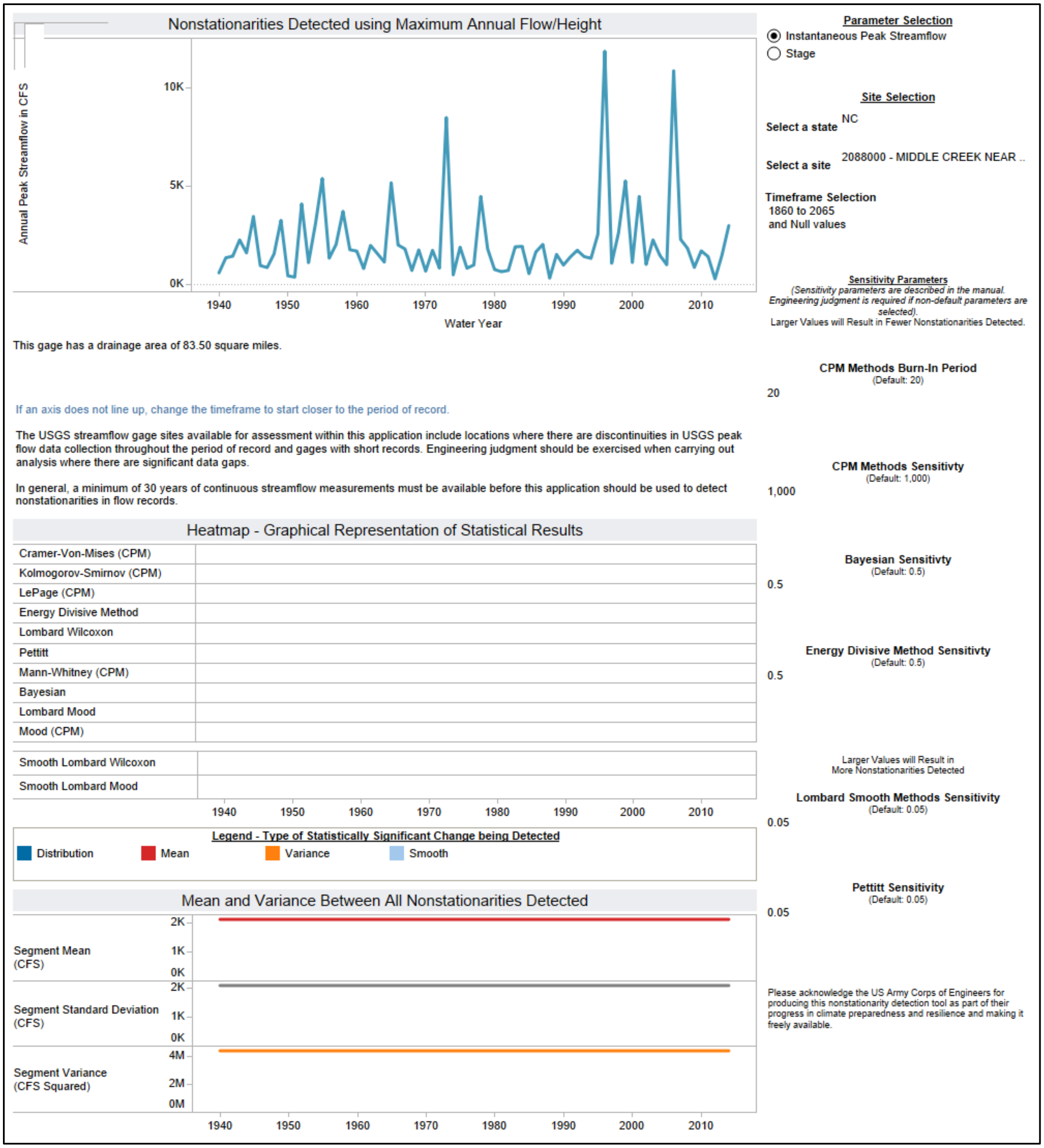

Figure 232. Nonstationarity Detection Results for Gage 02088000 Middle Creek near Clayton, NC

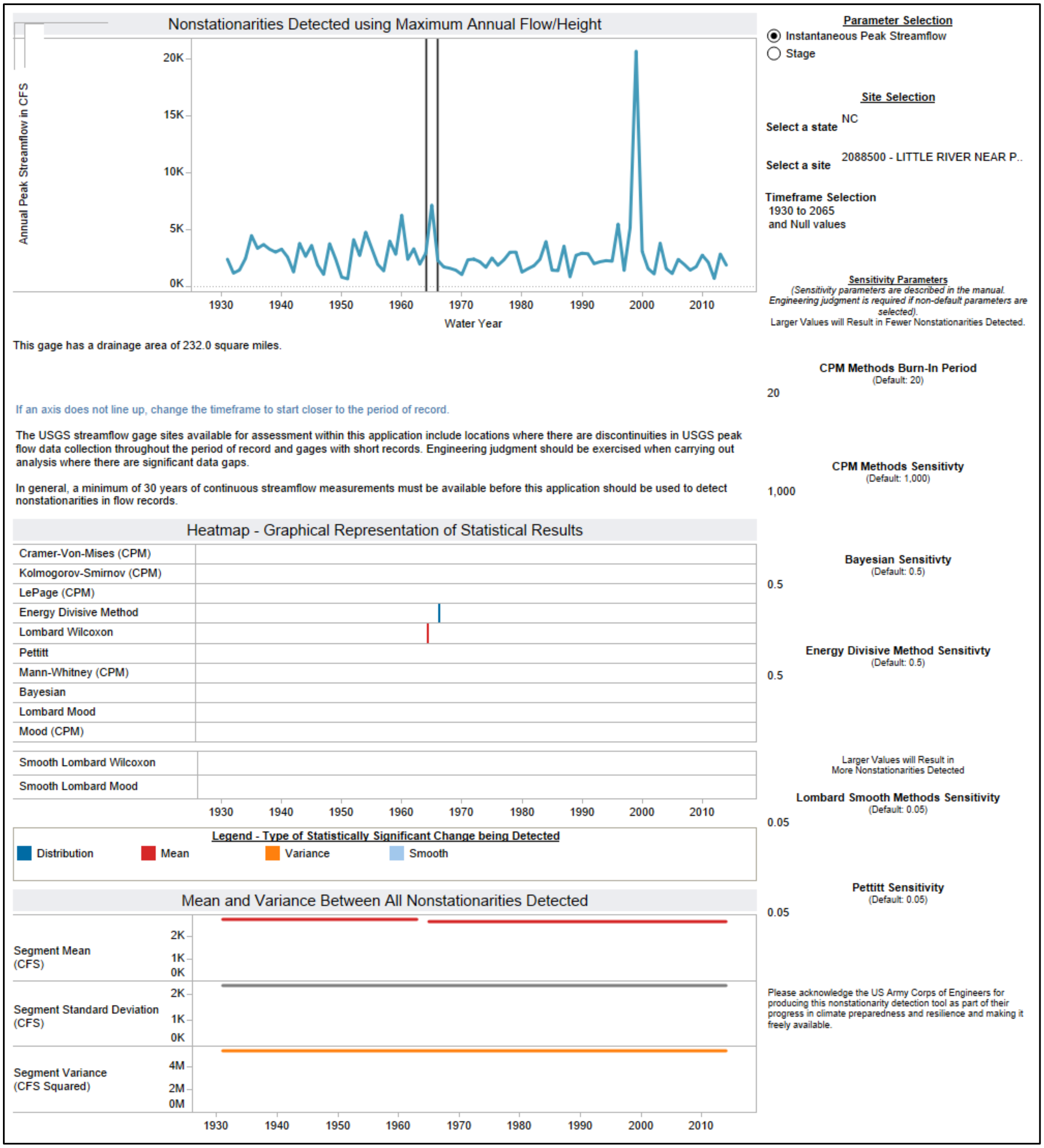

Figure 233. Nonstationarity Detection Results for Gage 02088500 Little River near Princeton, NC

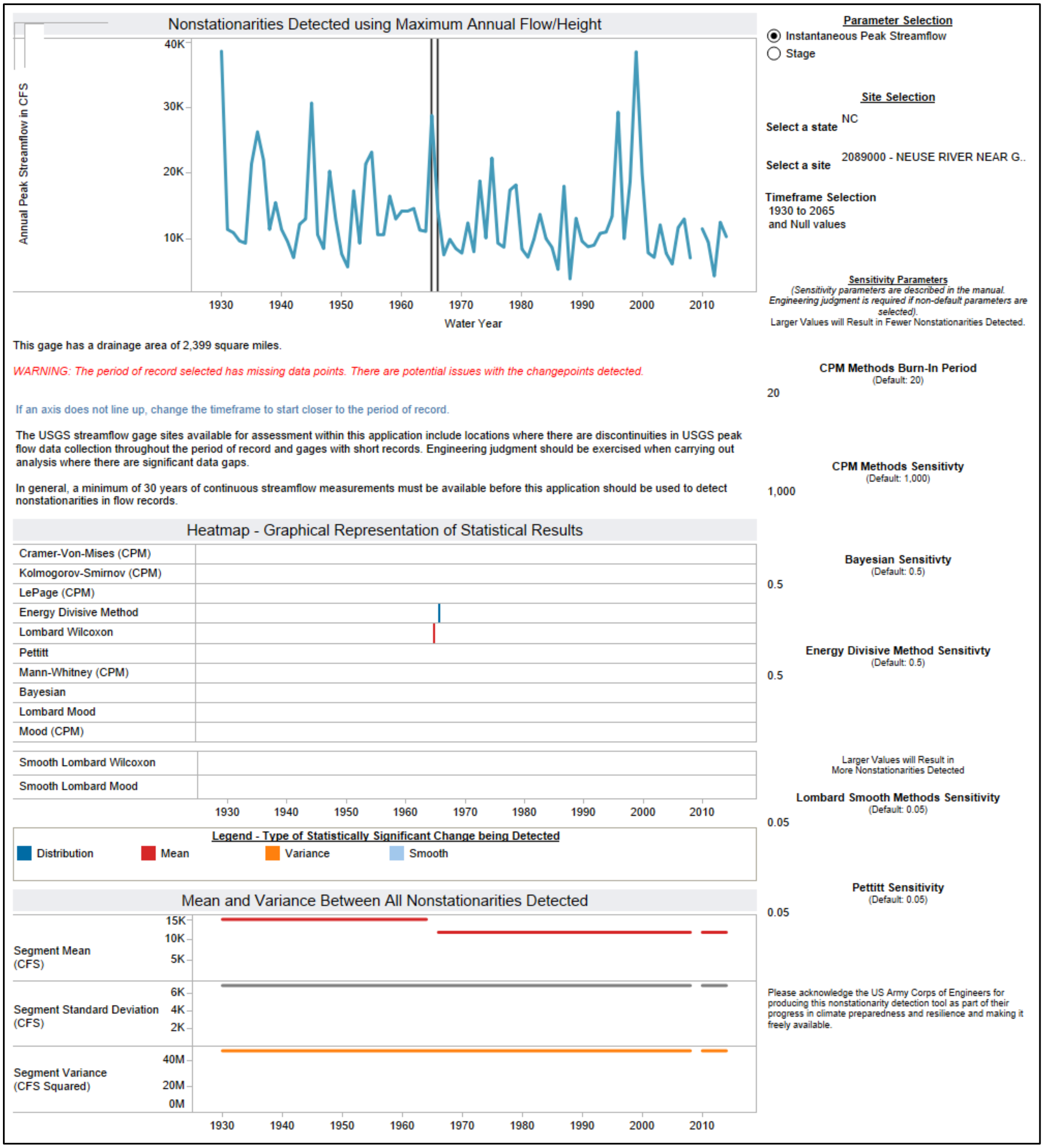

Figure 234. Nonstationarity Detection Results for Gage 02089000 Neuse River near Goldsboro, NC

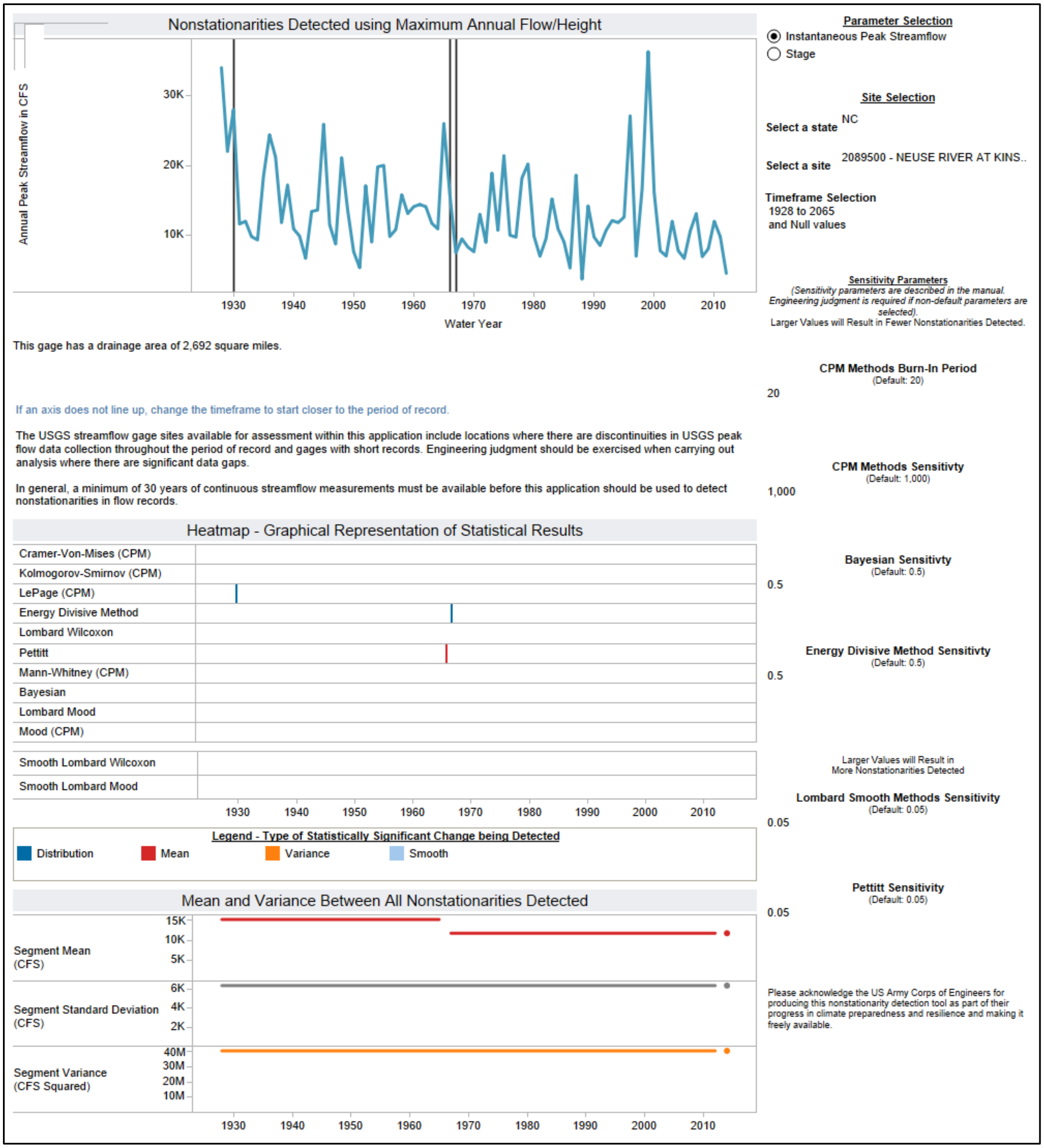

Figure 235. Nonstationarity Detection Results for Gage 02089500 Neuse River at Kinston, NC

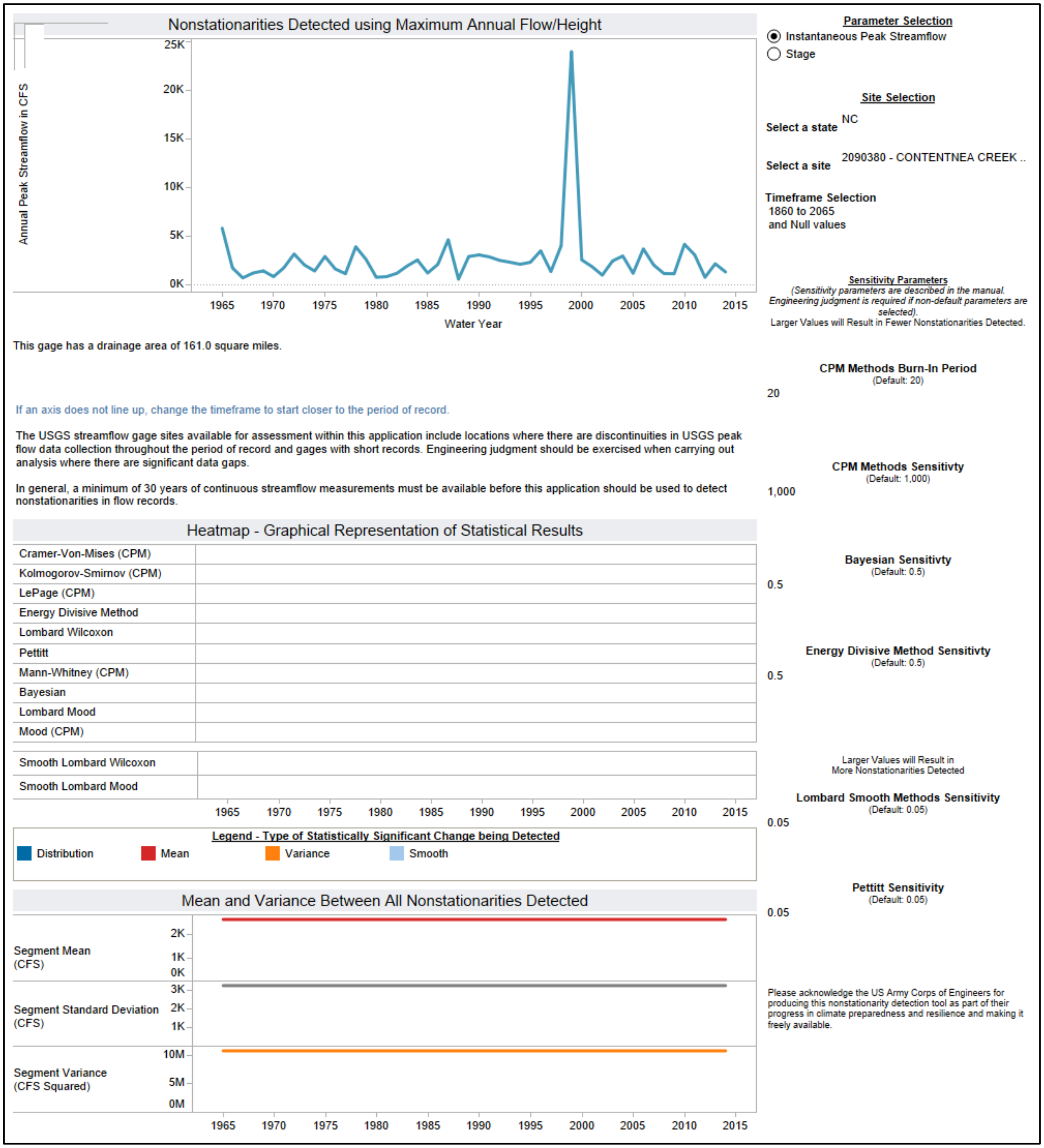

Figure 236. Nonstationarity Detection Results for Gage 02090380 Contentnea Creek near Lucama, NC

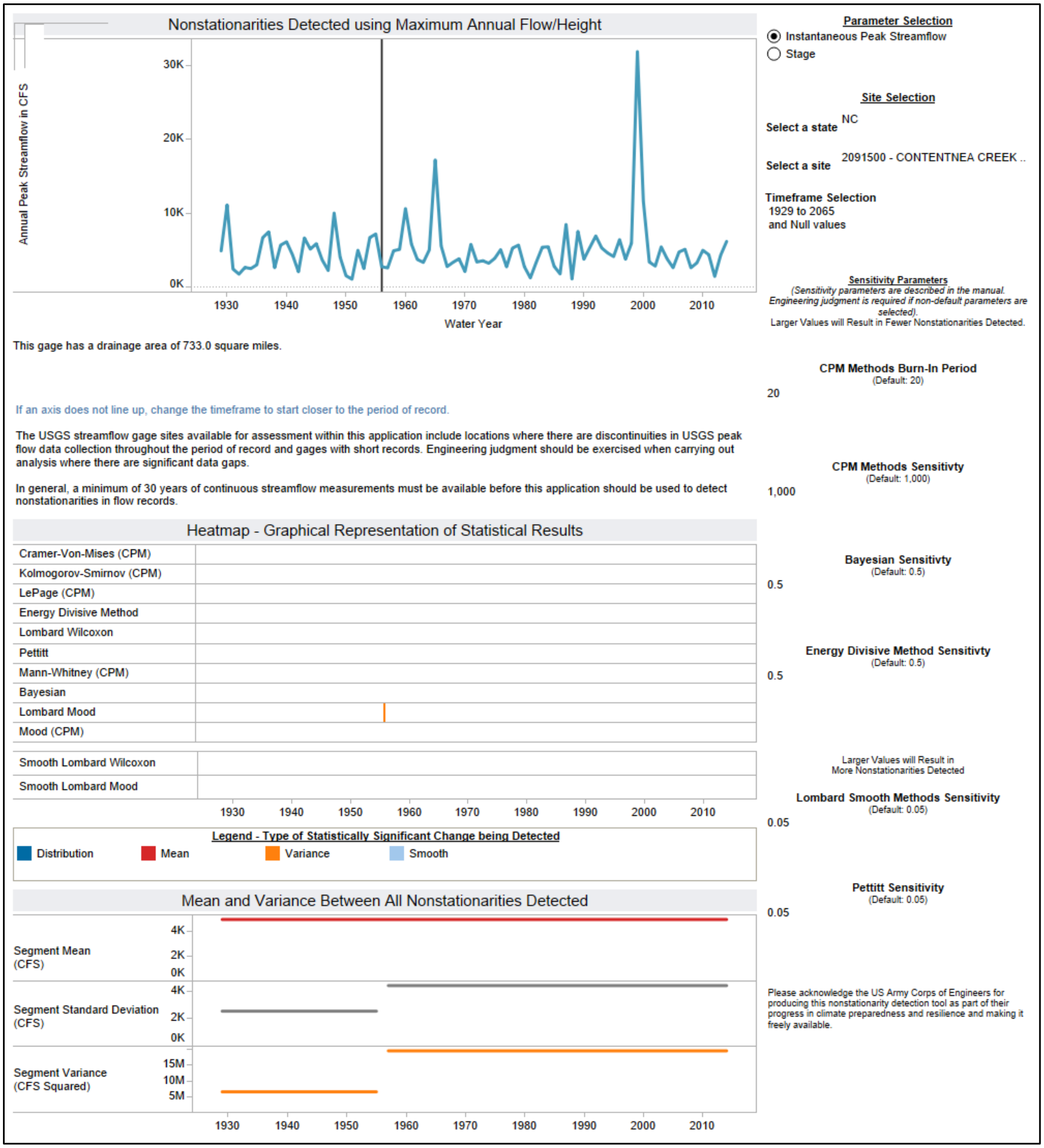

Figure 237. Nonstationarity Detection Results for Gage 02091500 Contentnea Creek at Hookerton, NC

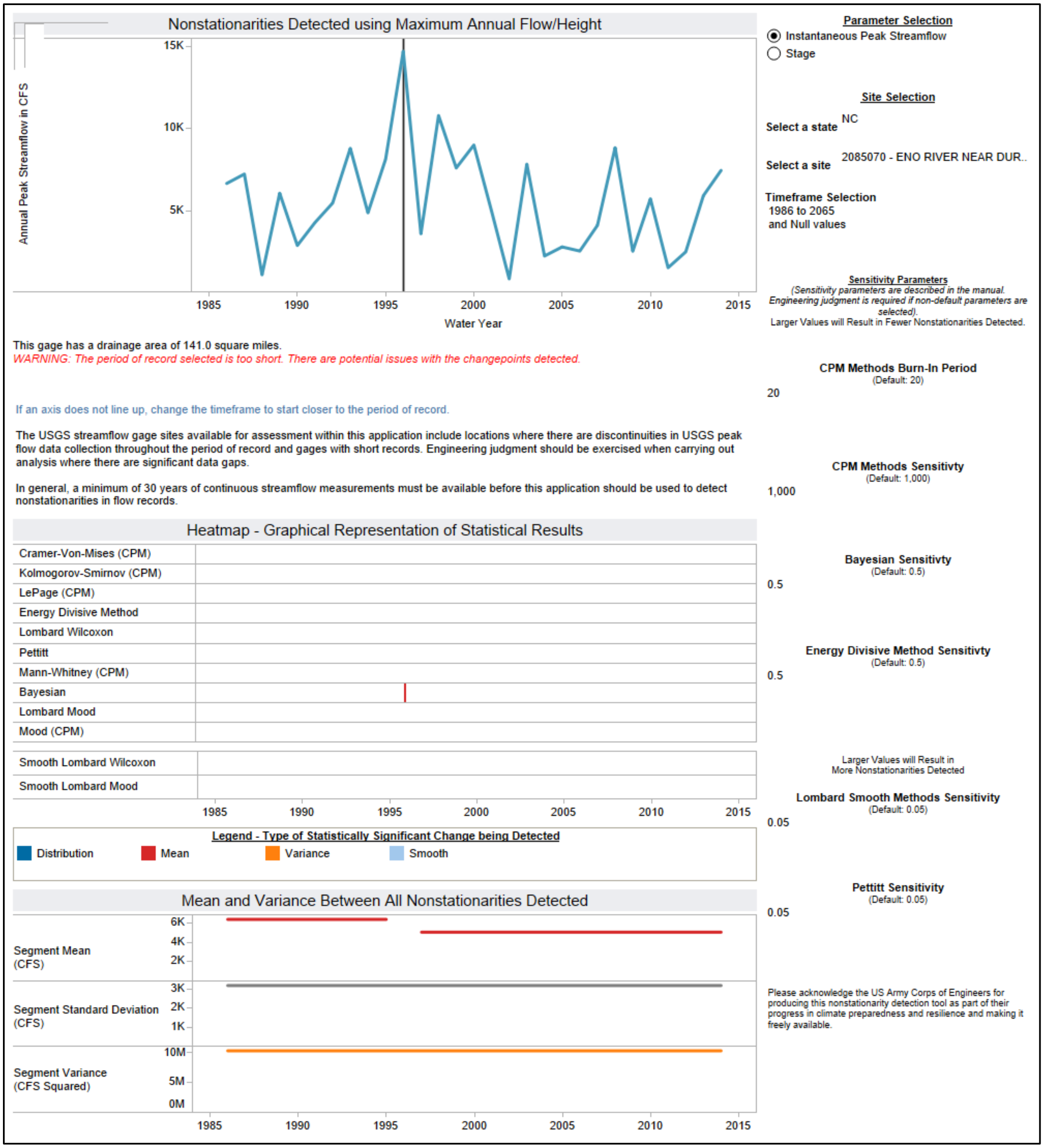

Figure 238. Nonstationarity Detection Results for Gage 02088070 Eno River near Durham, NC

Table 86. Summary of Observed Streamflow Trends in Annual Peak Streamflow using NSD

Gage Number	Gage Name and Location	POR for CHAT	POR for NSD	POR Note	Consensus	Robustness	Conclusion
02085070	Eno River Near Durham, NC	1985-2014	1964-2014	Complete	No	No	None
0208524975	Little River at Farintosh, NC	1996-2014	N/A	Not in NSD	N/A	N/A	N/A
02086500	Flat River at Dam near Bahama, NC	1985-2014	1928-2014	Gap 1959-1962, 1966-1983, 1994-1995, 1997-2000	No	No	None
02086624	Knap of Reeds Creek near Butner, NC	1985-2014	N/A	Gap 1996-2005	N/A	N/A	N/A
02086849	Ellerbe Creek near Gorman, NC	1985-2014	N/A	Gap 1989-1991, 1994-2006, 2008-2009	N/A	N/A	N/A
02087183	Neuse River near Falls. NC	1981-2014	1971-2019	Complete	Yes	No	CWM, LP, PT and MW in 2000
02087324	Crabtree Creek at US 1 at Raleigh, NC	1991-2014	N/A	Not in NSD	N/A	N/A	N/A
02087359	Walnut Creek at Sunnybrook Drive near Raleigh, NC	1996-2014	N/A	Not in NSD	N/A	N/A	N/A
02087500	Neuse River near Clayton, NC	1981-2014	1928-2014	Complete	Yes	Yes	CVM, KS, LP, PT, MW in 1966

02087580	Swift Creek near Apex, NC	2002-2014	1954-2014	Complete in CHAT, NAP gap 1972-2001	No	No	None
02088000	Middle Creek near Clayton, NC	1985-2014	1940-2014	Complete	No	No	None
02088500	Little River near Princeton, NC	1985-2014	1931-2014	Complete	No	No	None
02089000	Neuse River near Goldsboro, NC	1981-2014	1930-2014	Complete in CHAT, NSD gap 2009	No	No	None
02089500	Neuse River at Kinston, NC	1981-2014	1928-2012	Complete	No	No	None
02090380	Contentnea Creek near Lucama, NC	1965-2014	1965-2014	Complete	No	No	None
02091500	Contentnea Creek at Hookerton, NC	1929-2014	1929-2014	Complete	No	No	None
02091814	Neuse River near Fort Barnwell, NC	1997-2014	NA	Not in NSD	N/A	N/A	N/A

11.6 Projected Trends in Future Climate and Climate Change

11.6.1 Climate Hydrology Assessment Tool

The USACE Climate Hydrology Assessment Tool (CHAT) was used to assess projected, future trends within the Neuse-Pamlico watershed, HUC-0302. The tool displays the range of projected annual maximum monthly streamflows from 1950 - 2099, with the values from 1950 – 2005 representing hindcast modeled flows and 2006 – 2099 representing forecasted projections.

Figure 239 through Figure 241 displays the range of projections for 93 combinations of CMIP5 GCMs and RCPs produced using BCSD statistical downscaling for HUC 03020201 Upper Neuse, HUC 03020202 Middle Neuse, and HUC 03020204 Lower Neuse. These flows are simulated using an unregulated VIC hydrologic model at the outlet of respective HUC. It should be noted that the hindcast outputs do not replicate historically observed streamflow and should therefore not be compared directly with historical observations. This is in part because observed streamflows are impacted by regulation, while the VIC model used to produce the results displayed in Figure 239 through Figure 241 are representative of the unregulated condition.

The hindcast outputs have a range of 6,000-45,000 cfs, while the projections have a range of 6,000-65,000 cfs. The spread of the model results also increases with time, which is to be expected as uncertainty in future projection increases as time moves away from the model initiation point. Sources of variation and the significant uncertainty associated with these models include the boundary conditions applied to the GCMs, as well as variation between GCMs and selection of RCPs applied. Each GCM and RCP independently incorporate significant assumptions regarding future conditions, thus introducing more uncertainty into the climate changed projected hydrology. Climate model downscaling and a limited temporal resolution further contribute to the uncertainty associated with CHAT results. There is also uncertainty associated with the hydrologic models. The large spread of results shown in Figure 239 through Figure 241 highlights current climatic and hydrologic modeling limitations and associated uncertainty.

Figure 242 through Figure 244 display only the mean result of the range of the 93 projections of future, climate changed hydrology which are shown in Figure 239 through Figure 241. A linear regression line was fit to both the hindcast outputs and the forecast projections. For the Upper Neuse simulated historical time period no statistically significant trend was detected. For the Upper Neuse simulated future a statistically significant trend was found using the t-Test (p=0.000016), the Mann-Kendall Test (p=0.00038), and the Spearman Rank-Order test (p=0.000009) methods. There was found the be an increasing trend with a slope of 4.16 cfs/yr, meaning over the forecasted time period (2006-2099) the annual maximum of monthly mean streamflows increased 387 cfs or ~6% of the streamflow. For the Middle Neuse simulated historical time period no statistically significant trend was detected. For the Middle Neuse simulated future a statistically significant trend was detected using the t-Test (p=0.000025), the Mann-Kendall Test (p=0.00011), and the Spearman Rank-Order test (p=0.000019) methods. There was found the be an increasing trend with a slope of 8.55 cfs/yr, meaning over the forecasted time period (2006-2099) the annual maximum of monthly mean streamflows increased 795 cfs or ~7% of the streamflow. For the Lower Neuse simulated historical time period no statistically significant trend was detected. For the Lower Neuse simulated future a statistically significant trend was detected using the t-Test (p=0.00042), the Mann-Kendall Test (p=0.00053), and the Spearman Rank-Order test (p=0.00015) methods. There was found the be an increasing trend with a slope of 9.94 cfs/yr, meaning over the forecasted time period (2006-2099) the annual maximum of monthly mean streamflows increased 924 cfs or ~6% of the streamflow.

These outputs from the CHAT suggest that annual maximum monthly flows, and therefore annual peak flows, are expected to increase in the future relative to the current time. Above Falls Lake Dam these increases are expected to be regulated by dam operations, however below the dam the 6-7% increases in annual maximum monthly flows may cause isolated project performance issues but not cause widespread project failure.

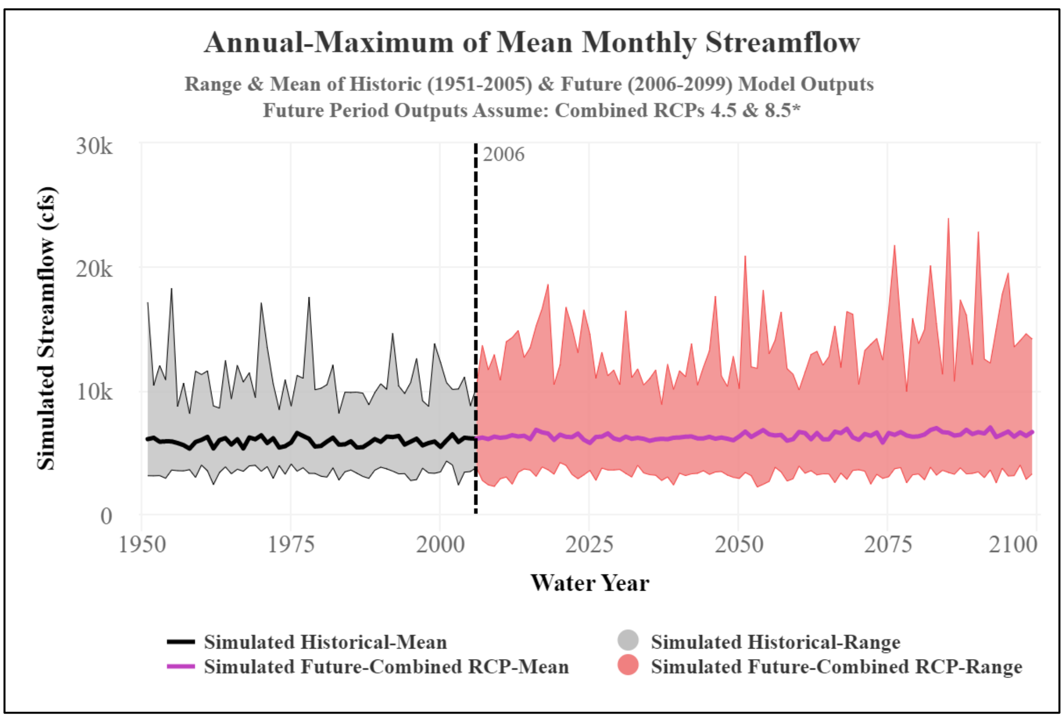

Figure 239. Range of GCM/RCP Projections for the HUC-03020201 Upper Neuse

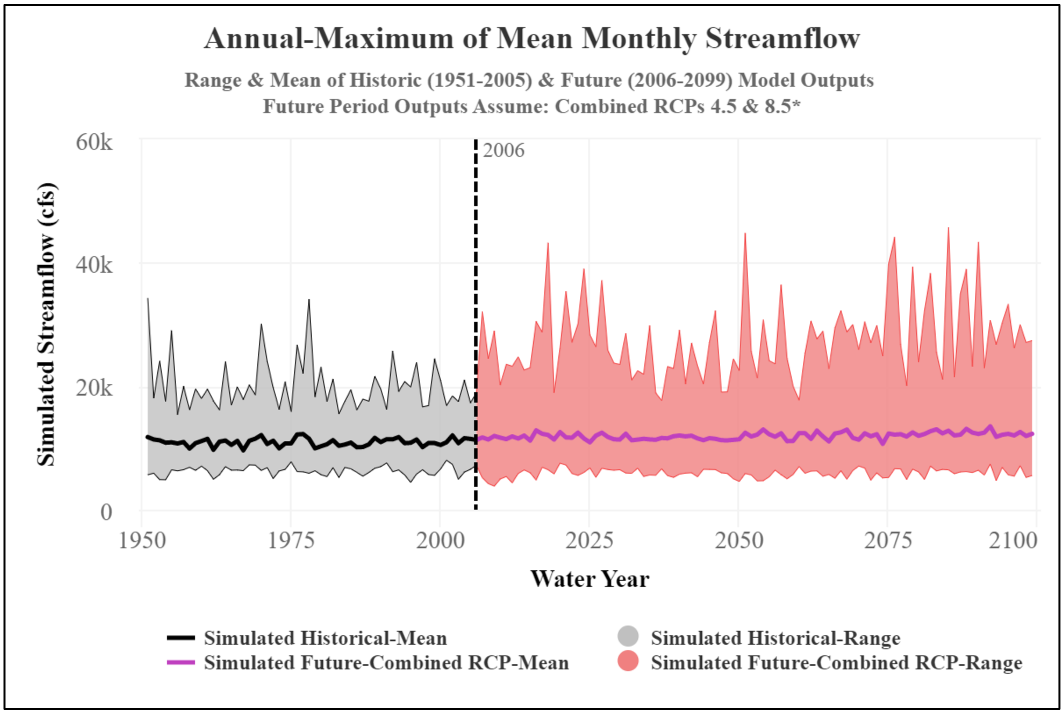

Figure 240. Range of GCM/RCP Projections for the HUC-03020202 Middle Neuse

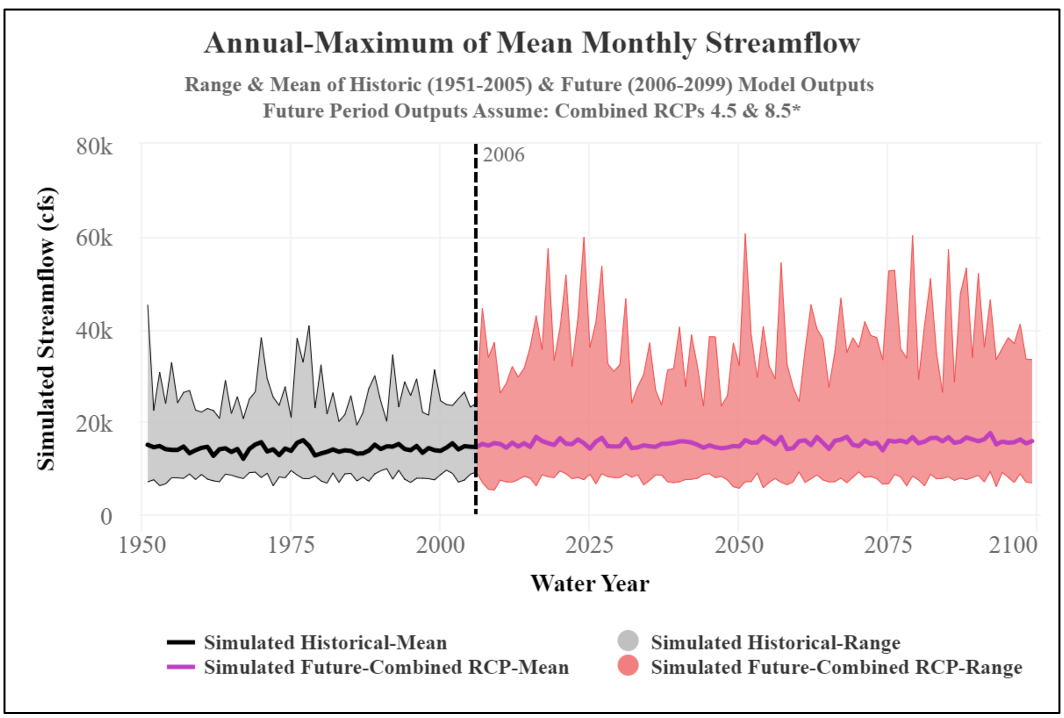

Figure 241. Range of GCM/RCP Projections for the HUC-03020204 Lower Neuse

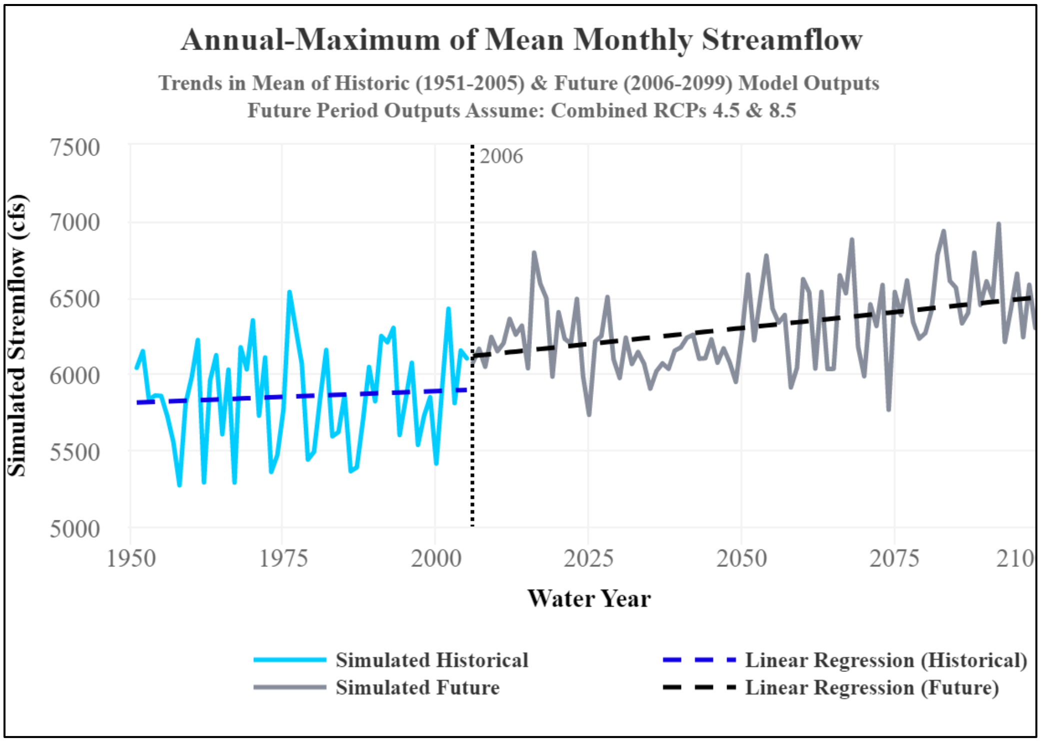

Figure 242. Mean of GCM/RCP Projections for the HUC-03020201 Upper Neuse

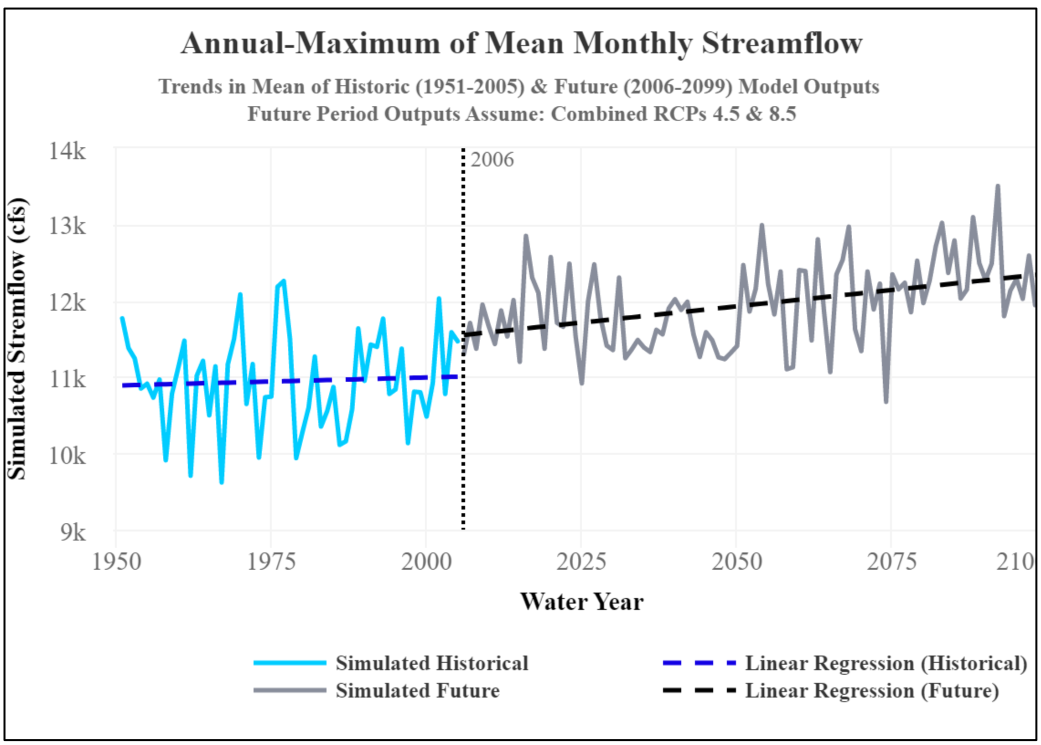

Figure 243. Mean of GCM/RCP Projections for the HUC-03020202 Middle Neuse

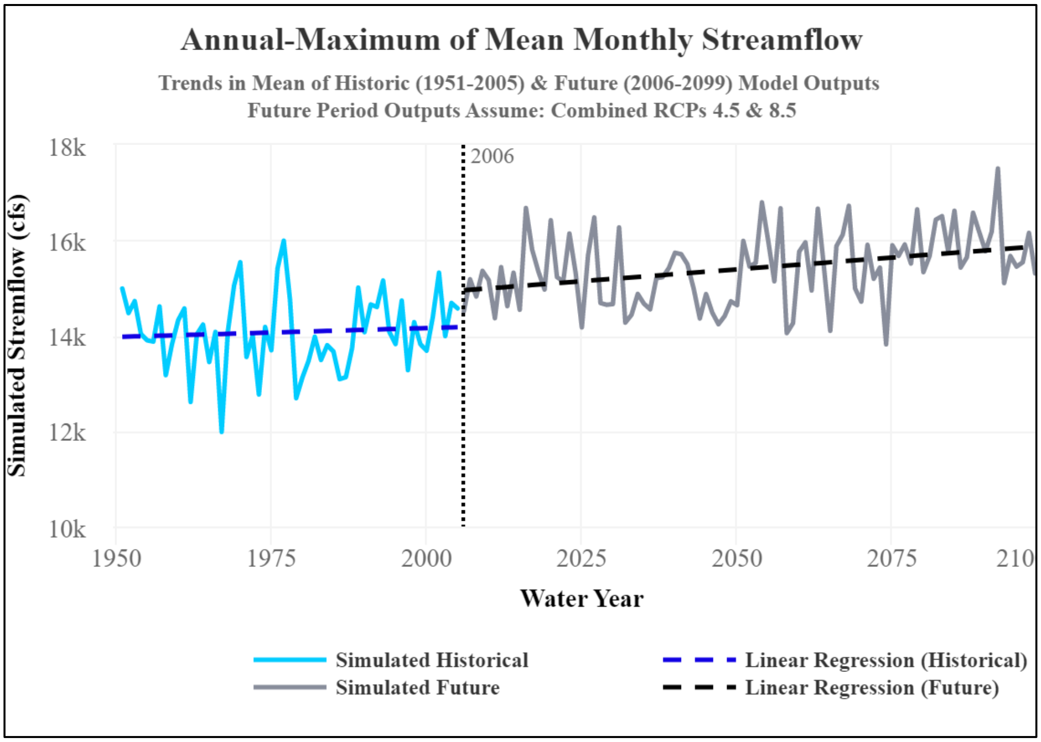

Figure 244. Mean of GCM/RCP Projections for the HUC-03020201 Lower Neuse

11.6.2 Vulnerability Assessment

The USACE Watershed Climate Vulnerability Assessment Tool (VA Tool) facilitates a screening level, comparative assessment of how vulnerable a given HUC-4 watershed is to the impacts of climate change relative to the other 201 HUC-4 watersheds within the continental United States (CONUS) using the same 93 projections in the CHAT. The tool can be used to assess the vulnerability of a specific USACE business line such as “Flood Risk Reduction” or “Navigation” to projected climate change impacts. Assessments using this tool help to identify and characterize specific climate threats and particular sensitivities or vulnerabilities, at least in a relative sense, across regions and business lines. The tool uses the Weighted Ordered Weighted Average (WOWA) method to represent a composite index of how vulnerable a given HUC-4 watershed (Vulnerability Score) is to climate change specific to a given business line. The HUC-4 watersheds with the top 20% of WOWA scores are flagged as being vulnerable.

Flood risk reduction is the most relevant business line for the Neuse River Basin Feasibility Study and is the primary business line analyzed with the USACE Climate Vulnerability Assessment Tool. Other business lines included in the VA Tool are ecosystem restoration, emergency management, hydropower, navigation, recreation, regulatory, and water supply. While the flood risk reduction is the main business line discussed in detail due to the Flood Risk Management authority under which this study was initiated, all other business lines were analyzed as well.

When assessing future risk projected by climate change, the USACE Climate Vulnerability Assessment Tool makes an assessment for two 30-year epochs of analysis centered at 2050 and 2085. These two periods were selected to be consistent with many of the other national and international analyses. The Vulnerability tool assesses how vulnerable a given HUC-4 watershed is to the impacts of climate change for a given business line using climate hydrology based on a combination of projected climate outputs from the GCMs and representative concentration pathway (RCPs) resulting in 100 traces per watershed per time period. The top 50% of the traces is called “wet” and the bottom 50% of the traces is called “dry.” Meteorological data projected by the GCMs is translated into runoff using the Variable Infiltration Capacity (VIC) macro-scale hydrologic model. For this assessment, the default National Standards Settings are used to carry out the vulnerability assessment.

For the Flood Risk Management business line, the HUC 0302 Neuse-Pamlico Basin is not within the top 20% of vulnerable watersheds within the CONUS for any of the four scenarios, which is not to say that vulnerability to future climate change does not exist within the basin. displays the overall vulnerability scores for the business line relevant to this study under both wet and dry scenarios and under both time epochs. The indicators driving the residual vulnerability for the flood risk management business line is shown in Figure 245. and Table 88 display the indicators contributing to vulnerability within the Neuse-Pamlico Basin for the flood risk reduction business line; the tables are generally sorted from largest to smallest average indicator contribution to vulnerability. Additionally, the tables display the indicator code, name, and a brief description of the indicator’s meaning.

Regarding the Flood Risk Reduction business line, the primary indicators driving vulnerability within the watershed are the flood magnification factor (indicator 568C), and acres of urban area within the 500-year floodplain (indicator 590). The flood magnification factor represents how the monthly flow exceeded 10% of the time is predicted to change in the future; a value greater than 1 indicates flood flow is predicted to increase, which is true for the Neuse-Pamlico Basin. The acres of urban area within the 500-year floodplain indicator measures the acres of urban area within the 500-year floodplain, which impacts the land use/landcover in the area.

Note that some of the indicators contain a suffix of “L” (local) or “C” (cumulative). Indicators with an “L” suffix reflect flow generated within only one HUC-4 watershed, whereas indicators with a “C” suffix reflect flow generated within a HUC-4 watershed and any upstream watersheds.

It is important to note the variability displayed in the VA tool’s results (Table 88) highlights some of the uncertainty associated with the projected climate change data used as an input to the VA tool. Because the wet and dry scenarios each represent an average of 50% of the GCM outputs, the variability between the wet and dry scenarios underestimates the larger variability between all the underlying projected climate changed hydrology estimates. This variability can also be seen between the 2050 and

2085 epochs, as well as various other analysis within this report, such as output from the CHAT (Figure 239 - Figure 241).

Table 87. Overall Vulnerability WOWA Score for Epochs and Selected Scenarios

Business Line	**Flood Risk Reduction**	
Epoch	**2050**	**2085**
Dry	45.13	47.59
Wet	48.16	51.99

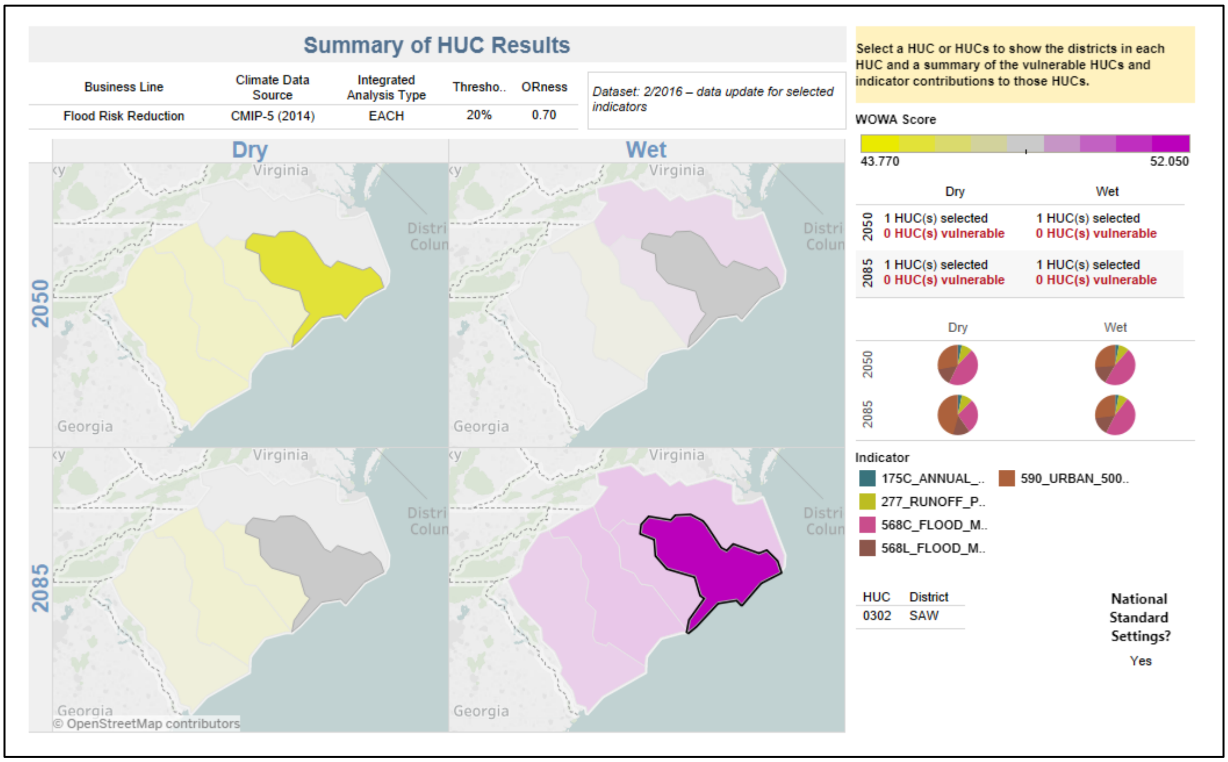

Figure 245. VA Tool Summary of WOWA HUC Results for Flood Risk Reduction Business Line

Table 88. Vulnerability Indicators for Flood Risk Reduction Business Line

Indicator Code	Flood Risk Reduction Indicator Name	Description	2050 Dry	2050 Wet	2085 Dry	2085 Wet
568C	Cumulative Flood Magnification Factor	Change in flood runoff: ratio of indicator 571C (monthly runoff exceeded 10% of the time, including upstream freshwater inputs) to 571C in base period.	45.15%	46.92%	28.07%	47.18%
277	Percent Change in Runoff Divided by the Percent Change in Precipitation	Median of: deviation of runoff from monthly mean times average monthly runoff divided by deviation of precipitation from monthly mean times average monthly precipitation.	8.84%	8.45%	8.94%	7.66%
568L	Local Food Magnification Factor	Change in flood runoff: Ratio of indicator 571L (monthly runoff exceeded 10% of the time, excluding upstream freshwater inputs) to 571L in base period.	14.82%	15.40%	14.18%	15.49%
175C	Cumulative Annual Covariance of Unregulated Runoff	Long-term variability in hydrology: ratio of the standard deviation of annual runoff to the annual runoff mean. Includes upstream freshwater inputs (cumulative).	3.18%	2.97%	3.28%	2.72%
590	Acres of Urban Area Within 500-Year Floodplain	Acres of urban area within the 500-year floodplain.	28.01%	26.25%	45.54%	26.96%

11.6.3 Sea Level Change Assessment

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

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

An initial step in evaluating sea level change for the Neuse River basin study was to identify a near-by NOAA water level gage with a sufficiently long data record. The analysis was based on the NOAA tide gauge located in Beaufort, North Carolina (Station #8656483), approximately 35 miles southeast of the City of New Bern, NC. The gage is compliant and active with a historic recording of verified hourly height water level from 1967 to present, there were two data gaps from 1967 to 1973 and1973 to 1977. Station location and datum information are shown in Figure 246 and Figure 247, respectively. From the linear relative sea level trend for this gauge is 3.36 mm/year (0.011 ft/year) with a 95% confidence interval of +/- 0.34 mm/year (0.0011 feet/year) based on monthly mean sea level data. For the 50-year analysis of 2040 to 2090 this is equivalent to an increase of 0.55 ft in sea level. For reference, the absolute global sea level rise is believed to be 1.7-1.8 millimeters/year, or roughly half of the relative rise predicted at the Beaufort, NC gauge. For stations with sufficient historical data the linear relative sea level trends were calculated by NOAA in overlapping 50-year increments. The variation on each 50-year trend is provided in Figure 249. The variation of each 50-year trend, with 95% confidence interval, is plotted against the mid-year of each 50-year period. The solid horizontal line represents the linear relative sea level trend using the entire period of record. Interannual variation at this site is shown in Figure 250.

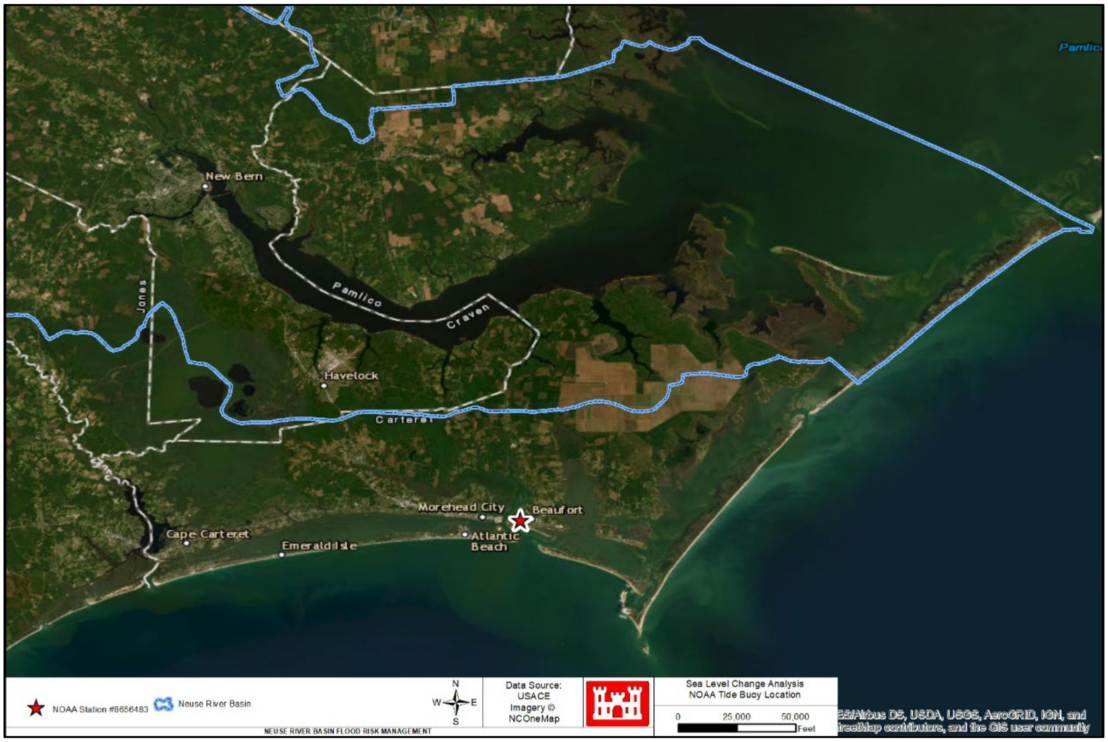

Figure 246. Location of Beaufort, NC Gage 8656483

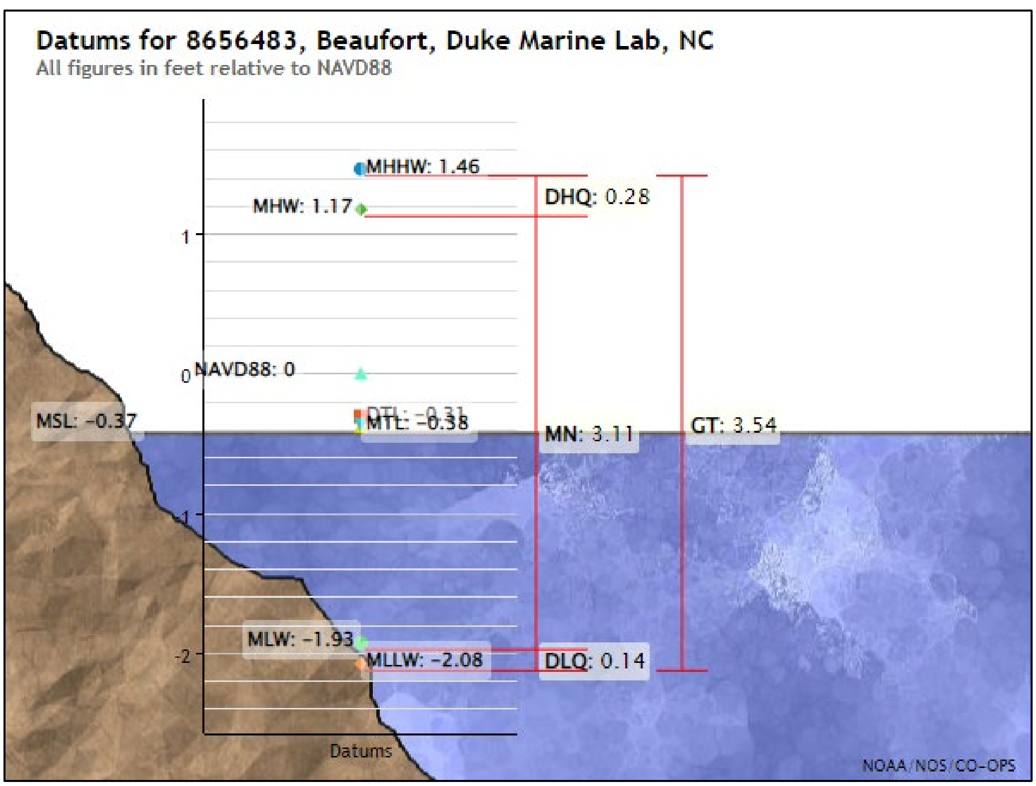

Figure 247. Beaufort, NC Gauge 8656483 Datum Information

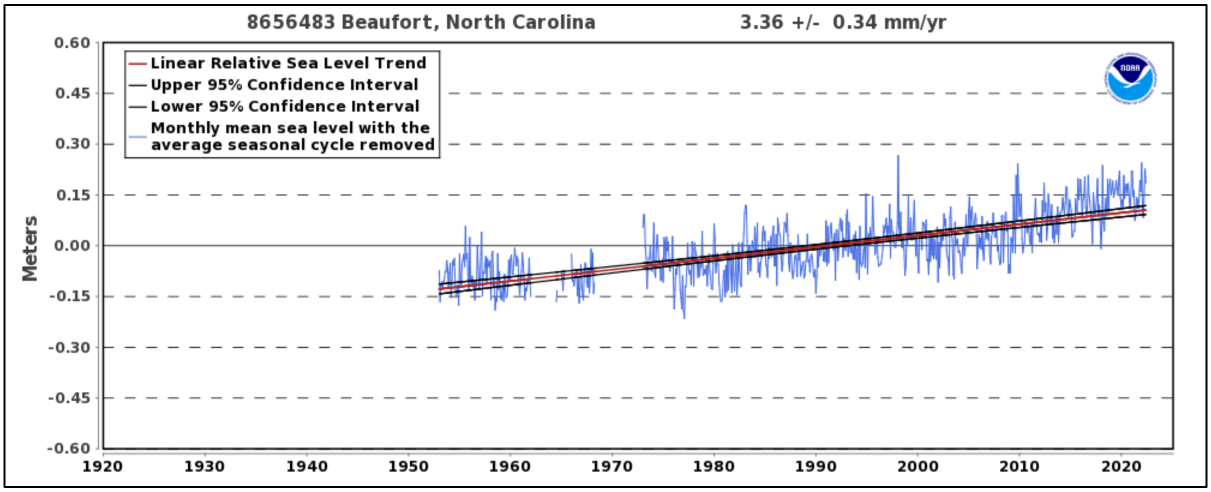

Figure 248. Beaufort, NC Gauge 8656483 Relative Sea Level Trend

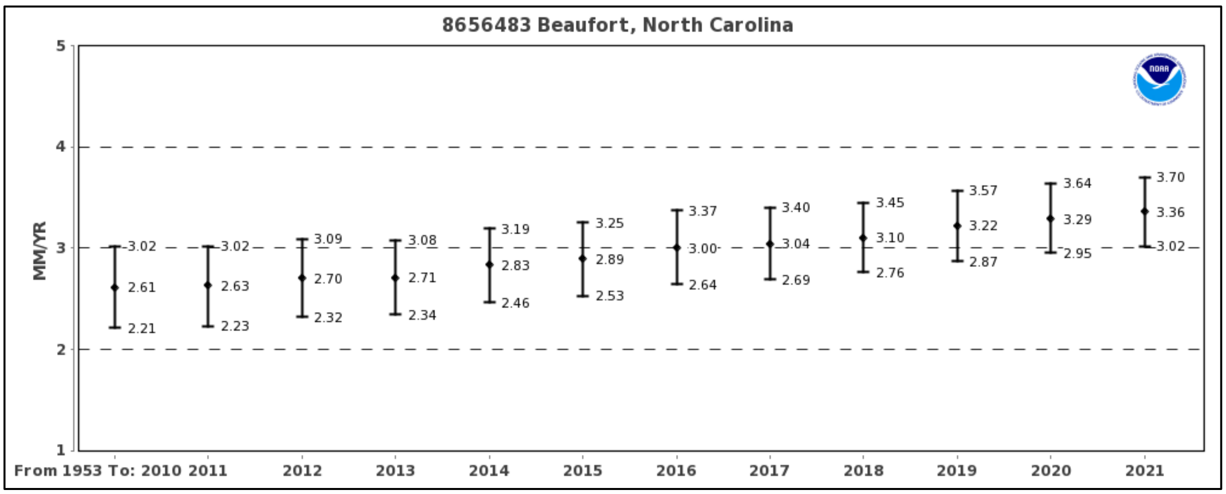

Figure 249. Beaufort, NC Gauge 8656483 Variation of 50-Year Relative Sea Level Trend

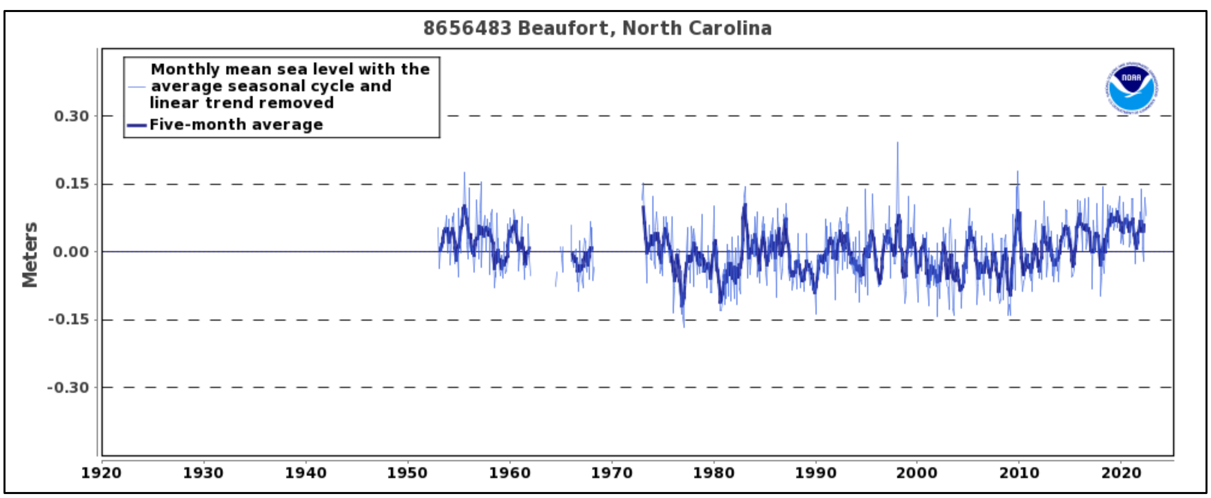

Figure 250. Beaufort, NC Gauge 8656483 Interannual Variation

In order to evaluate SLC for the Neuse River basin study with regards to future trends, mainly will the rate of sea level rise accelerate in the future, the USACE online tool Sea Level Tracker was utilized (https://climate.sec.usace.army.mil/slr_app/). Extreme water levels (EWL) incorporated into the tool are based on statistical probabilities using recorded historic monthly extreme water level values. The Sea Level Tracker is used to compare actual mean sea level (MSL) values and trends for specific NOAA tide gauges with the USACE SLC scenarios as described in ER 1100-2-8162 and Engineer Pamphlet (EP) 1100-2-1. The Sea Level Tracker tool calculates the USACE Low, Intermediate and High sea level change scenarios based on global and local change effects. Historical MSL is represented by either 19-year or 5-year midpoint moving averages. Guidance in using the Sea Level Tracker and technical background is provided in USACE “Sea Level Tracker User Guide”, Version 1.0, December, 2018.

The Sea Level Tracker tool was used to evaluate the Beaufort, NC NOAA tide buoy data. The regionally corrected rate of 0.00249 mm/yr (0.00817 ft/yr) was used as the rate of SLC and was sourced from Technical Report NOS CO-OPS 065 (NOAA, 2013) and accounts for vertical land motion. Based on the regional rate only, the sea level increase was 0.41 ft during the 50-year period of 2040 to 2090. Figure 251 presents the results of the Tracker tool focused on trends between 1967 to 2021. The light blue line represents the 5-year moving average and the heavy dark blue line represents the 19-year moving average. The 19-year average is useful in that this represents the moon’s metonic cycle and the tidal datum epoch. These estimates are referenced to the midpoint of the latest National Tidal Datum epoch, 1992. The reader is referred to ER 1100-2-8162 for a detailed explanation of the procedure, equations employed, and variables included to account for the eustatic change as well as site specific uplift or subsidence to develop corrected rates. The red line is the High SLC prediction, the green is the Intermediate and the blue is the Low rate prediction. From Figure 251 it can be noted that the 19-year moving average has covered a majority of the vertical distance that separates the Intermediate and High curves. The 5-year rate is tracking nearly on top of the High curve though displays more cyclical characteristics.

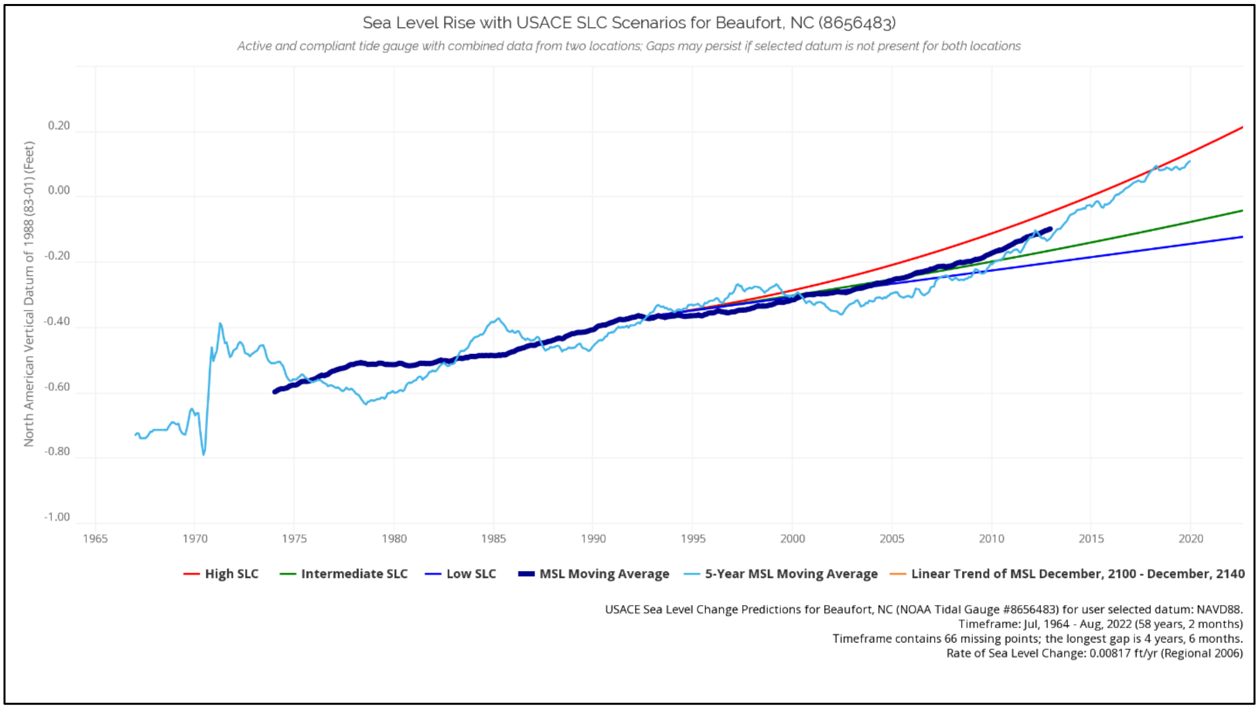

Figure 251. USACE Sea Level Tracker for Beaufort, NC (8656483) through Year 2021

The future USACE sea level predictions for the Neuse River basin study based on the Beaufort, NC NOAA station are provided in Figure 252. For predicted SLC through year 2090, the Low rate sea level rise was 0.133 m (0.44 ft), the Intermediate SLC increase was 0.395 m (1.30 ft), and the High SLC increase was 1.229 m (4.03 ft). For predicted SLC through year 2140, the Low rate sea level rise was 0.257 m (0.84 ft), the Intermediate SLC increase was 0.855 m (2.81 ft), and the High SLC increase was 2.75 m (9.02 ft).

A comparison of the predicted Beaufort, NC USACE SLC trends was attempted to the Oregon Inlet Marina, NC NOAA station (8652587), however, that gauge was considered non-compliant and unreliable due to miss data. A cursory review of this station within the USACE Sea Level Tracker tool, shown in Figure 253, suggested that historical moving averages are trending consistently above the Intermediate Curve.

The USACE High SLC scenario was selected for the Neuse River basin study because it tracked well with the 19-year and 5-year moving averages in Figure 251. This High SLC scenario with moving averages plotted consistently above the Intermediate SLC scenario was similarly noted at a regional tide gauge (Wilmington, NC NOAA station (8658120). The High rate was also selected in coordination with the USACE Climate Preparedness and Resilience Community of Practice.

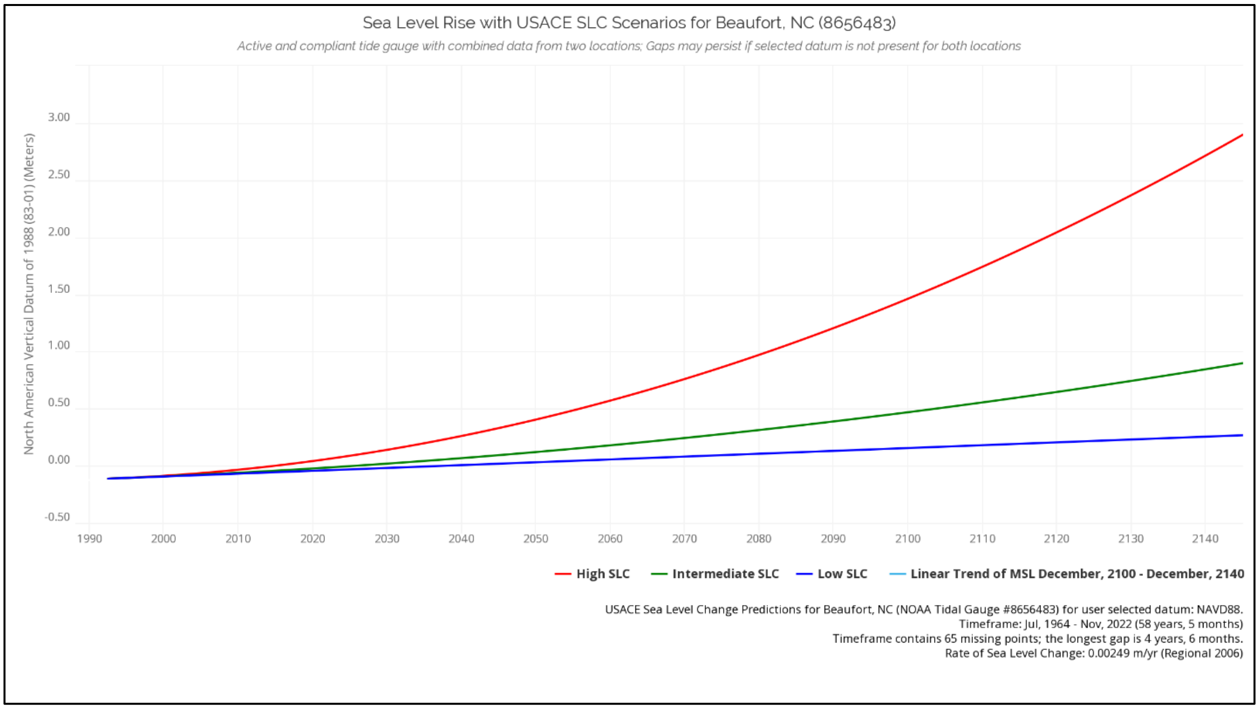

Figure 252. USACE Sea Level Change Predictions

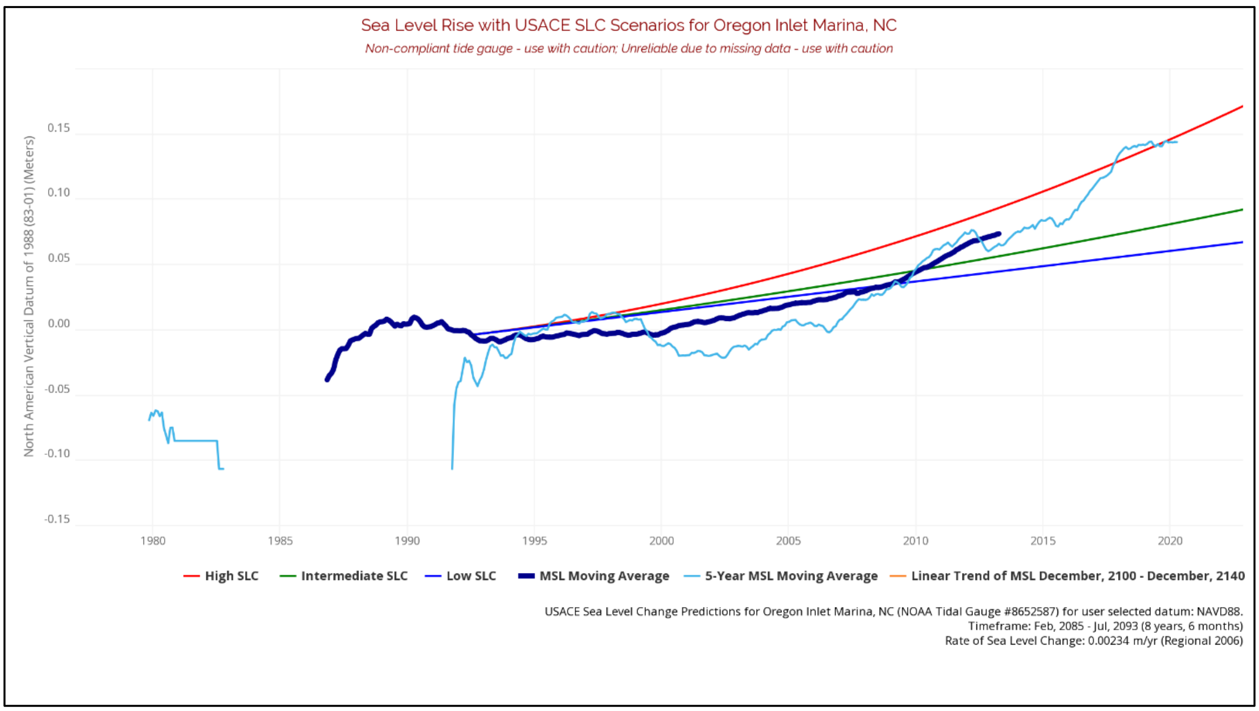

Figure 253. USACE Sea Level Tracker for Oregon Inlet Marina, NC (8652587) through Year 2021

11.7 Summary and Conclusions

11.7.1 Observed Summary and Conclusions

Based on the observed literature review, there is a consistent consensus that points toward mild increases in annual temperature in the South Atlantic-Gulf Region over the past century, particularly over the past 40 years. Annual precipitation totals have become more variable in recent years compared to earlier in the 20th century. Evidence has also been presented, but with limited consensus, of increasing trends in the magnitude of annual and seasonal precipitation for parts of the study area. These results are seemingly contradicted by several studies that have shown decreasing trends in streamflow throughout the area, particularly since the 1970s. The study authors evaluated watersheds that experienced minimal water withdrawals and/or transfers. Results presented here also suggest that increasing temperatures may also play a role in decreasing streamflows, despite the lack of corresponding precipitation decline.

Two of the gages analyzed via CHAT detected a statistically significant linear trend, Neuse River near Falls, NC and Little River Tributary at Fairntosh, NC. The Neuse River near Falls gage showed a statistically significant downward trend in observed peak annual flows but would be expected as the flow at this stationed is regulated by dam operations with one purpose being flood reduction. Little River tributary at Fairntosh also showed a statistically significant downward trend, however the results are highly driven by the observed peak flow in 1996. When that data point is removed the site no longer shows a statistically significant trend. Every other gage that was analyzed via Climate Hydrology Assessment Tool did not have a statistically significant linear trend. There were no statistically significant trends detected in either gage that would indicate significant changes in observed streamflow due to climate change, long-term natural climate trends, or land use/land cover changes.

Using the Nonstationarity Detection Tool two stream gages produced nonstationarities, 02087183 Neuse River near Falls, NC and 02087500 Neuse River near Clayton, NC. The NSD detected a consensus of the underlying distribution and the mean in 2000 at the Neuse River near Falls, NC, however this can be explained by a change in the flood operations of Falls Lake Dam. A monotonic trend analysis detected a statistically significant decrease over the entire gage period and in the period before the detected nonstationarity, however in the period after the nonstationarity there is no statistically significant trend. The NSD also detected a consensus in the change of the underlying distribution and the mean at Neuse River near Clayton, NC in 1966, however if the analysis is limited to after Falls Lake Dam went into operation, no nonstationarities were detected. A monotonic trend analysis over the entire gage period and the period up to the detected nonstationarity detected a statistically significant decreasing trend, however no statistically significant trend was detected in the period after the nonstationarity. All other gages either did not detect a nonstationarity, did not have enough data to perform an analysis, or the data that was found on USGS was not recent enough to be feasible for the analysis.

11.7.2 Projected Trends Summary and Conclusions

Based on the projected literature review, there is strong consensus in the literature that air temperatures will increase in the study area, and throughout the country, over the next century. The studies reviewed here generally agree on an increase in mean annual air temperature of approximately 2 to 4 ºC by the latter half of the 21st century for the South Atlantic-Gulf Region. Projections of precipitation in the study area are less certain than those associated with air temperature. Results of the studies reviewed here are roughly evenly split with respect to projected increases vs. decreases in future annual precipitation. Projections generated by coupling GCMs with macro-scale hydrologic models in some cases indicate a reduction in future streamflows but in other cases indicate a potential increase in streamflows in the study region. Of the limited number of studies reviewed here, results are approximately evenly split between the two.

Upon examination of the range of model results from the Climate Hydrology Assessment Tool, there is a clear increasing trend in the higher projections, whereas the lower projections appear to be relatively stable and unchanging through time. The spread of the model results also increases with time, which is to be expected as uncertainty in future projection increases as time moves away from the model initiation point. Sources of variation and the significant uncertainty associated with these models include the boundary conditions applied to the GCMs, as well as variation between GCMs and selection of RCPs applied. Climate model downscaling and a limited temporal resolution further contribute to the uncertainty associated with CHAT results. There is also uncertainty associated with the hydrologic models. The large spread of results shown in Figure 239 through Figure 241 highlights current climatic and hydrologic modeling limitations and associated uncertainty. Figure 242 through Figure 244 displays only the mean result of the range of the 93 projections of future, climate changed hydrology which are shown in Figure 239 through Figure 241. A linear regression line was fit to both the hindcast outputs and the forecast projections. For the Upper Neuse there was found the be a statistically significant increasing trend with a slope of 4.16 cfs/yr, meaning over the forecasted time period (2006-2099) the annual maximum of monthly mean streamflows increased 387 cfs or ~6% of the streamflow. For the Middle Neuse simulated historical time period no statistically significant trend was detected. For the Middle Neuse simulated future there was found the be a statistically significant increasing trend with a slope of 8.55 cfs/yr, meaning over the forecasted time period (2006-2099) the annual maximum of monthly mean streamflows increased 795 cfs or ~7% of the streamflow. For the Lower Neuse simulated historical time period no statistically significant trend was detected. For the Lower Neuse there was found the be a statistically significant increasing trend with a slope of 9.94 cfs/yr, meaning over the forecasted time period (2006-2099) the annual maximum of monthly mean streamflows increased 924 cfs or ~6% of the streamflow.

Results from the USACE Vulnerability Assessment tool were analyzed for the project area and found no outstanding vulnerabilities compared with other HUCs across the continental United States. While the project area is not within the top 20% of vulnerable HUCs nationally, that does not imply that vulnerability to climate change does not exist. The VA tool indicates that the change in flood runoff (cumulative), combined with the acres of urban area within 500-year floodplain, are driving flood risk reduction vulnerability.

The USACE Sea Level Tracker tool was used to project future USACE sea level predictions for the Neuse River basin study based on the Beaufort, NC NOAA station. For year 2090, the predicted Low rate sea level rise was 0.133 m (0.44 ft), the Intermediate SLC increase was 0.395 m (1.30 ft), and the High SLC increase was 1.229 m (4.03 ft). For year 2140, the predicted Low rate sea level rise was 0.257 m (0.84 ft), the Intermediate SLC increase was 0.855 m (2.81 ft), and the High SLC increase was 2.75 m (9.02 ft).

11.7.3 Recommended Plan and Climate Change Considerations

Primary goals of the Recommended Plan are to lower the risk of economic damages and life safety associated with flooding. However, residual risks, in particular those resulting from climate change conditions, exist within the watershed. There are inherent and explicit uncertainties with projections of climate change that have been assumed during the plan formulation process. For example, both observed and projected future precipitation trends, as described in Section 11.7.1 and Section 11.7.2 carry residual risk due to lack of consensus. Residual risk associated with the Recommended Plan specifically due to climate change are summarized in Table 89.

Table 89. Climate Risk Register – Recommended Plan

Feature or Measure	Trigger	Hazard	Consequence	Qualitative Likelihood
Dry Floodproofing	Increased water surface elevations near structure footprint due to higher intensity rainfall	Reduced assurance of structural integrity; increased probability of exceeding feature design capacity	Flooding of structure, economic damages	Possible to Likely - While there is less consensus on future rainfall projections, some climate models project an increase in frequency of heavy downpours, especially through atmospheric rivers
Dry Floodproofing	Increased precipitation from larger, slower-moving storms	Future flood volumes may be larger than present; large flood volumes may occur more frequently	Flood waters may surround structure for longer durations, and more frequently, potentially damages feature and/or structure	Possible to Likely - While there is less consensus on future rainfall projections, some climate models project an increase in frequency of heavy downpours, especially through atmospheric rivers
Dry Floodproofing	Increased water surface elevations near structure footprint due to sea level change	Reduced assurance of structural integrity; increased probability of exceeding feature design capacity	Flooding of structure, economic damages	Unlikely - Project area is outside of tidally influenced region and above elevation 50 ft, NAVD88

The Recommended Plan reduces flood damage by providing the nonstructural measure of dry floodproofing to structures within the study area. Unlike the nonstructural measure of elevation, where a structure is physically raised above a flood threshold, dry floodproofing is relatively more flexible and may potentially be modified to potential changes in climate. Limitations to the degree of retrofitting, depending on the final dry floodproofing design, could present a challenge.

Preliminary plan formulation considered alternatives of levees, floodwalls, overbank detention, and channel modification. However, identification of low engineering effectiveness at reducing flood risk and/or disproportionate benefit-to-cost resulted in their elimination from further consideration in this study. Preliminary alternatives such as levees and channel modification that either function to form a barrier or more efficiently handle flood flows would be sensitive to projected future residual risks of precipitation and overall flood volume with channel and overbank. For preliminary alternatives within the tidally influenced region, impacts from sea level change would likely decrease effectiveness and increase implementation and maintenance costs.

In summary, the potential for increased rainfall intensity and flood volume associated with projected future climate change is present for the Recommended Plan. These residual risks may lead to increased peak discharges and water surface elevations in the future. The Recommended Plan has not been modified as a result of the climate change assessment.

As mentioned earlier in this appendix and within the main report, there remains significant residual climate risk predominantly caused by sea level change present within the tidally influenced region of the Neuse River basin. Consequently, future study of this tidally influenced region is recommended in a separate study that will utilize more detailed coastal modeling tools to assess and investigate alternative measures to potentially reduce coastal flood risk.

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