Appendix A

Appendix A
Princeville, North Carolina Flood Risk Management Integrated Feasibility Report
And Environmental Assessment
Hydrology and Hydraulics

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Chapter 1 **Introduction**

General Study Purpose

The purpose of this report is to evaluate flood damage reduction measures for the Town of Princeville, NC. After significant flooding in 1958, the US Army Corps of Engineers constructed an earthen levee in 1967. In 1999, Princeville was devastated by flooding from the Tar River as a result of Hurricane Floyd. This flood was estimated as being less frequent than a 0.002 annual exceedance probability event (as defined in section 1.3). Flooding entered Princeville by flanking both the up- and down-stream sections of the levee as well as overtopping the levee proper.

This Appendix contains technical background information, modeling and results for the hydrology and hydraulics (H&H) analysis to support the flood risk management study. This information can also be used by reviewers and readers to evaluate the validity and accuracy of the H&H information and techniques used in this study.

Basic Definitions

The following terms are used throughout this appendix. A basic definition and example for each term are given below:

Exceedance Probability Event: This represents the median value of probability that a specified target will be exceeded in any given year. For example, an exceedance probability event of 0.01(1-percent) has a 1/100 chance of occurring in any given year.

Annual Exceedance Probability (AEP): As defined in EM 1110-2-1619, the AEP is a measure of the likelihood of exceeding a specified target in any year given the full range of annual possible flood discharges. An example is that the AEP of a 10-m levee might be 0.01 which implies that the annual maximum stage in any year has a 1-percent chance (0.01-probability) of exceeding the elevation of the top of the levee. AEP can be used to analyze a multitude of events in which, for instance, various heights of a levee will eventually be overtopped.

Conditional Non-exceedance Probability (CNP): The index of the likelihood that a specified target will not be exceeded, given the occurrence of a hydro-meteorological event. CNP is a useful indicator of performance because of the uncertainty in discharge-probability and stage-discharge estimates. An example is that the CNP of a proposed 5.0 ft high levee might be 0.75 for the 0.002 AEP event. This means that if the plan is implemented, the probability is 0.75 (75-percent) that the stage will not exceed 5.0 ft, given the occurrence of a 0.002 AEP event.

Conditional Non-Exceedance Probability and Assurance: These terms are interchangeable.

Survey Datum

A datum reference or datum plane is a level from which the vertical location of features can be measured, which can be physical or calculated. The term "NGVD '29" refers to the National Geodetic Vertical Datum of 1929 and is a geodetic (land-based) datum that was set based on the mean sea level (MSL) of 1929. NGVD '29 and MSL generally mean the same thing. Datum "NAVD '88" (North American Vertical Datum 1988) is an updated leveling adjustment made in 1991. For the Princeville and Tarboro area the conversion factor is as follows:

NAVD 88 elevation = NGVD 29 elevation – 1.06 feet

In the case of this project, the original levee was designed and constructed using the MSL datum, the original flood insurance study was based on the NGVD '29 datum, and the 2004 flood insurance study and this report are based on the NAVD '88 datum.

Acronyms

AEP – Annual Exceedance Probability
CFS - Cubic Feet per Second
CNP – Conditional Non-Exceedance Probability
DA – Drainage Area
EAD – Expected Annual Damage
EM – Engineer Manual
ER – Engineer Regulation
FEMA – Federal Emergency Management Agency
FIS – Flood Insurance Study
FPIS – Flood Plain Information Study
GIS – Geographic Information System
GPS – Geographic Positioning System
H&H – Hydrology and Hydraulics
HEC – Hydrologic Engineering Center
HEC-FDA – Flood Damage Reduction Analysis
HEC-RAS – River Analysis System
LIDAR – Light Detecting and Radar
MSL – Mean Sea Level
NAVD – North American Vertical Datum
NCDOT – North Carolina Department of Transportation
NCDWQ – North Carolina Division of Water Quality
NGVD – National Geodetic Vertical Datum
NRCS – National Resource Conservation Service
PED – Preconstruction, Engineering, and Design
Q – Discharge
USACE – U.S. Army Corps of Engineers
USGS – U.S. Department of Interior, U.S. Geological Survey
WS – Water Surface
WY – Water Year
XS – Cross Section

Chapter 2 **Previous Hydrologic and Hydraulic Studies**

The 1963 *Princeville Dike, Tar River Edgecombe County, North Carolina Detail Project Report* was completed in 1963 and indicates that the original authorized levee protection level was determined primarily using the Tarboro gage information. The levee height was designed based on the record flood elevation of 1919, plus an additional 4 feet (2 feet of freeboard plus 2 feet). Minor residual flood impacts associated with levee construction were mentioned in this report as resulting in less than a 0.05 foot water surface elevation increase. There is no indication that any type of backwater modeling was performed. It is important to note that discharges indicating levels of protection in this report were determined from the 1919 flood water surface elevation (stage-frequency relationship) and not derived from a discharge-frequency curve.

Flood Plain Information Studies (FPIS) were completed as part of the Corps' Flood Plain Management program in the 1960s and 1970s. These studies developed flood profiles, flood plain maps and flood-ways for the studied streams. The main purposes of these studies were to identify the flood plain that has a 0.01 AEP and define the required flood-way. In 1965, a study was prepared for the reach of the Tar River around Tarboro and a tributary named Hendricks Creek that produces flooding in Tarboro.

Around 1968 a basin study for the Tar River, titled *Tar River Basin, North Carolina* was prepared to develop a plan of development of the water and related land resources of the Tar River Basin for flood control, water supply, water-quality control, and recreation to meet current and future water-resource-conservation needs in the basin. The study proposed the construction of three reservoirs, Grey Rock Lake located on the upper end of the Tar River in Granville County, Spring Hope Lake Located on the Tar River in Nash County, and White Oak Lake located on the Fishing Creek in Halifax and Nash Counties. The total project had a benefit to cost (B/C) ratio of 1.03, while the Grey Rock and White Oak lakes had B/C ratios of 1.2 and 1.1, respectively. Table 2-1 indicates the following stage reductions at Tarboro due to new construction.

Table 2-1. 1968 Report Stage Reductions

AEP	0.005	0.01	0.02	20	50
Project:	**Grey Rock Lake**				
Stage Reduction (ft)	-0.5	-0.8	-1.2	-2.3	-4.1
Project:	**Spring Hope Lake (or Spring Hope Lake + Grey Rock Lake)**				
Stage Reduction (ft)	-1.7	-2.3	-3.4	-5.7	-5.6

In 1975 a Flood Hazard Information Report for the Town of Tarboro was prepared to provide additional information on the flooding that was defined in the1965 FPIS for Tarboro.

In 1978 a regional hydrologic study was prepared to develop regionalized regression equations for the computation of frequency related discharges for the coastal and piedmont areas of North Carolina, (*Composite Hydrologic Study of Floods in Coastal Plain and Piedmont Areas of North Carolina*, by Nathan O. Thomas, P.E. dated February 1978).

In 1980-81 a Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) was prepared for Edgecombe County, N.C. the Town of Princeville, and the Town of Tarboro. These studies were developed to identify the flood plains and floodway that is required for the local governments to enforce the National Flood Insurance Program. An update to the 1980-81 FIS was prepared in 2004 (State of North Carolina, 2004).

In 2000 through 2002 the U.S Geological Survey (USGS), in the report The National Flood-Frequency Program-Methods for Estimating Flood Magnitude and Frequency in Rural and Urban Areas in North Carolina, 2001, recalculated the frequency statistics for the gages in North Carolina using information from the state wide stream gage system. This was done in part as a result of the flooding produced by Hurricane Floyd and in part to bring the data up to date. This re-analysis of the gage data produced a new frequency-stage-discharge relationship for the Tarboro Stream gage.

In 2004 the State of North Carolina and a contractor prepared a new FIS for Edgecombe County and Incorporated communities. This study used the newly developed gage frequency data from the USGS and new topographic data produced using Light Detecting and Ranging (LIDAR) collected ground elevation data. All of the water surface elevations were developed using Hydrologic Engineering Center – River Analysis System (HEC-RAS) and a combination of survey and topographic data. As a result of the USGS basin wide frequency discharge calculations performed in 2000-2002 the Tarboro gage frequency - stage curve shifted up causing the frequency stage relationship in the 2004 FIS for Princeville and Tarboro to cause an increase in the Tar River stages. The 0.01 AEP event increased in the range of 3 feet. The Tar River hydraulics for this flood risk management project was developed using the backwater model from the 2004 FIS.

Chapter 3 **Existing Conditions**

Basin Characteristics

The Tar River, North Carolina rises in Person County near the Virginia state line. It flows southeasterly for about 190 miles where it changes its name to the Pamlico River, at Washington, North Carolina. At Washington the Tar River has a drainage area of 3,081 square miles. The Pamlico River continues another 38 miles and flows into the Pamlico Sound which joins the Atlantic Ocean through several inlets. The basin flows through two geographic areas of the state, the piedmont and the coastal plains. About 50% of the basin is in each region, with the dividing line being called the "fall line" and crosses the Tar River channel at Rocky Mount. The basin has several large tributaries: Fishing Creek (DA of 863 square miles) is a 94 mile long stream that enters the Tar River from the north, about 7 miles upstream of Princeville. Deep Creek (DA of 270 square miles) is about 80 miles long and enters Fishing Creek from the east near its confluence with the Tar River. Swift Creek (DA of 270 square miles) is about 80 miles long and located between the Tar River and Fishing Creek and enters the Tar River about 14 miles above Princeville. Town Creek (DA of 199 square miles) is about 35 miles long and enters the Tar River about 9 miles south of Princeville. Conetoe Creek is about 14 miles long and enters the Tar River about 17 miles downstream from Princeville. The lower 46 miles of Fishing Creek, all of Deep Creek, the lower 21 miles of Swift Creek, and all of Town Creek and Conetoe Creek are in the coastal plain region. The channel slope east of Greenville is very flat with slope less than 1 foot of drop per mile. From Greenville to Rocky Mount the channel slope averages around 1 foot per mile and from Rocky Mount to US 15 in Granville County, the channel slope is about 2 feet per mile. Upstream of US 15 the slope is about 8 feet per mile.

The basin consists mainly of rural areas comprised of farm land with associated dwellings and wooded areas. There are numerous scattered small urban areas with less than 10,000 residents and only several areas that have in excess of 10,000 occupants. Most of the income is from farming or farm-related activities. The largest metropolitan area in the basin is Rocky Mount, located in Nash and Edgecombe Counties.

Basin Land Use

There are 50 municipalities within 16 counties in the Tar-Pamlico River basin, with the largest being Rocky Mount, Greenville, Henderson, Oxford, Tarboro, and Washington. According to the 2000 Census, the population within the basin is 414,929. The population density in the Tar-Pamlico Basin is 75 people per square mile which is roughly half of the statewide density. Thus the basin is relatively rural (NCDWQ 2004a).

Land use in the lower Tar River subbasin, which encompasses Princeville, is approximately 40% cultivated crops, 1% pasture, 43% forest, 10% urban & built-up, 1% federal, and 5% other (NCDWQ 2004a). Publicly owned lands include three National Wildlife Refuges (Lake Mattamuskeet, Pocosin Lakes, Swan Quarter) and two State Parks (Goose Creek and Medoc Mountain). North Carolina's largest lake, Lake Mattamuskeet, also is located in this basin (NCDENR 2003).

The basin contains three "major" reservoirs including Lake Devin (a water supply reservoir for Oxford), Lake Royale, and Tar River Reservoir in Rocky Mount. Several old millponds and beaver impoundments are found throughout the Tar-Pamlico River basin as well (NCDWQ 2003).

Rainfall Characteristics

The rainfall is generally well distributed throughout the year. Showers and thunderstorms produce most of the precipitation during the spring and summer. The heaviest and most extended rains in the area are experienced from hurricanes and tropical storms which usually occur during late summer and autumn. The mean annual precipitation ranges from approximately 48 to 53 inches.

Seasonal Runoff Effects

As expected, during the summer season evapotranspiration rates reach their peak. Snowfall constitutes a negligible portion of precipitation and does not affect runoff appreciably, with November typically being the driest month. The low rate of development in the basin produces a low rate of change in the impervious cover and thus allows the rainfall-runoff relationship to remain fairly constant. A change in the rainfall runoff may also be created by the conversion of wooded areas to cultivated lands or other types of open areas. The Tar River is a perennial river and is not dramatically affected by seasonal change. Runoff throughout the Tar River Basin amounts to roughly one-third of the annual rainfall. The mean annual runoff is 13.87 inches.

Past Storm Events

Storms that generally occur in the Tar River Basin are seasonal thunder storms, northeasters, and tropical cyclones (storms, depressions and hurricanes). Movement of warm, moist air into contact with surrounding cooler air produces violent thunderstorms and intense precipitation during the summer months, these storms will occasionally contain high winds and hail. Northeasters are similar to thunderstorms; however, they are offshore disturbances and are usually larger than thunderstorms and have stronger winds and more intense precipitation. Tropical cyclones (storms and hurricanes) are the most severe of the storms to occur on the east coast. They generally occur in late summer and fall and generate heavy and prolonged precipitation. The following storms were summarized from the USACE report, *Tar River Basin, North Carolina, 1968*.

Storm of August 23-26, 1908: This storm produced widespread rains over the South Atlantic States. The heaviest 3-day rainfall was at Monroe, N.C., where 15.56 inches fell. Approximately the same amount fell at several places in Georgia and South Carolina. This storm was preceded by heavy general rainfall on August 19-21. This is considered one of the greatest known to occur in the Tar River Basin. The Tarboro gage recorded a stage of 28.0 feet.

Storm of July 19-23, 1919: The month of July 1919 was one of the wettest on record in the Tar River Basin. Numerous thunderstorms kept the soil well saturated and streams stages above normal throughout the month. The heavy shower activity reached maximum intensity on July 23 when from 2 to 5 inches of precipitation was recorded over the entire basin. A peak stage of record at Tarboro occurred on July 27, with a peak discharge of 52,800 cubic feet per second and a stage of 34.0 feet.

Storm of September 26-30, 1924: During September 1924 two tropical disturbances passed up the Atlantic Coast. The first hurricane passed by Cape Hatteras opposite the Tar River Basin on the 15th and was accompanied by high winds and heavy rains. Thunderstorms followed this storm, maintaining high stages and keeping the grounds in the basin wet. The second hurricane passed over the Tar River basin on the 30th, causing heavy rains over the basin. The heavy rainfall of the second hurricane combined with saturated ground conditions of the basin caused by the earlier rains resulted in a very high rate of runoff and peak stages second to those occurring July 1919 on the Tar River.

Storm of September 17-19, 1928: Heavy rains fell over the basin of the Tar River, beginning on September 1st and continued through the 6th. This precipitation caused high stages on all of the streams and thoroughly saturated the soil. Showers of high intensity occurred almost daily until the 17th, when a West Indian hurricane passed over the basin causing high winds and heavy precipitation. Because the ground was saturated, high runoff resulted and caused extensive flooding on already-swollen streams. This event caused the fourth highest flood peaks of record along the Tar.

Storm of August 10-17, 1940: Intense rains fell over the drainage basin of the Tar River on the 14th - 16th associated with a hurricane which had meandered over the eastern half of North Carolina. Centers of maximum rainfall occurred in or near the basin for the total storm duration of 186 hours, approximately 8 days: East side (Swansboro) – 19.6 inches, west side (Buck Creek) – 16.4 inches, and north side (Keysville, Va.) – 17.5 inches. The precipitation over the basin for this event ranged from 5 to 11 inches. See Figure 3-1 for the discharge hydrograph.

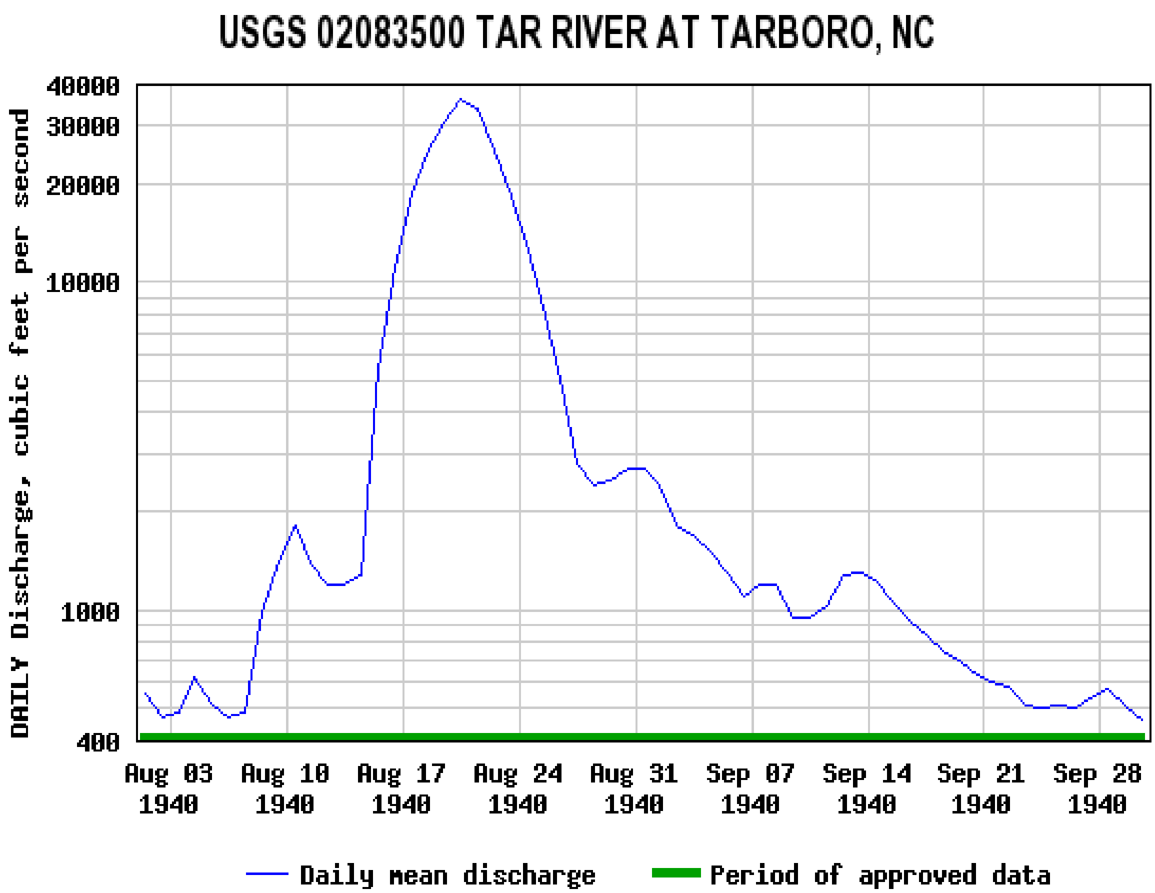

Figure 3-1. Discharge Hydrograph for 1940 Flood

Storm of September 14-18, 1945: This storm was caused by a West Indian hurricane that struck the Florida coast below Miami on the afternoon of September 15. The center of the hurricane moved up the middle of the Florida Peninsula and up through the Atlantic States passing over the Tar River Basin during the night of September 17. The highest rainfall recorded anywhere in the basin for this event was 9.25 inches near Louisburg. The total average rainfall for this event over the basin amounted to 6.06 inches above Tarboro and 5.51 inches above Greenville. See Figure 3-2 for discharge hydrograph.

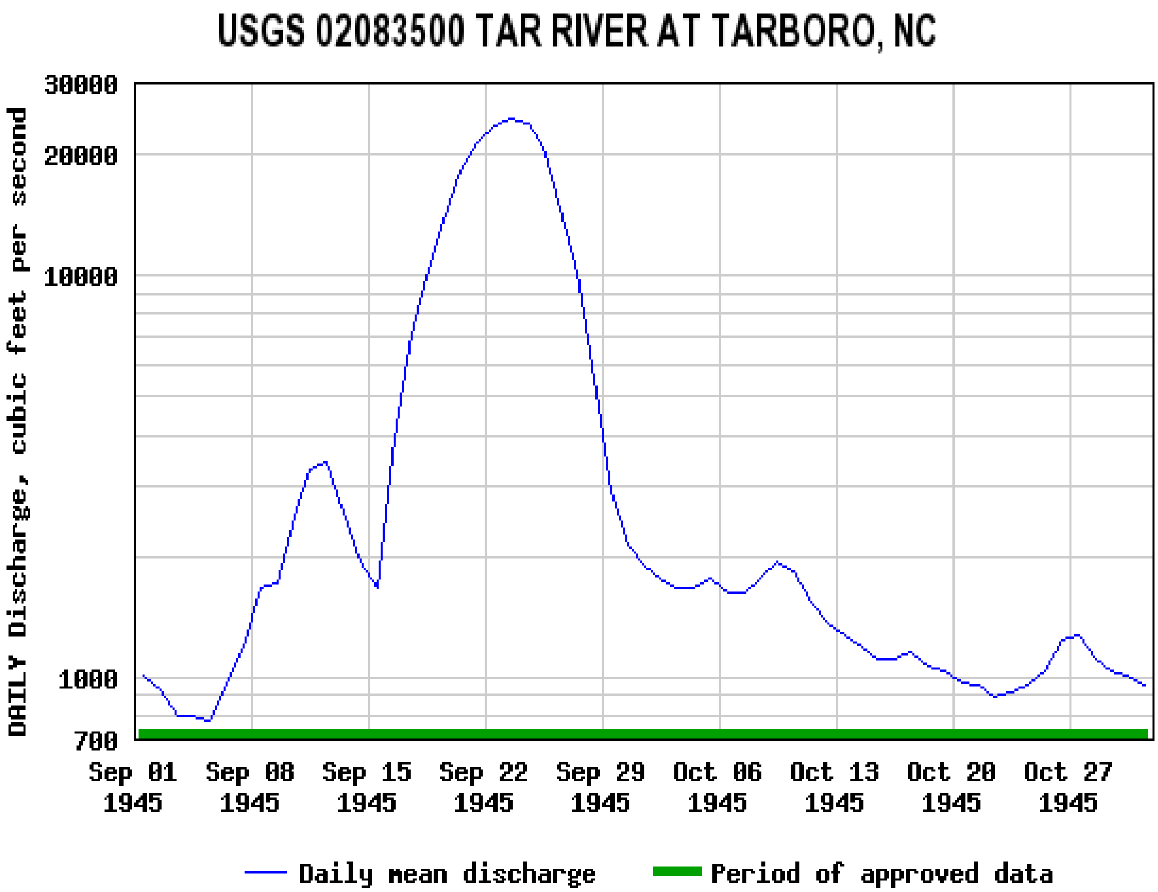

Figure 3-2. Discharge Hydrograph for 1945 Flood

Storm of August 16-19, 1955: This storm was caused by a West Indian hurricane which moved inland along the North Carolina coast at Wilmington, N.C. on the morning of August 17. The hurricane moved across the state in a northwesterly direction and into Virginia by the evening of August 17. This storm was preceded by another hurricane which touched the North Carolina coast during the day of August 12. About 48 hours of intense rainfall was associated with both storm events over the Tar River Basin. For the latter storm, the average rainfall occurring over the basin above Tarboro amounted to 3.91 inches and above Greenville 4.54 inches. Figure 3-3 shows the discharge hydrograph for calendar year 1955 which shows both the storm related discharges in August as well as other events earlier in the year.

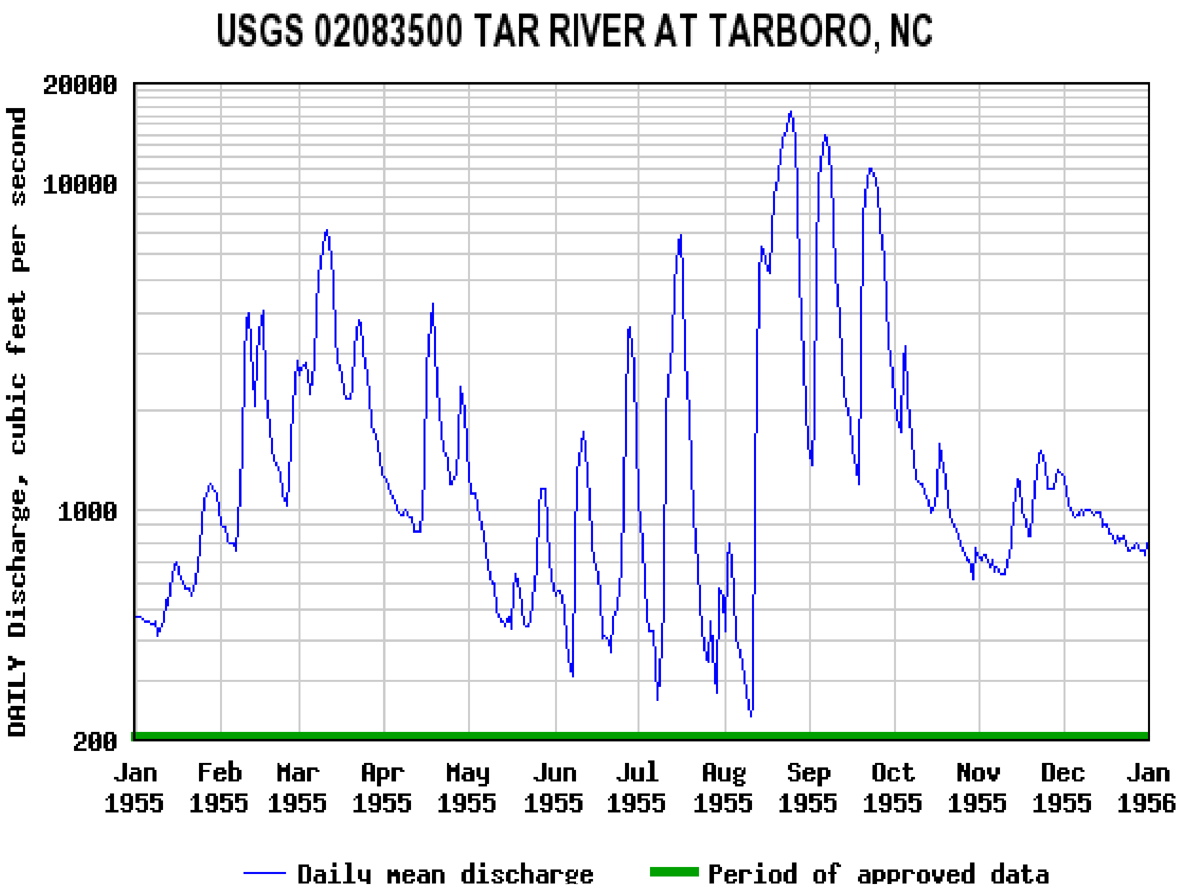

Figure 3-3. Discharge Hydrograph for 1955 Flood

<u>Storm of May 7-17, 1958</u>: Intense rain fell over the eastern piedmont and western coastal plain regions encompassing the Tar River; discharges at Tarboro gage peaking on May 12th. The gage recorded a peak discharge of 26,900 cfs, peak stage of 29.17 ft, and a runoff measured at 3.08 inches. This event instigated the existing Princeville levee construction. Also, a noted peak discharge occurred in September of this year when Hurricane Helene passed by the North Carolina coast causing high winds which drove heavy rain inland. See Figure 3-4 for discharge hydrograph.

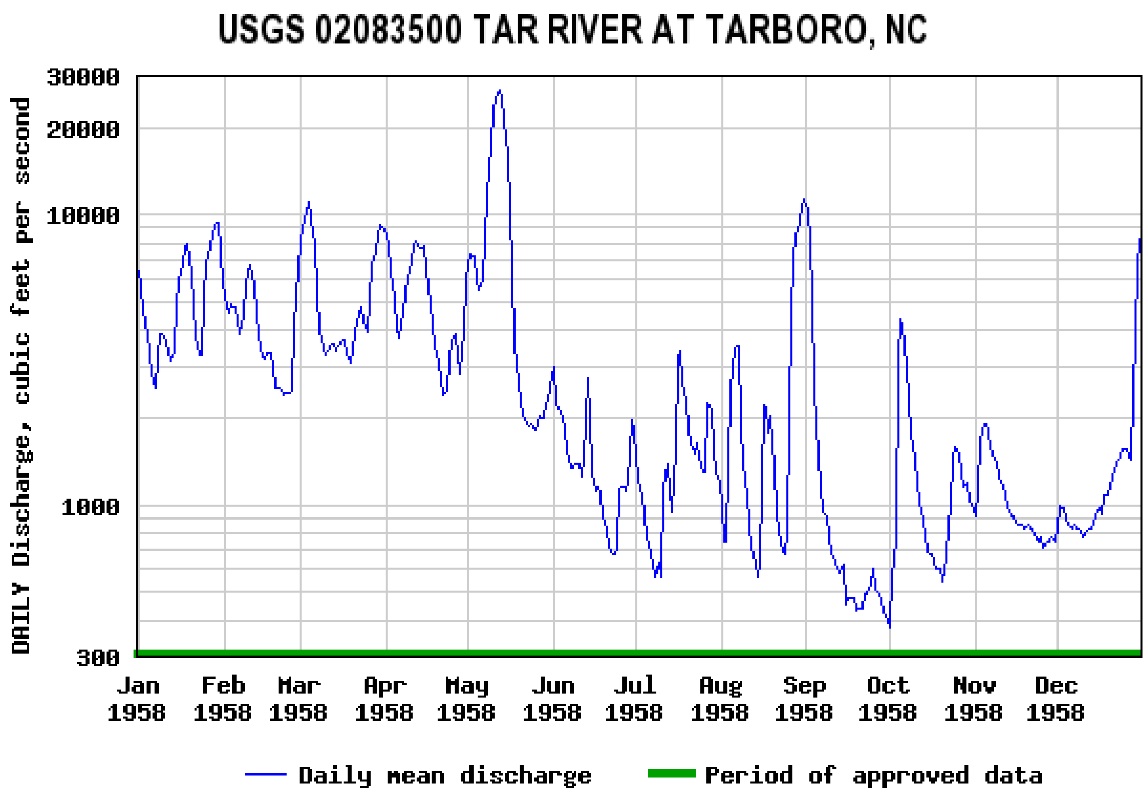

Figure 3-4. Discharge Hydrograph for 1958 Flood

<u>Storms of September and October 1999</u>: The following discussion of rainfalls and hurricane tracks was taken from the US Geological Survey Water-Resources Investigations Report 00-4093, *Two Months of Flooding in Eastern North Carolina, September – October 1999: Hydrologic, Water-Quality, and Geologic Effects of Hurricanes Dennis, Floyd, and Irene* by Jerad D. Bales, Carolyn J. Oblinger, and Asbury H. Sallenger, Jr. published in 2000.

During this two month period the eastern and Southeastern part of North Carolina felt the effects of three Hurricanes – Dennis, Floyd, and Irene. Hurricane Dennis approached the coast and then made an east turn and meandered offshore for several days and then approached the coast in the vicinity of Morehead City. Dennis then preceded inland on September 4 and 5 along a track that took it over New Bern and Kinston and just south of Rocky Mount crossing the Tar-Pamlico Basin just east of Rocky Mount. This track put it along the drainage divide between the Neuse River and the Tar-Pamlico Basins and dumped in excess of 7 inches of rainfall in the central Neuse and Tar-Pamlico Basins.

Hurricane Floyd made landfall near Cape Fear on September 15 with the center making landfall east of Wilmington. The storm moved in a north-northeasterly direction crossing the lower Cape Fear, Neuse, Tar-Pamlico, lower Roanoke and Chowan River Basins. The storm delivered 12 to 18 inches of rain to much of the Neuse and Tar-Pamlico Basins causing regional flooding that continued through the remainder of September and most of October.

Hurricane Irene never made landfall in North Carolina, but moved in a northeasterly direction just off the coast on October 17. However, the rainfall associated with the storm exceeded 5 inches in the eastern part of the Neuse Basin and in the central and eastern parts of the Tar-Pamlico basin. This rainfall plus the already saturated soils and elevated river stages ensured that the Neuse and Tar-Pamlico Rivers remained above flood stages for most of September and October.

This combination of storms produced record rainfalls and runoffs that generated record river stages in the lower two-thirds of the Tar-Pamlico and Neuse River Basins. The Floyd flood was 7.4 feet above the previous flood of record in 1919. Figure 3-5 shows the discharge hydrograph for the August – December 1999 time period.

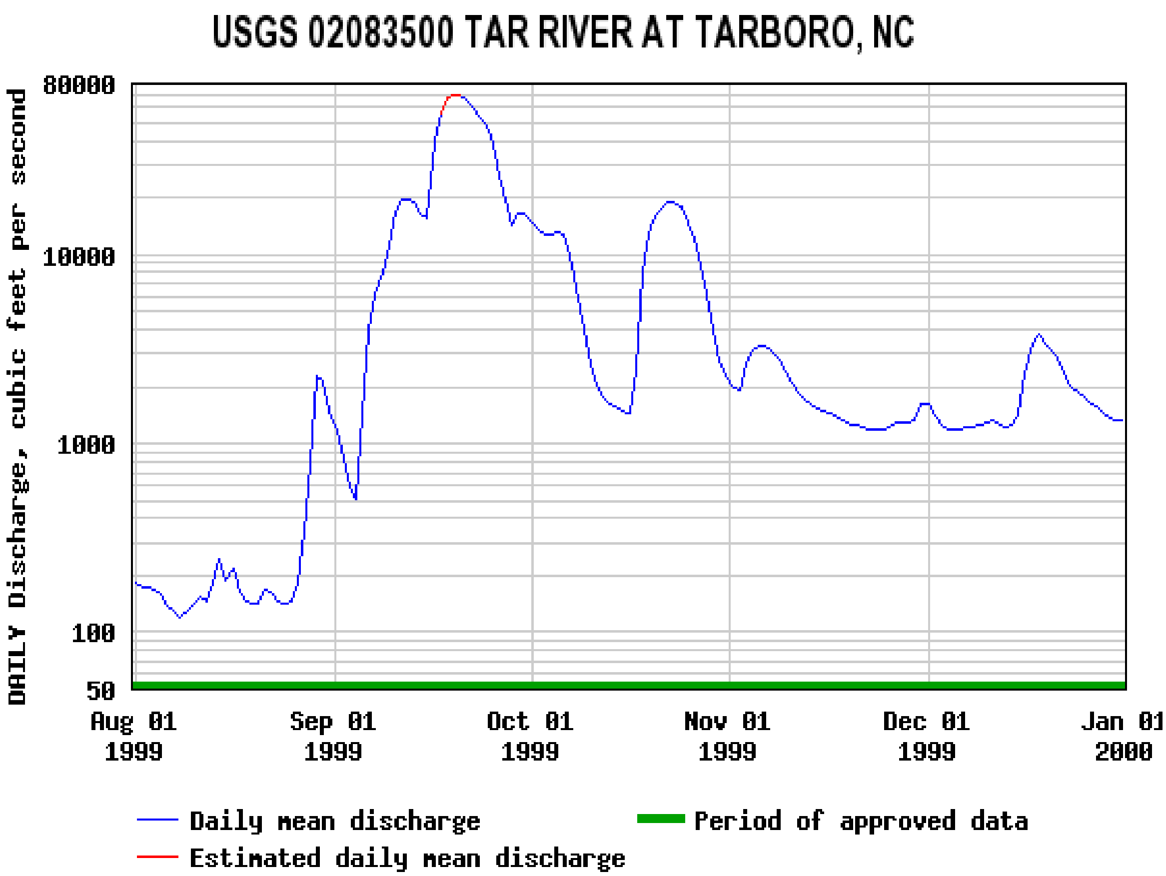

Figure 3-5. Discharge Hydrograph for 1999 Floods

Duration of Flood Events

In 2000 – 2002, the USGS recomputed the frequency – discharge analysis for each of the gages in the Tar River Basin. As a result of these new frequency-discharge curves the new predicted water surface elevations increased. This new analysis shows the significance of Hurricane Floyd as the new flood of record. Hurricane Floyd flooding was 7.4 feet above the old flood of record in 1919 and 6.7 feet above the 0.01 AEP event. Based on the Tarboro gage, flooding from Hurricane Floyd was approximately 3.5 feet above the top of the Princeville levee. The new stage-discharge rating curve for the Tarboro gage is shown in Figure 3-6 with flood frequencies and other major floods. The storm that resulted in the construction of the Princeville levee was the 1958 flood. Based on the rating curve the 1958 flood was only a 0.1 AEP event.

The only hydrographs that are available for this area are based on recorded floods at the Tarboro gage. The nearest event to the 0.01 AEP event for which either daily peak discharge or stage data is available is the 1940 flood. To develop an approximate current 0.01 AEP event hydrograph, the 1940 flood hydrograph was scaled up by using the ratio of the 1940 peak discharge (36,100 cfs) to the 0.01 AEP event discharge (46,208 cfs). This ratio, calculated as 1.28, was applied to the individual discharges from the 1940 flood and the number of days from the peak. This hydrograph gives an indication of the approximate height and width of the 0.01 AEP event. The 0.01 AEP event hydrograph is shown in Figure 3-7, as well as hydrographs for the 1940 flood, Fran and Floyd floods. The rising side of the 0.01 AEP event hydrograph was modified in order to remove the vertical anomaly between -10 and -5 days observed in the Fran hydrograph.

Events causing major flood damage have historically occurred at intervals of roughly 10 to 25 years, prior to the levee construction. Generally it takes a couple of days for the river to start rising after the storm passes. From the time the river started rising during Floyd it took about 10 days to reach the 0.04 AEP event level, approximately 3 additional days to reach the 0.01 AEP event level, and about 2 more days to peak. The Floyd flood stayed above the top of the levee for approximately 4 days before it started to recede. It was approximately 6 days before the water receded back to the 0.01 AEP event level, when water had stopped coming around the north end of the levee, and another 4 days before it dropped below the culverts on the south side. It would have taken as much as another 10 -15 days for all of the water to drain out of Princeville, if pumps had not been used. For a 0.01 AEP event the water level would be above the 0.04 AEP event level for only about 4 days and take approximately 5 to 7 days to drain out of the town. This indicates that one of the problems is getting the water out of the community after the flood.

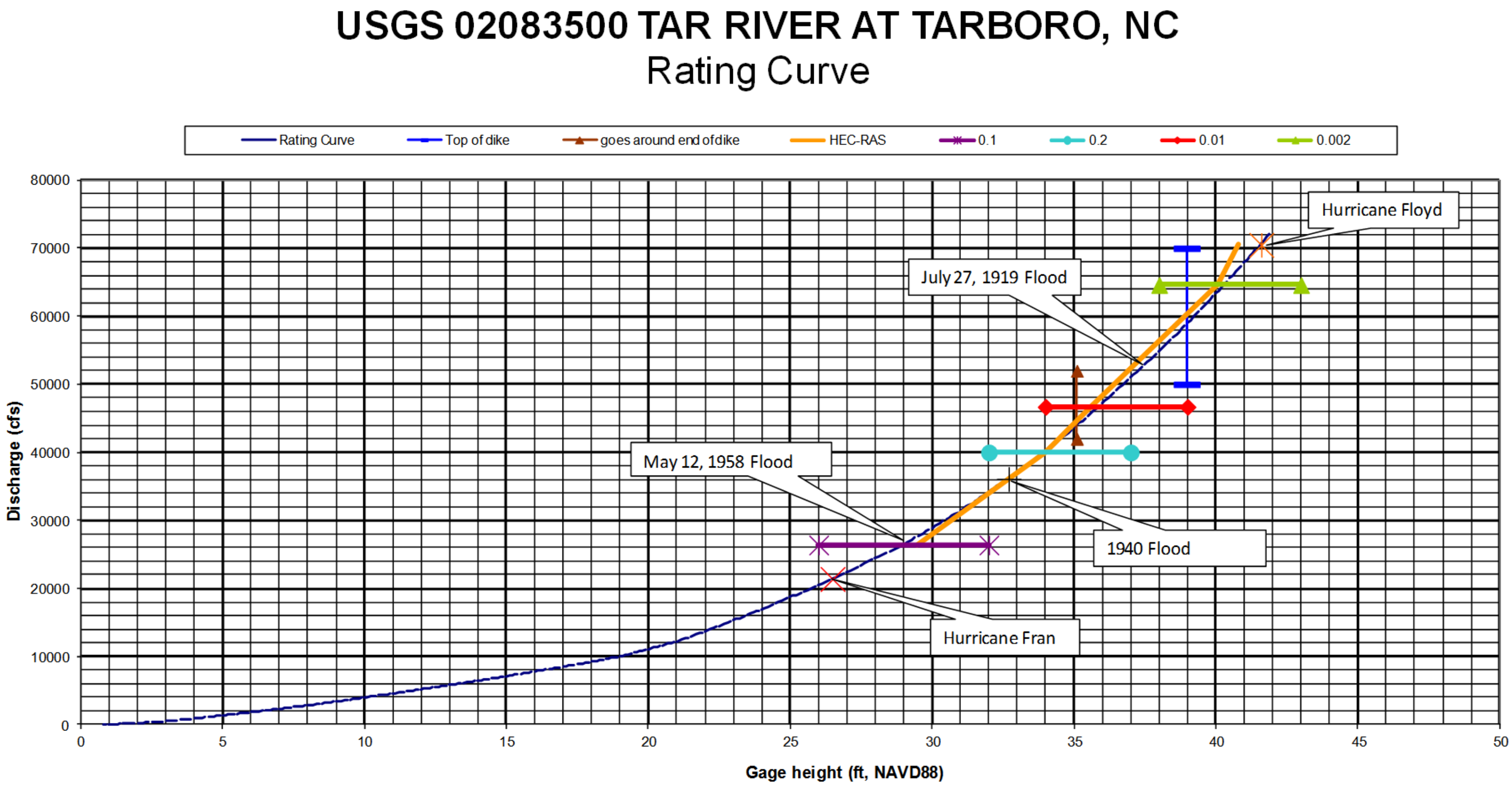

Figure 3-6. Tarboro Gage Rating Curve

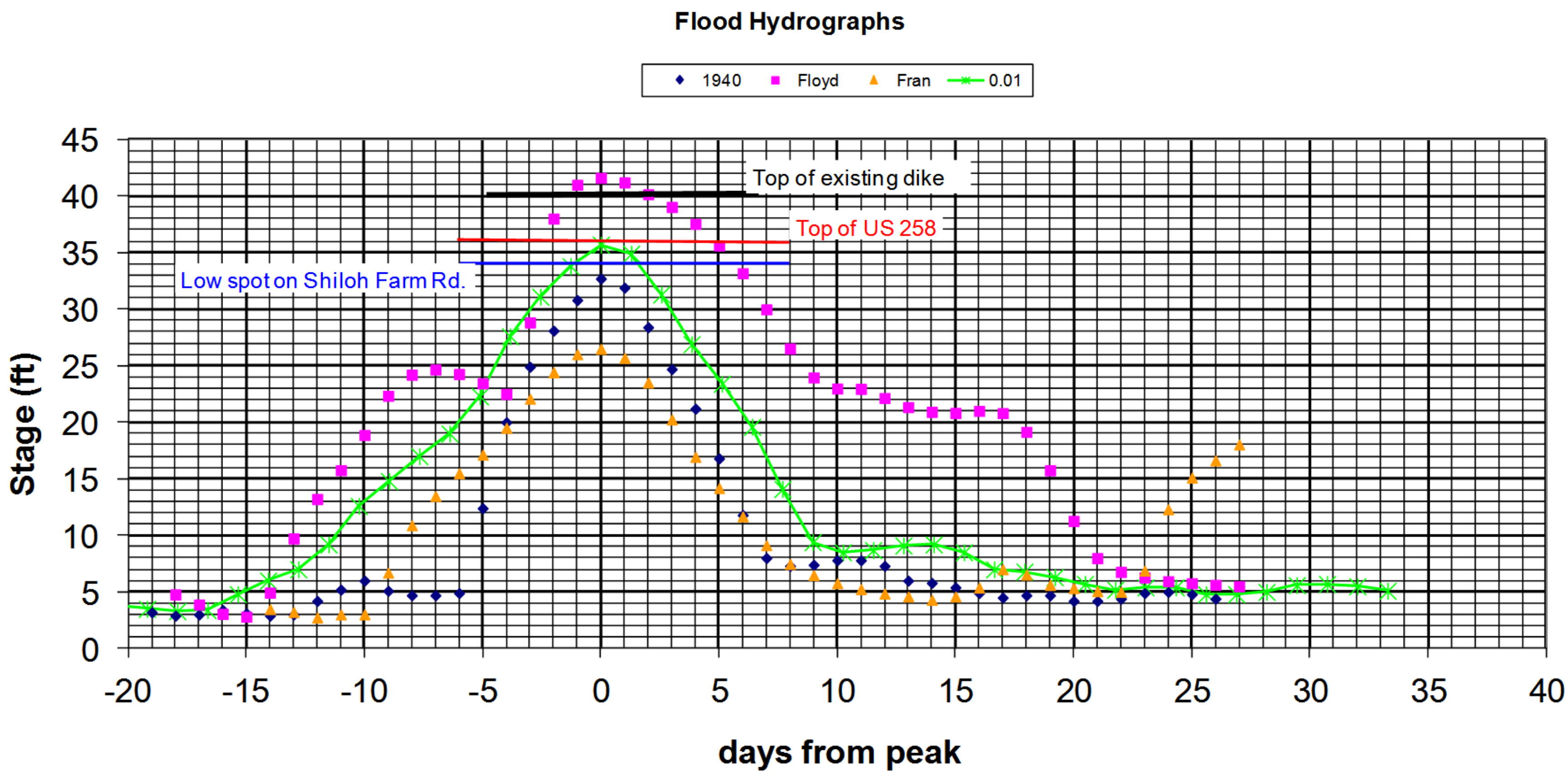

Figure 3-7. Flood Stage Hydrographs

Existing Flood Damage Reduction Features

Levee Alignment and Profile

Exhibit 3-1 shows the existing levee alignment around Princeville. Figure 3-8 shows the levee profile.

Interior Drainage and Culverts

Exhibit 3-3 shows existing culverts within the levee alignment and potential impacts associated with backflow devices. Table 3-1 gives a description and location of each culvert. Backflow devices, or flap gates, are installed on the outlet end of the culvert pipe to prevent water from high river stages backing into the town and flooding areas within the leveed area. As indicated in Exhibit 3-3, there are several existing culverts that do not have flap gates.

In determination of the 0.01 AEP event ponded-water surface elevations inside of the levee, it was assumed that all rainfall-runoff would flow to the lowest point and accumulate. After a study of the topography inside of the levee it was determined that some flooding would occur in the sub-basins before water levels got high enough to flow over the ridges and into the storage areas (see Exhibit 3-2). The shaded areas in Exhibit 3-2 represent areas where water would accumulate prior to flowing into the storage area. Under normal conditions (i.e., normal river stages) the runoff would exit through the outlets through the levee and not accumulate inside of the levee. The ponding problem will be addressed as part of the flood risk management project by connecting the storage area to the outlets.

The existing levee only affects the drainage areas shown in Exhibit 3-2. The remaining drainage areas do not have backflow devices on the outlets, thus water can both enter and leave through these points. The original interior drainage computations were based on storing the 0.01 AEP event 72-hour rainfall (approximately 11 inches). The use of this rainfall appears to be inappropriate since it only takes a river stage produced from a 0.04 AEP event to cause the flap gates to close.

In 2006 the Natural Resource Conservation Service (NRCS) conducted a study of the existing interior drainage system and made several improvements to the drainage system on the west side of Princeville. These changes consisted of ditch modifications and repairing or replacing culverts. This project has helped reduce flooding in the town that resulted from local rainfall.

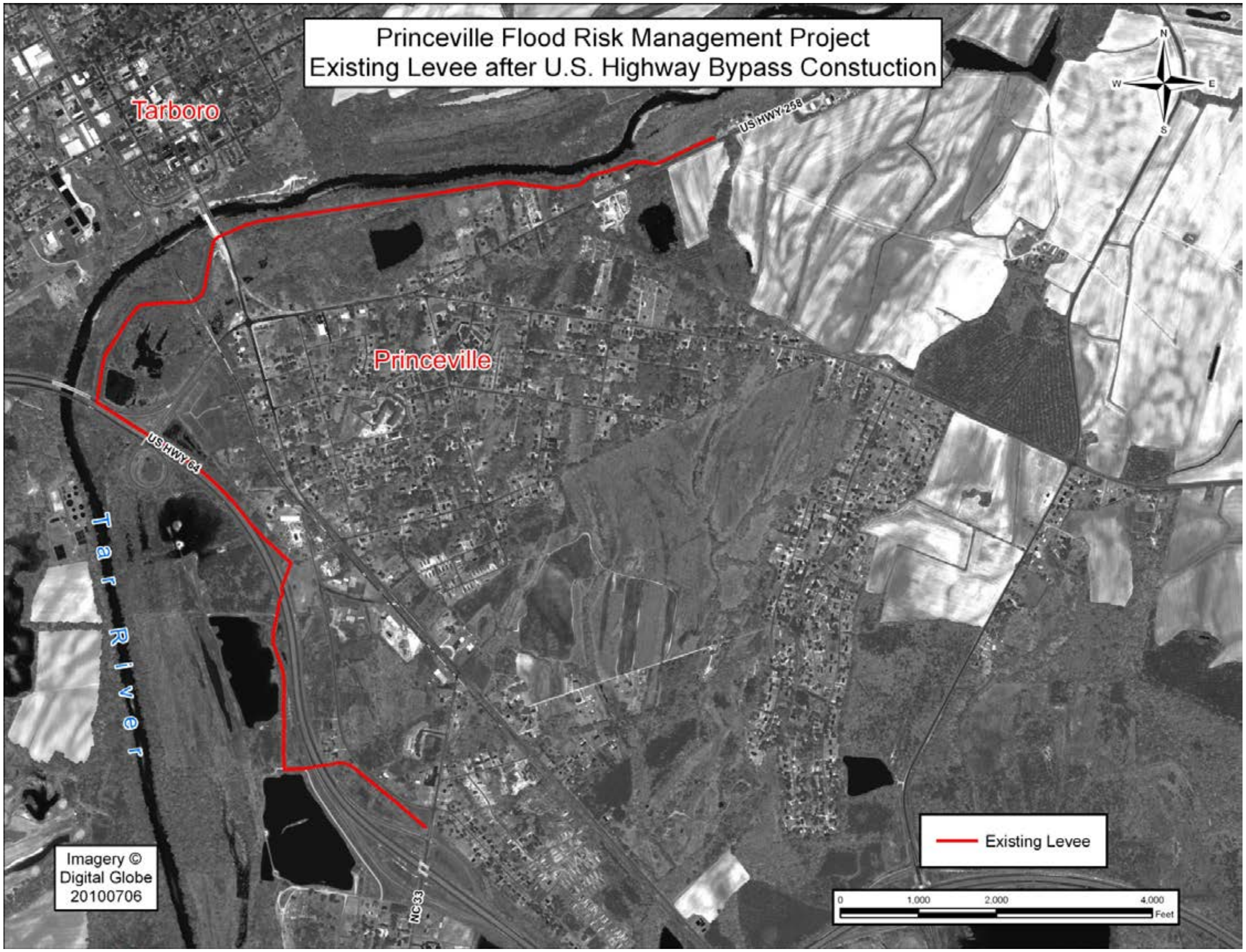

Exhibit 3-1. Current 64 Highway and Levee Alignment

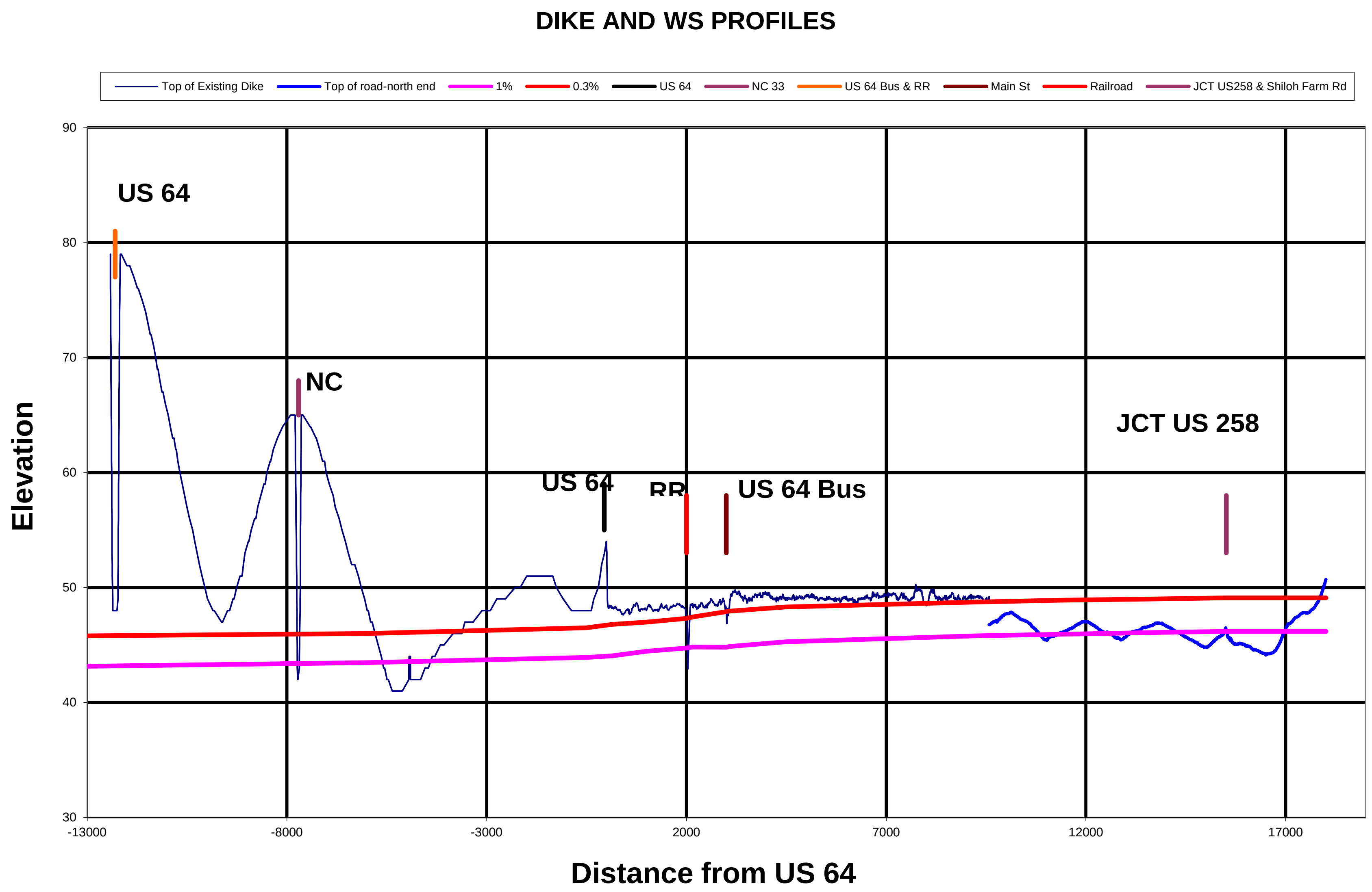

Figure 3-8. Levee and Water Surface Profile

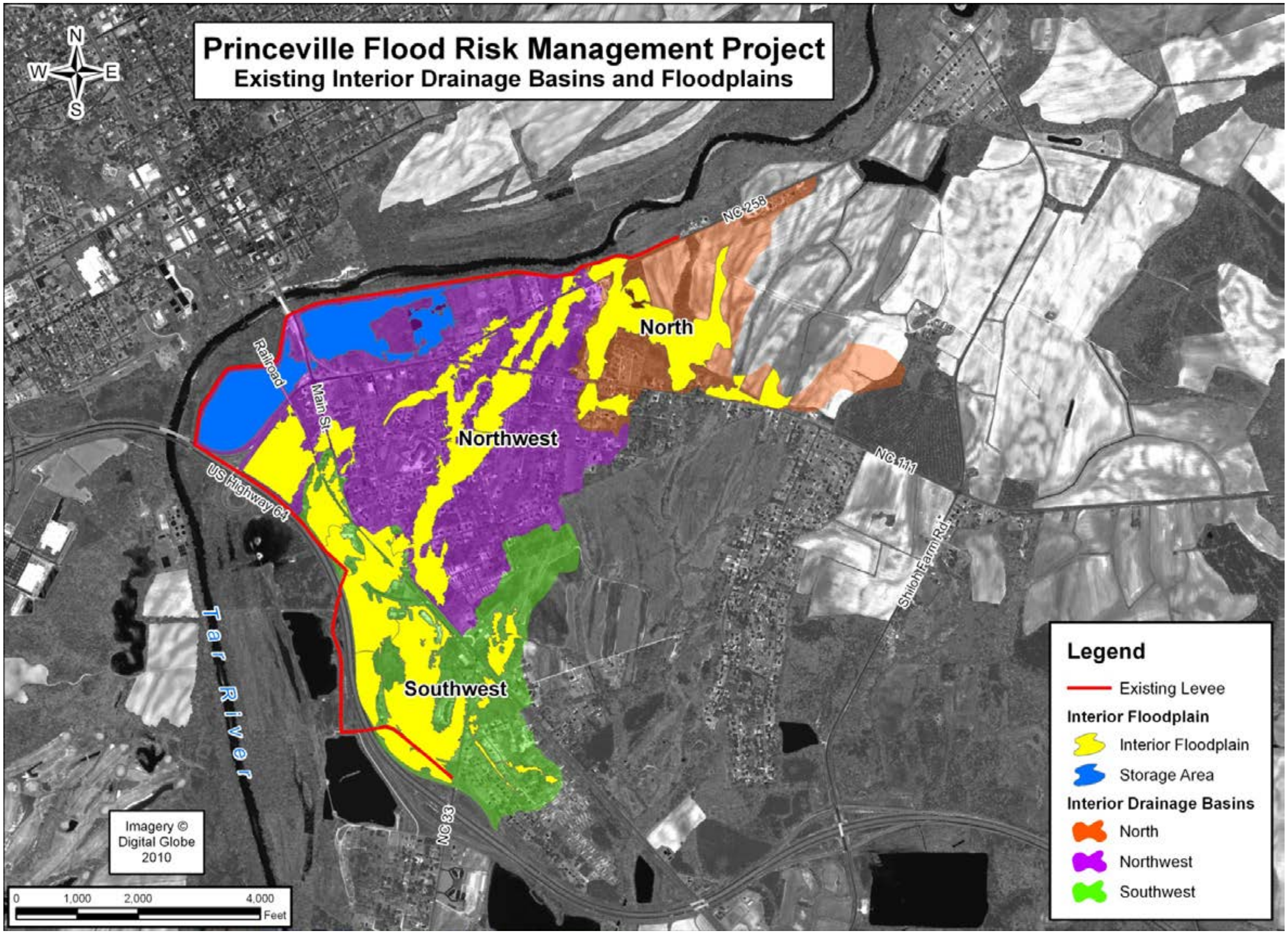

Exhibit 3-2. Interior Drainage

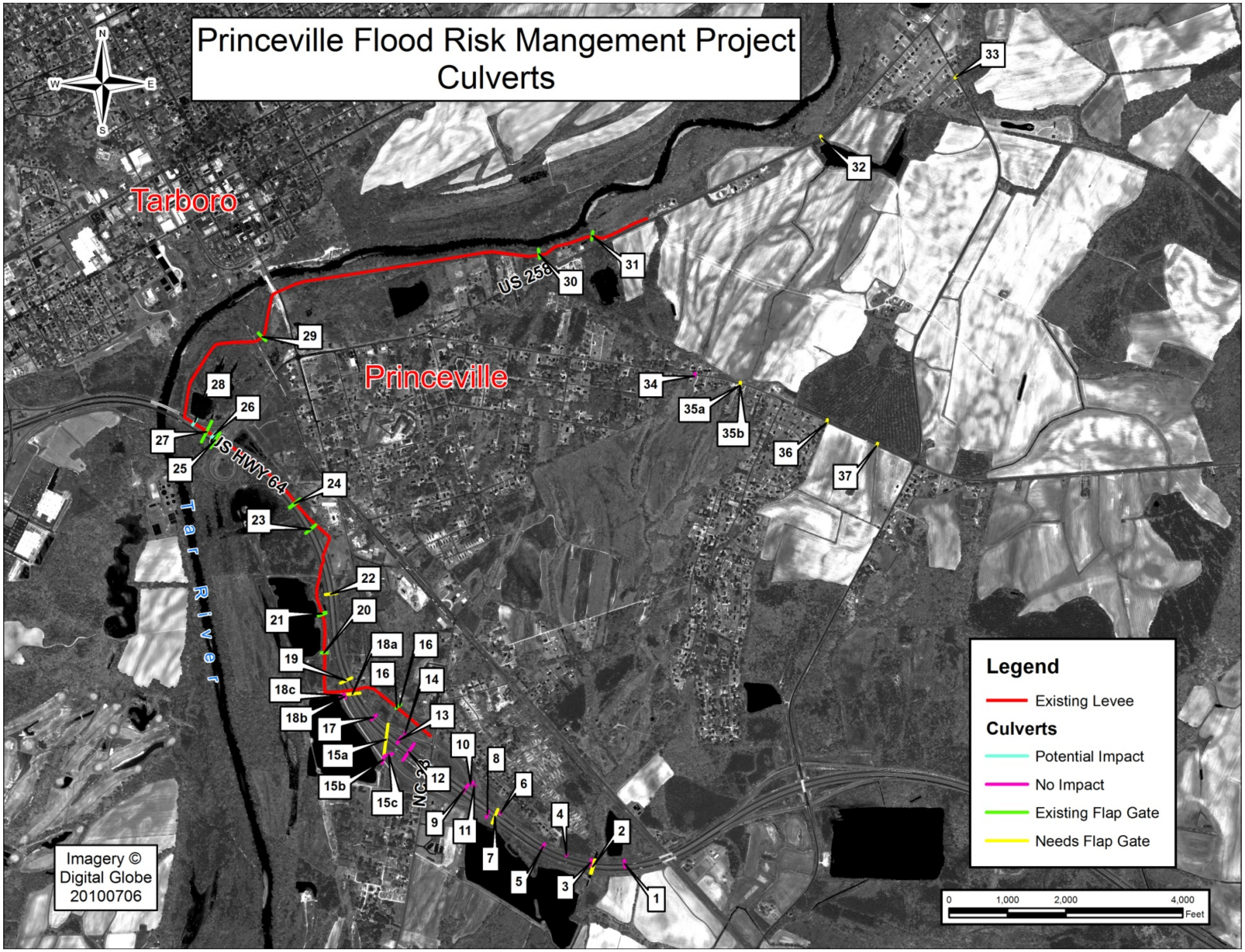

Exhibit 3-3. Existing Culverts

Table 3-1. Drainage Culverts and Descriptions

Pipe Number	Size	Material	Note
Vicinity of US Highway 64 (Segment 1)			
1	15"	CMP	Nothing required for Recommended Plan
2	7'x6'	Triple Box	Needs New Flap Gate
3	15"	CMP	Nothing required for Recommended Plan
4	12"	CMP	Nothing required for Recommended Plan
5	15"	CMP	Nothing required for Recommended Plan
6	15"	CMP	Nothing required for Recommended Plan
7	36"	RCP	Needs New Flap Gate
8	15"	CMP	Nothing required for Recommended Plan
9	30"	CMP	Nothing required for Recommended Plan
10	24"	RCP	Nothing required for Recommended Plan
11	15"	CMP	Nothing required for Recommended Plan
12	24"	RCP	Nothing required for Recommended Plan
Vicinity of US Highway 64 (Segment 2)			
13	15"	CMP	Nothing required for Recommended Plan
14	12"	CMP	Nothing required for Recommended Plan
15a	60"	RCP	Needs New Flap Gate
15b	60"	RCP	Nothing required for Recommended Plan
15c	36"	RCP	Nothing required for Recommended Plan
16	48"	RCP	Part of existing Princeville FRM project
	48"	RCP	Part of existing Princeville FRM project
	60"	RCP	Part of existing Princeville FRM project
17	15"	CMP	No Impact to project
18a	18"	RCP	Needs New Flap Gate
18b	18"	RCP	Nothing required for Recommended Plan
18c	24"	RCP	Nothing required for Recommended Plan
19	30"	RCP	Extend pipe 50-feet, add new flap gate , adjust existing inlet elevation
20	30"	RCP	Part of existing Princeville FRM project
21	48"	RCP	Remove

Pipe Number	Size	Material	Note
	48"	CMP	Remove
	60”	RCP	Construct new (100 LF) with new flap gate
22	54"	RCP	Extend pipe 75-feet, add new flap gate, add box with inlets, adjust existing inlet elevation
23	24"	CMP	Extend pipe 50-feet, add new flap gate, adjust existing inlet elevation
24	24"	RCP	Extend pipe 75-feet, add new flap gate, adjust existing inlet elevation
	36"	RCP	Extend pipe 75-feet, add new flap gate
25	30"	RCP	Part of existing Princeville FRM project and has flap gate
26	12"	CMP	Need rim elevation, may need to re-grade and raise rim
27	30"	CMP	Part of existing Princeville FRM project and has flap gate
28	12"	CMP	Need rim elevation, may need to re-grade and raise rim
Vicinity of U.S. Highway 258 (Segment 3)			
29	60"	CMP	Part of existing Princeville FRM project and has flap gate
30	48"	RCP	Part of existing Princeville FRM project and has flap gate
31	48"	CMP	Part of existing Princeville FRM project and has flap gate
32	4'x4'	Box	Beyond proposed alignment, does not have flap gate
Vicinity of Shiloh Farm Road			
33	48"	CMP	Beyond proposed alignment, does not have flap gate
Vicinity of N.C. Highway 111 (Segment 4)			
34	18”	RCP	Beyond proposed alignment, does not have flap gate
35	2-27”	ERCP	Replace with new 48” RCP and new flap gate
36	18”	RCP	Potential removal to be evaluated during PED
37	15”	RCP	Potential removal to be evaluated during PED

Inundation Maps for Existing Conditions

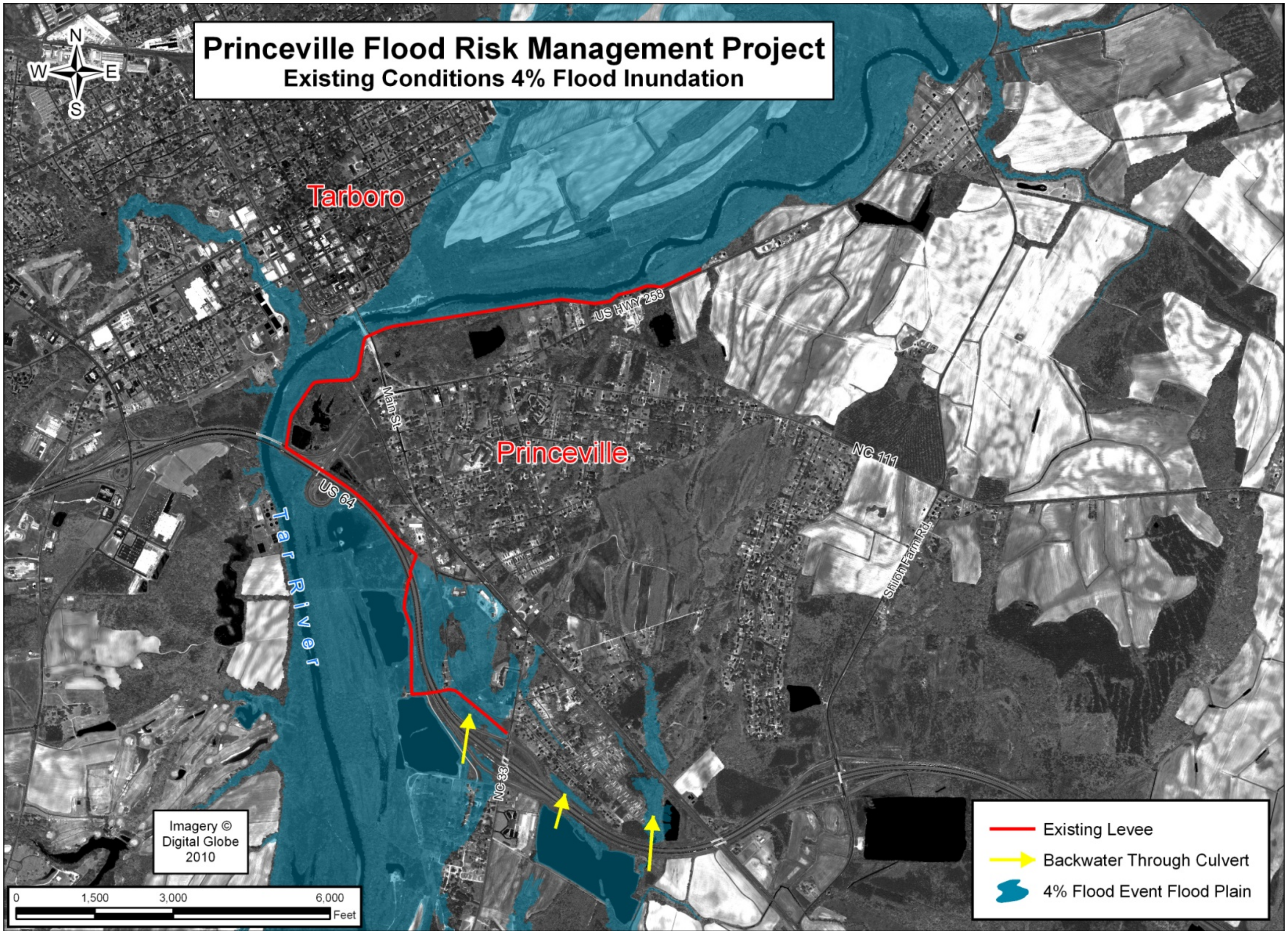

Exhibit 3-4. 4% Flood Event Inundation

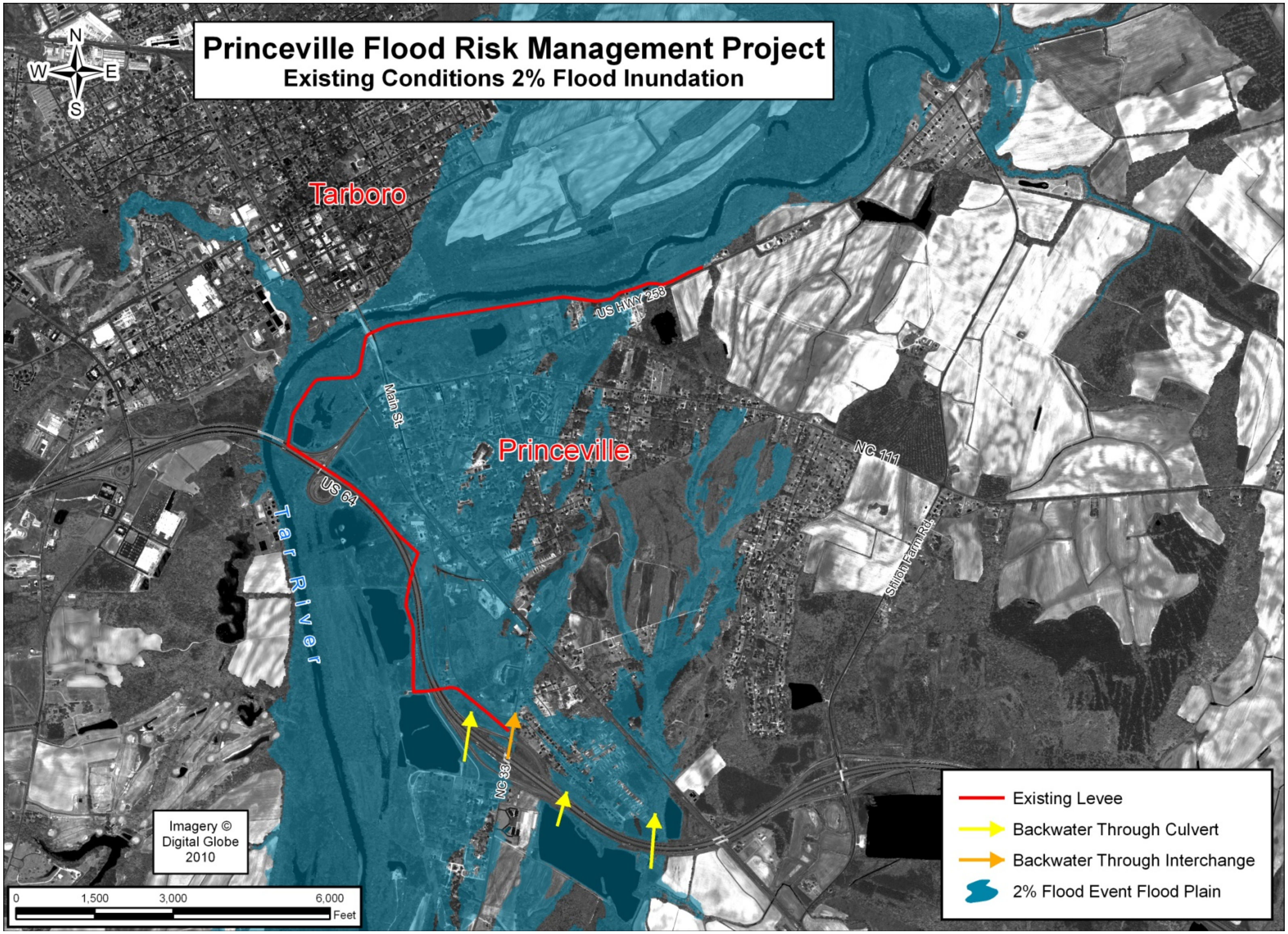

Exhibit 3-5. 2% Flood Event Inundation

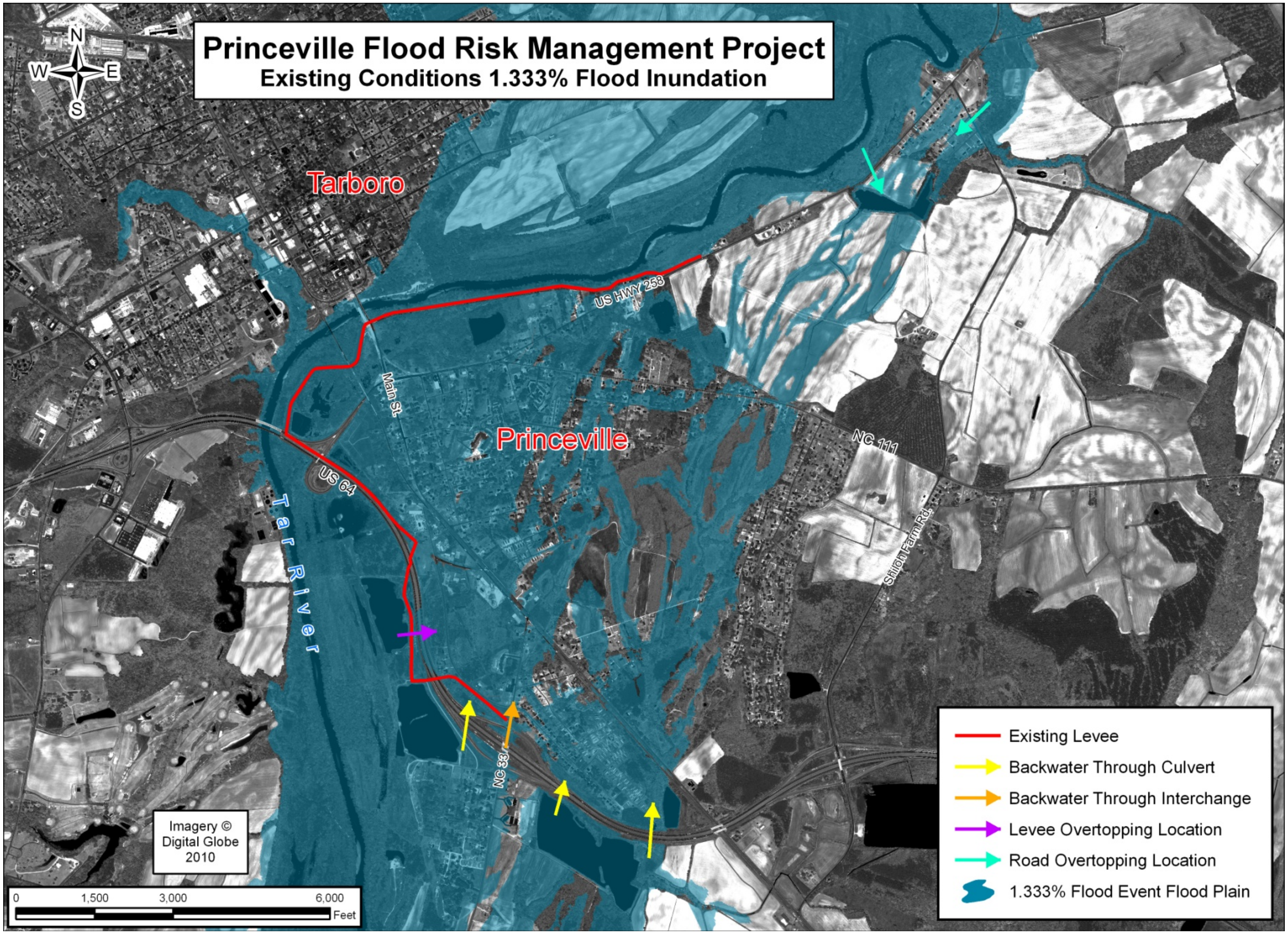

Exhibit 3-6. 1.333% Flood Event Inundation

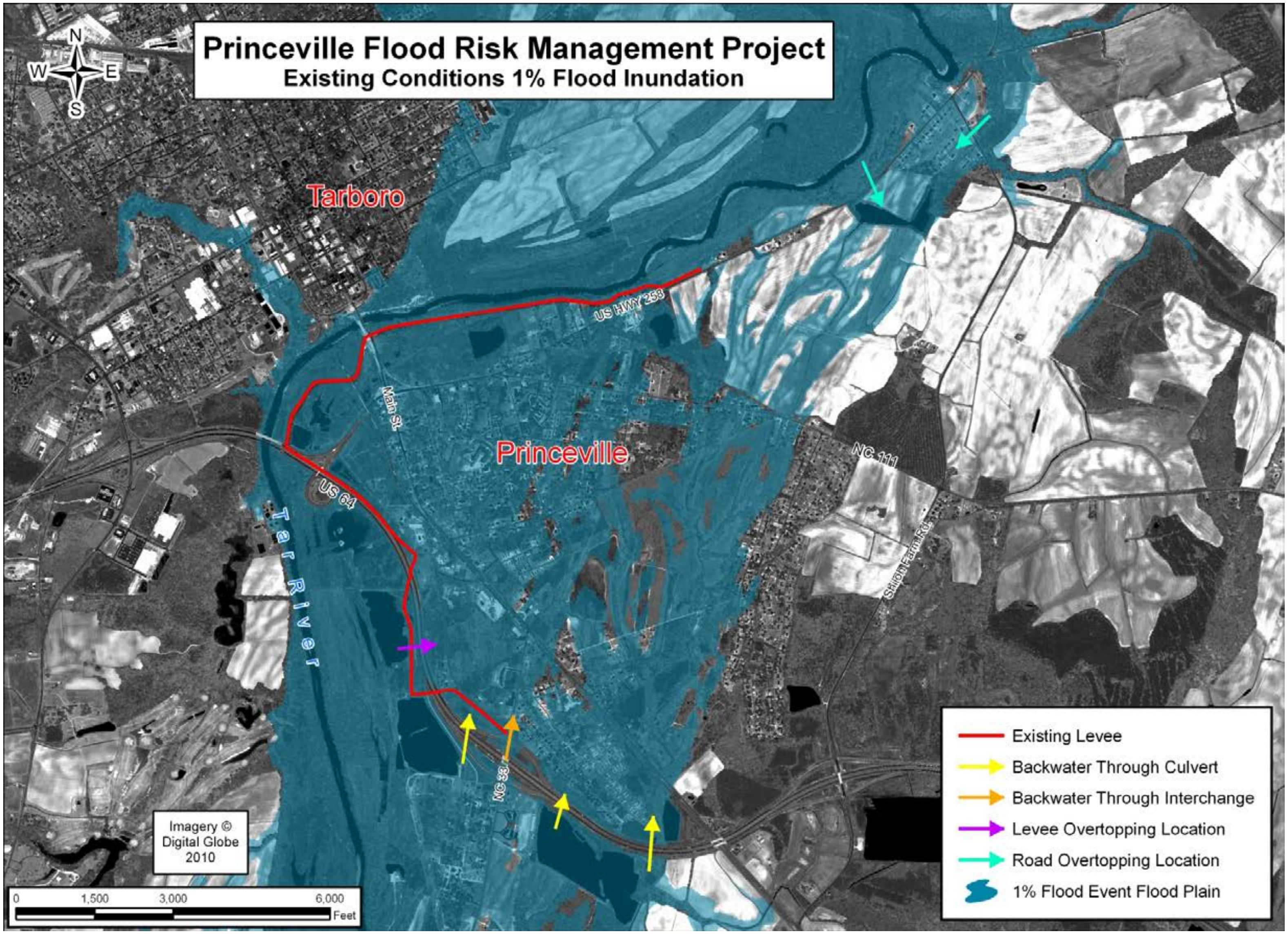

Exhibit 3-7. 1% Flood Event Inundation

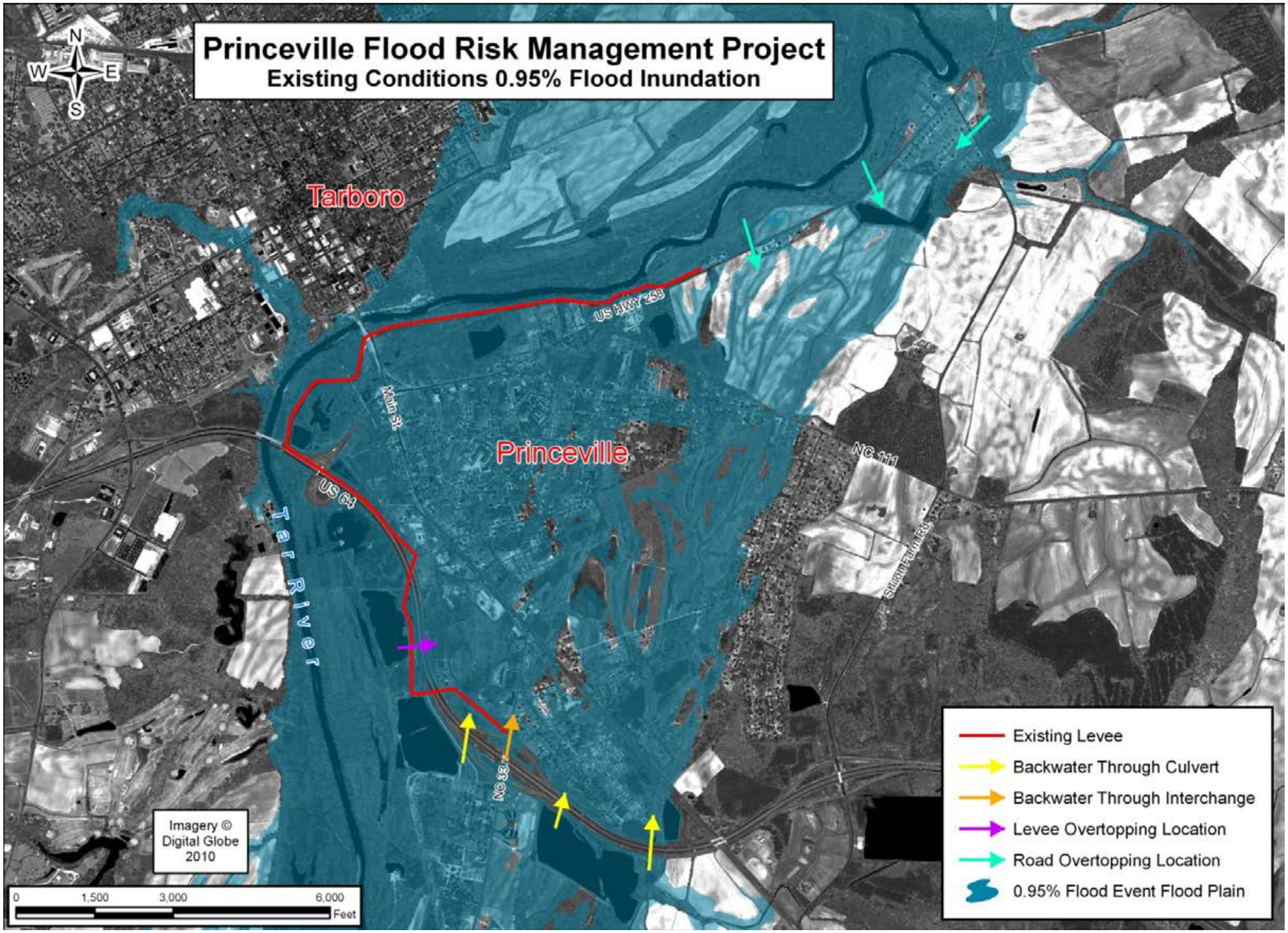

Exhibit 3-8. 0.95% Flood Event Inundation

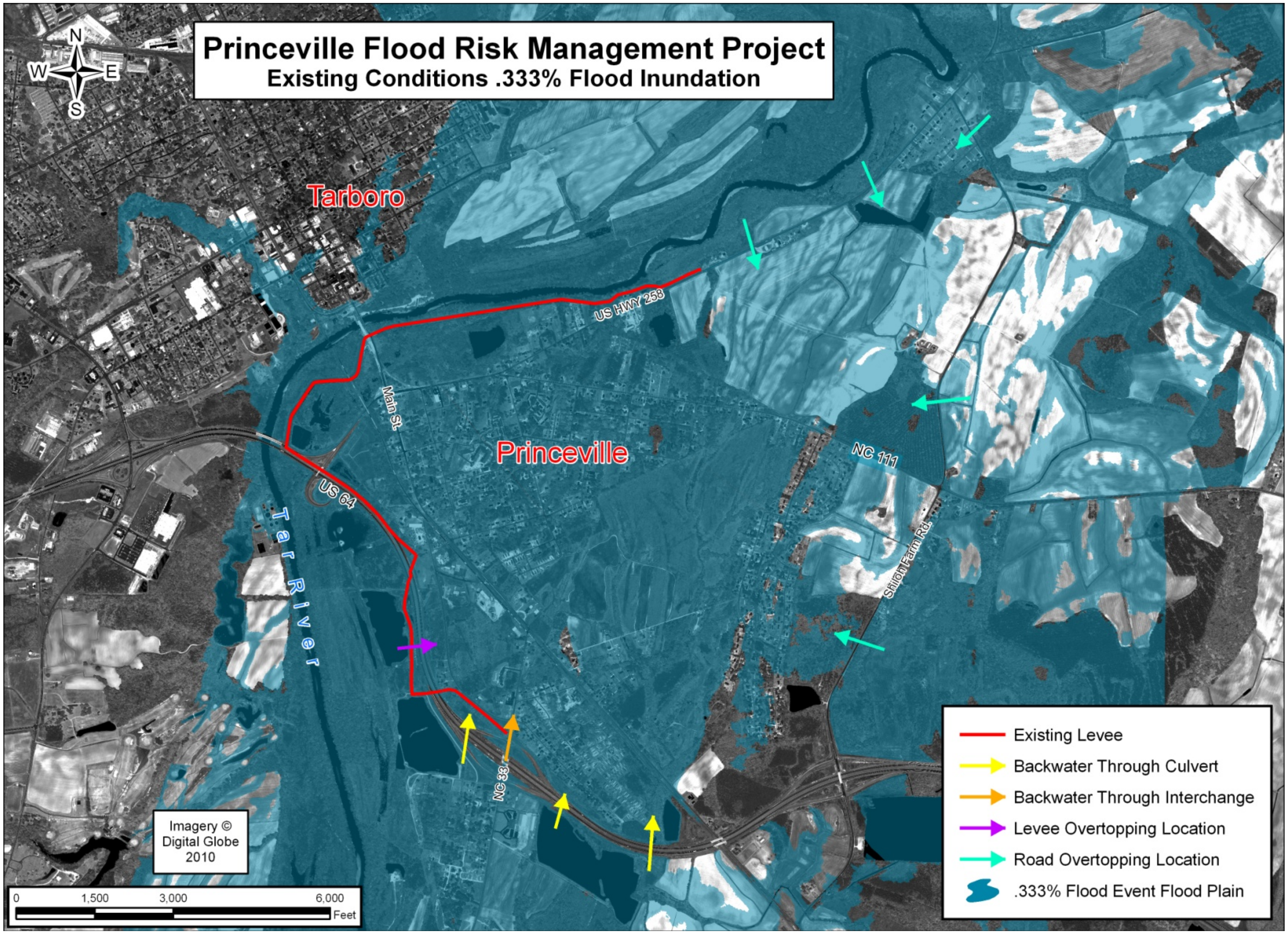

Exhibit 3-9. 0.333% Flood Event Inundation

Chapter 4 **Discharge Frequency Analysis**

Tar River Gage at Tarboro (02083500)

The Tar River gage at Tarboro is located 50 feet downstream from the US 64 bridge in Tarboro, 6.5 miles downstream of the confluence of Fishing Creek and 49.2 miles upstream of the mouth of the Pamlico River. The drainage area at the gage is 2,183 square miles. The period of record for the gage extends from July 1896 to December 1900 and then from October 1931 to the present. The initial gage (July 1896 – December 1900) was a non-recording gage approximately 600 feet downstream of the current gage and referenced to a different datum. The present gage is a water-stage recorder located at 9.32 feet above NAVD 88. Table 4-1 shows the monthly mean discharge for the Tar River Gage at Tarboro for the period of record 1896 – 2009, and Table 4-2 shows summary statistics. The 2004 FIS frequency discharges for Tar River at Tarboro (02083500) are presented in Table 4-3.

Table 4-1. Monthly Mean Discharge Statistics

Statistics of Monthly Mean Discharge (cfs) for Tar River Gage at Tarboro, Water Years 1896 – 2009 by Water Year (WY) (USGS 2009)												
	Oct	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**
Mean	1,116	1,334	2,041	3,213	4,107	4,330	3,192	1,809	1,374	1,266	1,387	1,599
Max	8,896	7,306	6,195	10,020	12,920	11,050	8,553	8,411	6,214	6,291	8,260	26,760
(WY)	(2000)	(2007)	(1949)	(1936)	(1899)	(1989)	(1987)	(1958)	(2006)	(1975)	(1940)	(1999)
Min	56.7	93.9	191	253	497	967	688	344	146	165	142	59.7
(WY)	(1934)	(2008)	(1934)	(1934)	(1934)	(2006)	(1995)	(2002)	(2002)	(2002)	(2008)	(2007)

Table 4-2. Summary of Tar River at Tarboro Gage Statistics

Summary Statistics for Tar River Gage at Tarboro, Discharge in cfs (USGS 2009)			
	CY 2008	**WY 2009**	**WY (1896 – 2009)**
Annual total	413,440	450,926	
Annual mean	1,130	1,235	2,228
Highest annual mean			4,199 (2003)
Lowest annual mean			594 (1981)
Highest daily mean	7,150 (Mar 11)	9,410 (Apr 2)	70,500 (Sep 19, 1999)
Lowest daily mean	57 (Aug 26)	102 (Jul 13)	28 (Oct 17, 2007)
Annual seven-day minimum	90 (Jun 27)	131 (Jul 13)	30 (Oct 13, 2007)
Maximum peak flow		9,670 (Apr 2)	70,600 (Sep 19, 1999
Maximum peak stage		18.57 (Apr 2)	41.51 (Sep 19, 1999)
Instantaneous low flow		70 (Aug 11)	24 (Oct 16, 2007)
Annual runoff (cfsm)	0.517	0.566	1.02
Annual runoff (inches)	7.05	7.68	13.87
10 percent exceeds	2,690	3,070	5,580
50 percent exceeds	670	650	1,220
90 percent exceeds	134	148	270

Table 4-3. Discharge Frequencies for Tar River at Tarboro Gage

Discharge Frequencies for Tar River at Tarboro (2004 FIS)	
AEP	Discharge (cfs)
0.5	14200
0.2	21200
0.1	26400
0.04	34100
0.02	40300
0.01	47100
0.002	65200

USGS Studies

The following reports were used as references for frequency analysis regression methods, annual discharge and stage trends, and overall history of methodology and uncertainty of this study region:

Estimation of Flood-Frequency Characteristics of Small Urban Streams in North Carolina, 1996. Characteristics of both flood-frequency and river basin evaluated to produce estimation equations from regression analysis. These equations included drainage area, impervious area, and rural flood discharge.

Estimating the Magnitude and Frequency of Floods in Rural Basins of North Carolina, 2001. In 2001 USGS published a revision to the 1999 report, Estimation of Flood-Frequency Characteristics of Small Urban Streams in North Carolina Water Resources Investigations Report 99-4114. The revision provided updated flood-discharges for all recurrence intervals that varied by as much as 17% from the 1999 report values.

Magnitude and Frequency of Rural Floods in the Southeastern United States, through 2006: Volume 2, North Carolina, 2009. Annual peak flows recorded at gage stations through the 2006 water year were analyzed to update previous flood-frequency studies.

Discharge-Frequency Relationship

Several methods were considered in establishing an effective discharge frequency for the Tar River reach within project boundaries. The Regional regression equation method is available for this geographic region and should only be used in the absence of nearby flow data. Due to the ideal gage location on the Tar, a general frequency analysis was chosen as the most accurate method. The FIS, Flood Insurance Study Report: Edgecombe County, North Carolina and Incorporated Areas (2004) developed a flood-frequency relationship at the 02083500 gage based on USGS's Estimation of Flood-Frequency Characteristics of Small Urban Streams in North Carolina Water Resources Investigations Report 99-4114 (1999). At this gage, recurrence interval discharges were not weighted with regression equation estimates because the period of record at this gage is sufficiently long as to provide statistical flood frequency analysis that is more representative than the weighted estimate (FIS 2004). Due to the size of the drainage basin at the project site and the limited amount of development in the basin, the rate of change in the frequency-discharge relationship is not significant and thus does not cause a significant drop in risk reduction provided by the proposed project. The change in the frequency - discharge relationship that was created by the USGS Hydrologic study in early 2000's is not a result of changes in the population density in the basin but rather additional data and improved modeling techniques. In addition, enforcement of the Tar River Buffer rules (no significant vegetation disturbance within 50 feet of river banks) will also contribute to maintaining the frequency - discharge relationship. To validate the 2004 FIS relationship, a HEC-SSP Bulletin 17B frequency analysis was performed. As seen in Figure 4-1, there were minor differences due to skew coefficient choice but overall fit well with the FIS analysis. The statistical analysis of this comparison is presented in Table 4-4. Stages correlated to the discharge of the expected AEP were based on the USGS published rating table for the Tarboro gage site. A static +9.32 ft was then added to the stage to get an elevation in NAVD 1988.

An overview of the methodology used in determining recurrence interval discharges for the Tar River is included in the most recent Flood Insurance Study for Edgecombe County, NC. The following text was from the 2011 Preliminary Flood Insurance Study for Edgecombe County:

> Review of station records and the history of flooding in eastern North Carolina indicate that the flood discharge resulting from Hurricane Floyd was not exceeding during the gap in the record at 02083500 Tar River at Tarboro (1900 – '31); therefore the flood frequency analysis for this gage was computed using a historical period of 103 years (1897 –1999).
>
> At gage 02083500, Tar River at Tarboro, recurrence interval discharges were not weighted with regression equation estimates because the period of record at this gage is sufficiently long as to provide a statistical flood frequency analysis that is more representative than the weighted estimate."
>
> Discharge estimates for study reaches on the Tar River were computed by transferring the log-pearson III discharge estimates at USGS gages 02082585 (Tar River at NC 97 at Rocky Mount) and 02083500 (Tar River at Tarboro) to points upstream and downstream of the gages. Discharges at points between the two gages were computed by linear interpolation using the relation between the logs of selected recurrence interval discharge at the gage and the log of the drainage area. Discharges at points downstream of 02083500 were computed by linearly extrapolating the relation between the logs of discharge and drainage area." (FIS 2011).

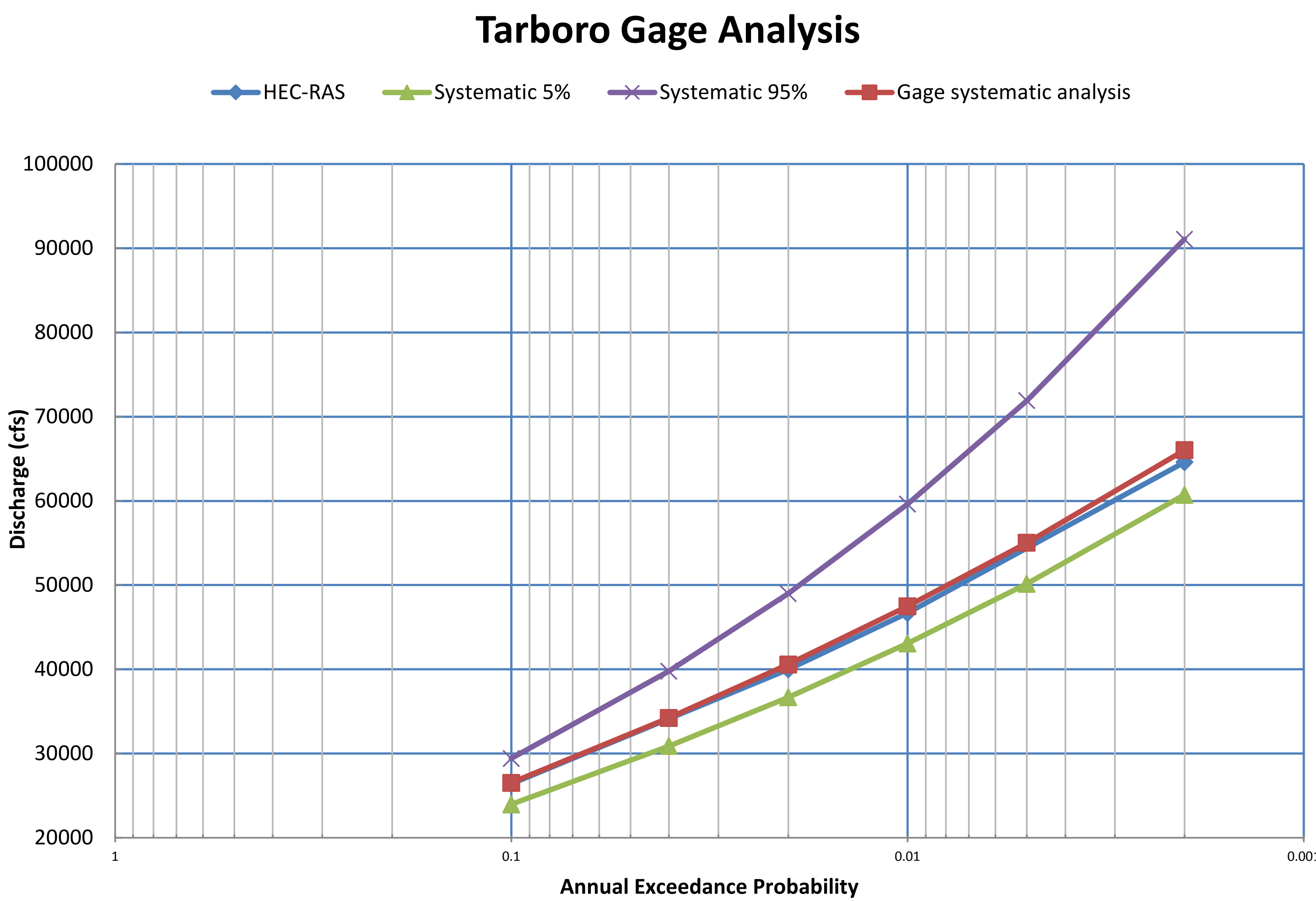

Figure 4-1. Discharge Frequency Curve

Table 4-4. Discharge at Selected Exceedance Probabilities

	ANNUAL FREQUENCY CURVE -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES							
	ANNUAL EXCEEDANCE PROBABILITY	BULL.17B ESTIMATE	SYSTEMATIC RECORD	'EXPECTED PROBABILITY' ESTIMATE	95-PCT CONFIDENCE LIMITS FOR BULL. 17B ESTIMATES LOWER	UPPER	stage ft	elev ft
99.5	0.995	--	4761	--	--	--		
99	0.99	5964	5217	5885	5195	6688	12.00	21.32
95	0.95	7308	6789	7253	6500	8069	14.50	23.82
90	0.9	8255	7875	8213	7428	9041	16.10	25.42
80	0.8	9698	9497	9670	8842	10530	18.30	27.62
66.67	0.6667	11430	11400	11410	10530	12330	20.20	29.52
50	0.5	13770	13920	13770	12780	14830	22.10	31.42
42.92	0.4292	14950	15160	14950	13880	16110	22.90	32.22
20	0.2	20750	21070	20830	19130	22710	26.30	35.62
10	0.1	26340	26520	26560	23970	29420	29.00	38.32
4	0.04	34640	34230	35200	30890	39780	32.00	41.32
2	0.02	41800	40590	42780	36680	48990	34.10	43.42
1	0.01	49850	47500	51480	43080	59630	36.00	45.32
0.5	0.005	58940	55030	61480	50160	71910	38.00	47.32
0.2	0.002	72780	66030	77110	60720	91070	40.50	49.82

Uncertainty with Frequency Analysis

Discharge frequencies from the different USGS reports with respect to the 2004 FIS are shown in Table 4-5.

Table 4-5. Tar River at Tarboro Gage (02083500) Discharge Comparison

	2001	% Diff	2004 FIS	% Diff	2009
Q2 (50%)	13,900	-2.1	14,200	-0.7	14,100
Q5 (20%)	20,300	-4.2	21,200	-0.5	21,100
Q10 (10%)	24,900	-5.7	26,400	0.4	26,500
Q25 (4%)	31,200	-8.5	34,100	0.3	34,200
Q50 (2%)	36,300	-9.3	40,000	1.5	40,600
Q100 (1%)	41,700	-11.5	47100	0.8	47,500
Q200 (0.5%)	47,500	---	N/A	---	54,900
Q500 (0.2%)	55,700	-14.6	65,200	-5.3	61,900

The estimates determined from systematic records for the 2009 study typically are larger in discharge than those from the previous study for the highest percent chance exceedances (50 and 20 percent) and tend to be smaller than those from the previous study for the lower percent chance exceedance (USGS 2009). A contributor to this discharge increase in the 2009 equations appears to be the inclusion of Hurricane Floyd in the gage records of much of southeastern North Carolina.

Uncertainty can also be assessed by the stability of a gage's rating. A stable rating at the gage location implies that the river channel geometry is able to maintain its shape and is largely unaffected by sediment transport, localized erosion, or other factors. A gage that has historically shown signs of an unstable rating should generally not be used in a discharge frequency analysis. Based on correspondence with the North Carolina USGS, the Tar River at Tarboro gage (02083500), which has a period of record from 1896-current, has only had 10 adjustments made to its rating curve. The last rating shift made to the Tar River at Tarboro gage occurred during the 2002 water year. The limited number of rating shifts to this gage indicates that this location should be considered stable and therefore applicable for use in frequency analysis.

Discharge Probability function uncertainty is displayed with confidence limits in Figure 4-2. These limits are curves that interconnect discharge values computed for each exceedance probability according to Appendix 9, Bulletin 17B procedures. Table 4-6 is confidence limits in tabulated form.

Table 4-6. Tabulated Confidence Limits

5% discharge for each AEP	
Exceedance Probability Event	Discharge (cfs)
0.002	81,860
0.005	66,651
0.01	56,497
0.02	47,376
0.04	39,177
0.1	29,555
0.2	23,027
0.5	14,940

95% discharge for each AEP	
Exceedance Probability Event	Discharge (cfs)
0.002	55,751
0.005	47,164
0.01	41,202
0.02	35,647
0.04	30,446
0.1	23,997
0.2	19,301
0.5	12,831

Exceedance Probability Event	Max Discharge
0.002	115,982
0.005	91,019
0.01	74,931
0.02	60,941
0.04	48,799
0.1	35,190
0.2	26,462
0.5	16,485

Exceedance Probability Event	Min Discharges
0.002	46,777
0.005	40,218
0.01	35,578
0.02	31,179
0.04	26,975
0.1	21,611
0.2	17,549
0.5	11,598

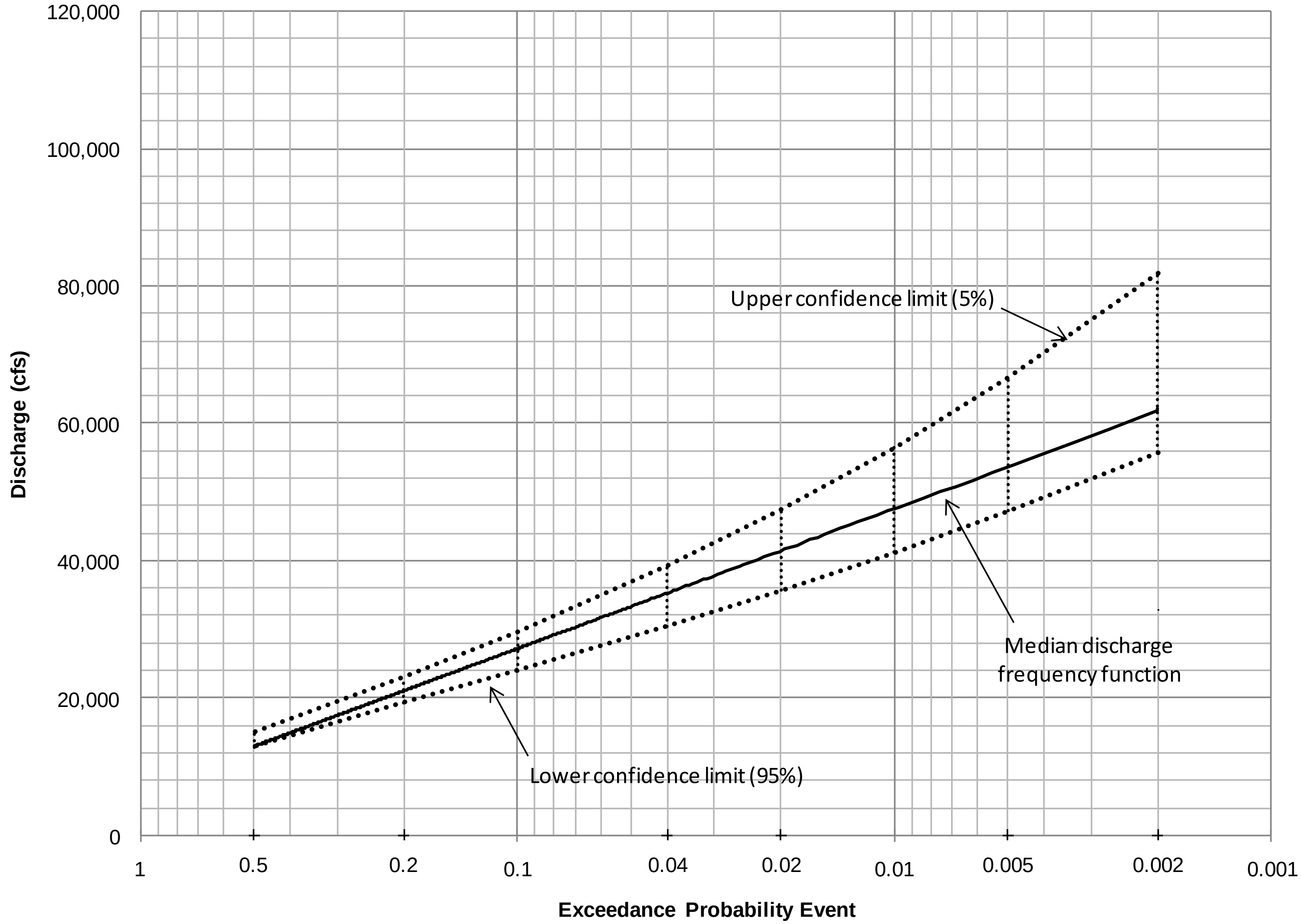

Figure 4-2. Confidence Limits

Many of the gage data measurements were for flows less than 2000 cfs and were excluded from the gage analysis since the study deals primarily with less frequent flood events. Using all of the measurements above 2000 cfs, 250 measurements were analyzed resulting in a standard deviation of 0.94 when compared with the HEC-RAS rating curve. Table 4-7 shows the standard deviations calculated when examined by certain time periods and flow ranges.

Table 4-7. Standard Deviation of Discharges

Time Period	Number of pts	Standard deviation
1900 to 1940	65	1.26
1941 to 1960	59	0.89
1961 to 1980	52	0.70
1981 to 2006	74	1.09
Flow Range	Number of pts	Standard deviation
2,000 to 10,000 cfs	171	0.91
10,000 to 40,000 cfs	77	1.24
Greater than 40,000 cfs	2	0.84

From this table a couple of things may be deduced. First, there is not a significant change in uncertainty over time although there is variation. Secondly, almost all of the measurement points are for discharges less than 40,000 cfs or about a 0.04 exceedance probability flood event. Still, the standard deviations are similar. Assuming the calibration to historic data (primarily the rating curves) was fair, the minimum standard deviation from the EM 1110-2-1619 would be 0.9 feet. If the calibration was considered Poor, the minimum standard deviation would be 1.5 feet.

Chapter 5 **Development of Hydraulic Model**

Existing Flood Insurance Study Model

The Tar River hydraulics for the Princeville flood risk management project utilized a backwater model from an existing Flood Insurance Study. These models were used as a skeleton to build upon the future without and with-project conditions. A 1-dimensional, steady flow HEC-RAS model was developed by a contractor for the State of North Carolina Flood Plain Mapping Office in association with a new Flood Insurance Study in 2004. A similarly modeled reach for the Tar River contained within Pitt County was also used in development of the project model. The original FIS HEC-RAS models for Edgecombe and Pitt Counties were not geo-referenced.

Computed water surface elevations from these models assumed unobstructed flow. “The computed flood elevations are considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail” (FIS, 2011).

Both County models included flood profiles for the 0.1, 0.2, 0.01, and 0.002 AEP events.

Existing Model Topography

The following text was taken from Section 5.0 Engineering Methods in the 2011 Preliminary Flood Insurance Study and is used to describe the existing model geometry.

> The cross section geometries were obtained from a combination of digital elevation data obtained by Light Detection and Ranging (LIDAR) and field surveys. All bridges and inline culverts along the Tar River were field surveyed to obtain elevation data and structural geometry. Natural floodplain cross sections were survey approximately every 4000’ along the Tar River study reach to obtain the channel geometry between bridges. Overbank cross section data for the backwater analyses were obtained from flown LIDAR data in 2001 (FIS, 2011).

The original HEC-RAS model simulates a single reach approximately 50 miles in length covering the portion of the Tar River contained within Edgecombe County. The energy gradient slope was about 0.00009 in the lower 20 miles and about 0.0002 in the upper 30 miles. The Pitt County model contained approximately 31 river miles of the Tar River with a flat energy gradient slope where the Tar River empties into the Pamlico Sound.

Future Without and With-Project Model

A model that met the modeling standards for an FIS was also suitable for development of hydraulic information for the Flood Risk Management Project. This also made the project information compatible with information published in the Edgecombe County Flood Insurance Study. The future without and with-project condition models were geo-referenced. The feasibility model included flood profiles for discharges produced by the 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, 0.002, and Floyd AEP events.

Supplemental Project Model Topography

All inline bridges in the existing FIS models were considered to be accurate and up to the date, with the exception of the Main Street Bridge between the Towns of Princeville and Tarboro. This bridge was replaced in 2010 and as-builts supplied by NCDOT were used for bridge geometry in the future without and with-project models.

All culvert structures passing through the Princeville Levee as part of the original Princeville FDRP were ground surveyed as part of this FRMP. All culvert structures passing through the embankment of Highway 64 within the original project limits and additional culvert structures within the interior of the Town of Princeville were also surveyed. These surveys included inlet and outlet invert elevations, culvert type, size, and length, and photographs depicting current conditions at the inlet and outlet.

The project reach was modeled with a total of 106 cross sections. 31 new cross sections were added within the proximity of the existing Princeville Levee alignment. The bathymetry of the new cross sections was created by linearly interpolating between surveyed cross sections from the existing model. A portion of the model geometry is shown in Figure 5-1.

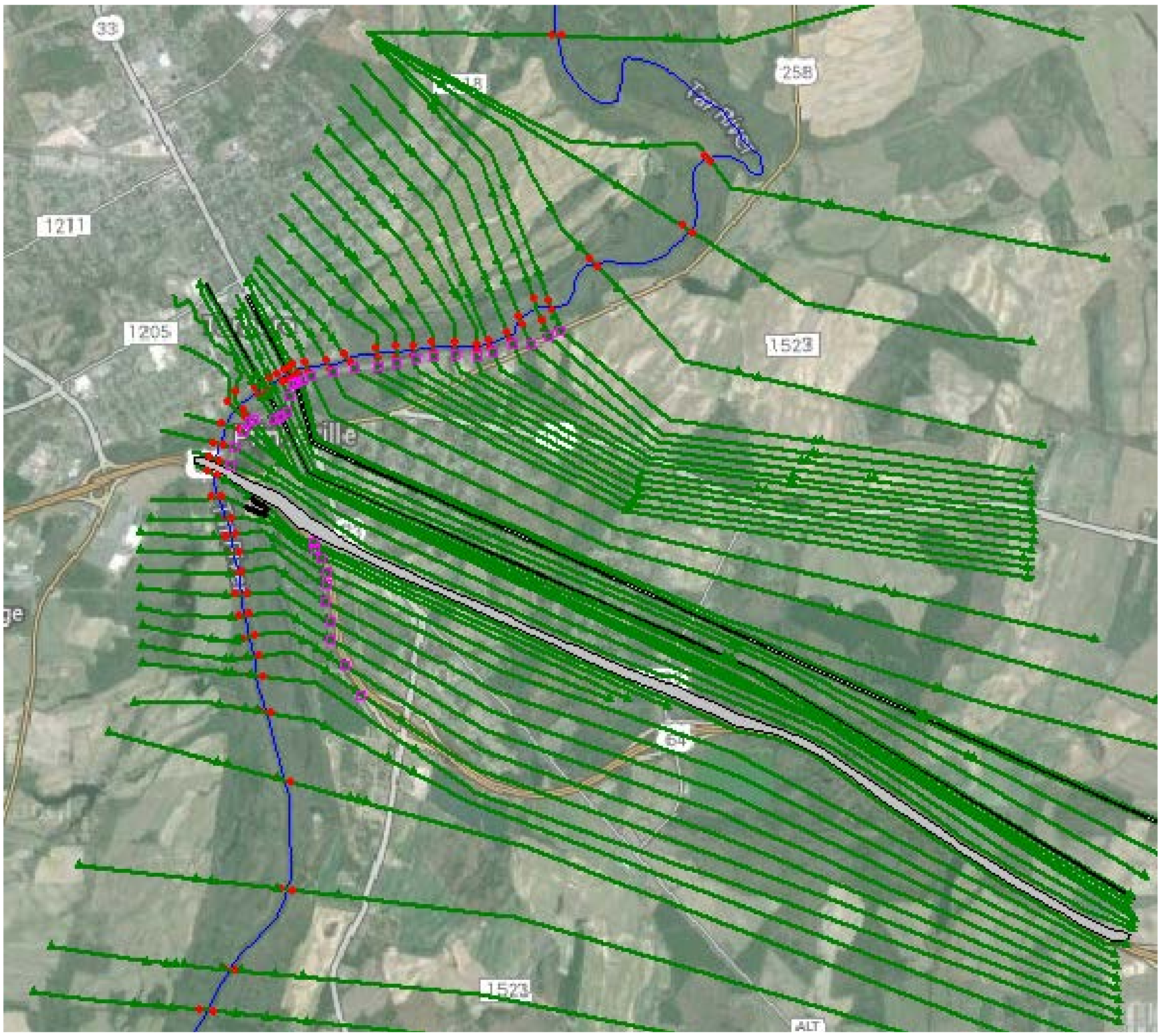

Figure 5-1. HEC-RAS Geometry

The project model contained 10 bridges. For low flow methods the highest energy answer between the Energy (Standard Step) and Momentum was used for all bridges. The coefficient for the force of drag on piers was set to 2.0 for all bridges. For high flow conditions, 4 out of the 10 bridges were modeled using energy only (standard step), the rest were modeled using pressure and/or weir with an assumed submerged inlet + outlet coefficient of 0.8.

Boundary Conditions

A known water surface elevation was used for the downstream boundary condition in the project model for each flow profile. WSELs were set to match the FIS model for Pitt County at the last cross section (FIS cross section 103773) of the project model. A sensitivity analysis was then conducted to ensure the project model extended sufficiently far enough downstream of the project area such that no bias was introduced during model calibration. By deviating the known WSEL within a reasonable range a location of convergence was identified. If this convergence location was downstream of the project area the boundary condition was appropriately placed. Known water surfaces for the Floyd event profile deviating up to +/- 5.0 feet were simulated in the future without conditions model. The convergence location was approximately 6 river miles downstream of the Princeville levee project area. The resulting flood profile is shown in Figure 5-2.

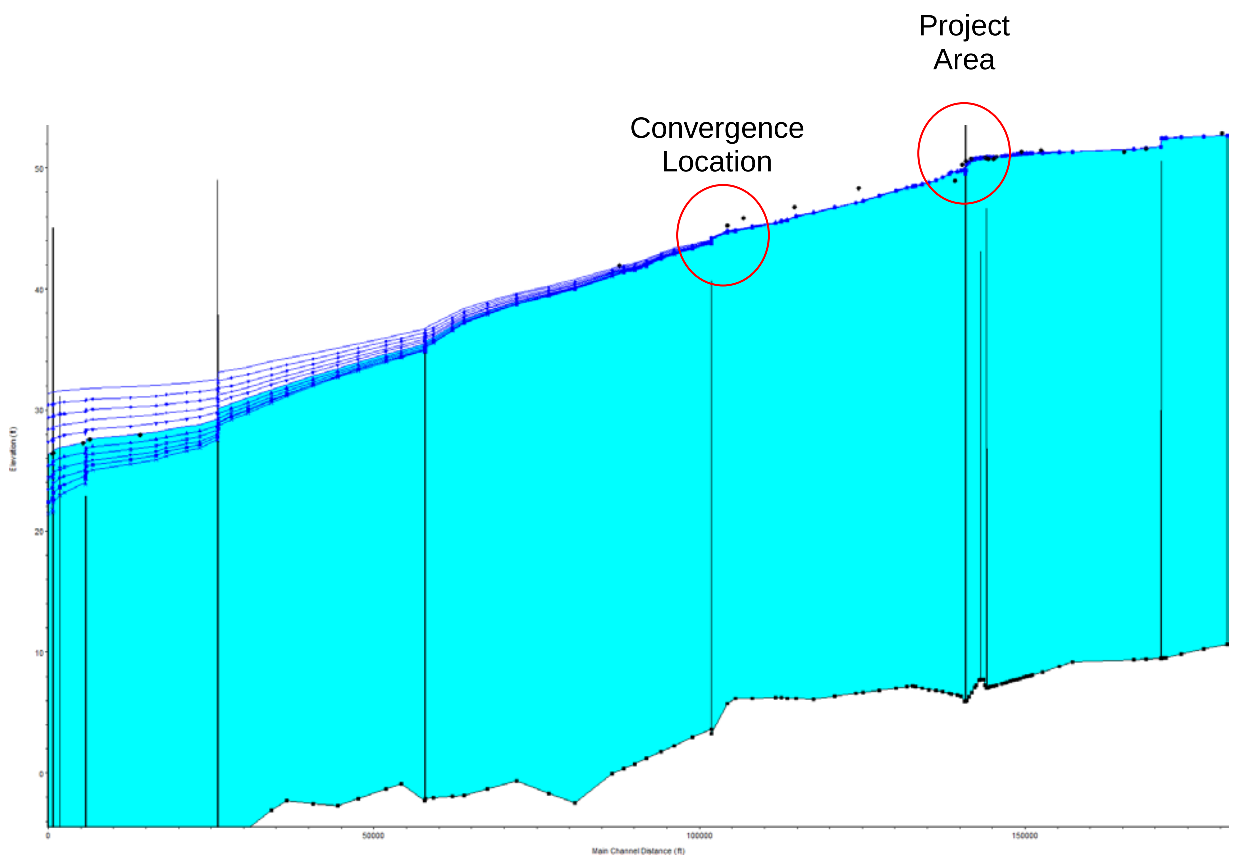

Figure 5-2. Boundary Condition Sensitivity Analysis Profile

Model Levee Feature

The FIS model for Edgecombe County modeled the existing Princeville levee using the Levee feature. When levees are established, no water can go to the landside of the levee station until the levee elevation is exceeded.

Through analysis of the sequence of flooding that occurred during Hurricane Floyd, the levee feature was removed for the future without-project condition model. The first two triggers of flood inundation into the interior of Princeville involved backflow through ungated culverts and flow bypassing the northern end of the existing levee. This sequence meant that Princeville actually began flooding prior to the Tar River overtopping the levee.

For the with-project condition model levee elevations were incrementally adjusted based on the alignment of the selected plan. The overall shape of the Princeville levee embankment was captured well through the processed bare earth LIDAR. The levee crest elevations were adjusted from the LIDAR dataset based on ground surveys taken for the National Levee Database in 2010. Proposed levee extension measures, as well as existing, are represented as an earthen levee in the topography along the cross sections. Figure 5-3 is a typical cross section, shown with the 0.01 AEP event water surface elevations.

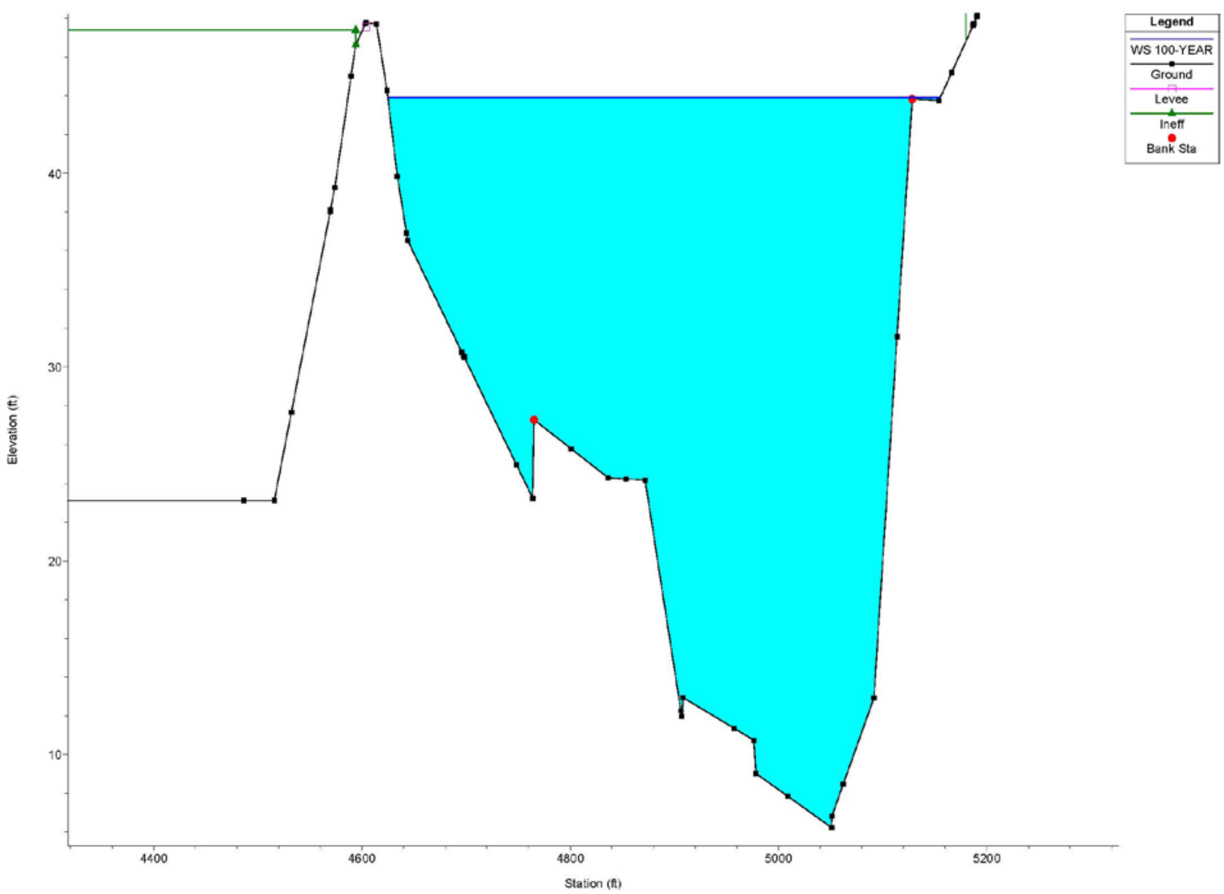

Figure 5-3. Typical Cross Section

Ineffective Flow Areas

Ineffective flow areas were incorporated into the project model due to the lack of relief in the floodplain topography behind the Princeville levee. The levee also served as a barrier to prevent portions of overland flow from returning to the Tar River. Ineffective flow areas were also modeled at upstream and downstream cross sections of bridge structures to account for bridge deck geometry.

Energy Loss Coefficients

According to the HEC-RAS Reference Manual, Manning's n values are very significant to the accuracy of the computed water surface profiles. Adjustments to these values were the primary method of calibrating the model output to observed water surface profile information. Field investigations, photography, and engineering judgment were used to develop initial energy loss coefficients. Manning's n values for the main channel ranged from approximately 0.035 to 0.075. Channel conditions are characteristic of these values with meanders, weedy reaches, and floodways with heavy stands of timber and low lying brush. Manning's n values for the overbank regions ranged from 0.08 to 0.2. Overbank n values are characteristic of dense brush willows in the floodplains. HEC-RAS computed n values were varied transversely for each cross section based on the local anticipated distribution of roughness.

There were significant changes made to the Flood Insurance Study models that resulted in lowering Manning's n values. Both county models used an artificially high Manning's n value of 1.0 for large portions of cross section overbanks. These values may have been used in conjunction with ineffective flow areas to characterize the ponding/obstruction effects of the flat floodplain regions. The future without and with-project condition models replaced these large values with more applicable values in the outer floodplain. These revised values ranged from 0.14 to 0.2.

Typical Contraction and expansion coefficients were adjusted just upstream and downstream of bridge structures. Contraction coefficients were set to 0.1 for gradual transitions and to 0.3 at typical bridge sections. Expansion coefficients were set to 0.3 for gradual transitions and to 0.5 for typical bridge sections.

Calibration of Existing FIS County Models

To a degree the use of existing FIS models meant a certain level of calibration would have already been incorporated into the published models. Calibration of hydraulic models typically consists of a comparison between observed data and model output (water surface elevations). It was discovered that calibration of discharges based on historic high water marks was not done for the detailed study of the Tar River for the Flood Insurance Study. This modeling decision was based on the availability of long-term gage records at the USGS Tar River at Tarboro gage (02083500). Calibration of discharges based on historic high water marks is typically carried out more so for ungaged basins whose discharges were derived from regional regression equations. Generally, the use of regression equations introduces additional uncertainty in the discharge frequency analysis. FIS justification for this modeling approach was as follows:

> Because discharges were computed using long-term gage record for the detailed study reaches of the Tar River, HWM comparison was not used to evaluate reasonableness of discharges. A comparison of computed WSEL's with historic HWM elevations was used to evaluate the hydraulic models used on the Tar River and is included in Attachment 2. Gage data was used to estimate the peak discharge for Hurricane Floyd on the Tar River; WSELs were computed for the Floyd peak discharge and compared to the HWM elevations (Watershed Concepts, 2002).

The FIS model for Edgecombe County included 6 observed high water marks from Hurricane Floyd. Comparison of FIS computed WSEL to observed high water marks from Hurricane Floyd is shown in Table 5-1.

Table 5-1. FIS Computed WSEL vs. Hurricane Floyd High Water Marks

FIS HEC-RAS Model - Edgecombe County

River	Station	Equivalent Feasibility Model Station	FLOYD (ft)	Computed WSEL (ft)
Tar	269891	169267	51.6	52.0
Tar	245644	144946	50.8	50.2
Tar	219544	118914	46.7	45.8
Tar	210220	109590	45.8	44.3
Tar	205613	104846	44.9	43.4
Tar	190400	89769	41.9	40.9

On average it appeared the Edgecombe County model under-predicted resulting water surfaces produced by Hurricane Floyd discharges when compared to the observed high water marks. Several factors may have contributed to higher water surfaces downstream of the Princeville area than to those computed. As stated above in earlier sections FIS models do not account for obstructions at hydraulic structures. Due to the extensive flooding from Hurricane Floyd it is not unrealistic to assume some blockages occurred along the series of bridges on the Tar River in the model reach. These obstruction assumptions were tested during the calibration of the future without and with-project condition models. Obstructions at bridges were only assumed to exist during the Hurricane Floyd event. All other AEP's assumed no obstructions. Another factor for the under-predicted Hurricane Floyd water surfaces in the lower end of the FIS model may have been possible effects of wind tide flooding from the Pamlico Sound that were not captured in the model.

The FIS model for Pitt County included neither Hurricane Floyd event discharges nor observed high water marks measured from this event. Therefore model calibration to Hurricane Floyd was not carried in the FIS Pitt County model.

Calibration of Future Without-Project Conditions Model

Calibration of the future without and with-project condition models was performed over the entire river reach, which covered Edgecombe and Pitt Counties. The future without-project conditions model included the same observed high water marks as those in the FIS Edgecombe County model. Supplemental observed high water marks in the vicinity of the Tar River were also included, which made a total of 19. A number of these additional marks were well away from the River channel. The comparison of the energy grade was more appropriate for calibration purposes in these situations. 5 observed high water marks from Hurricane Floyd were used in calibration of the lower end of the feasibility model in Pitt County. A Comparison of Feasibility model WSELs to Hurricane Floyd observed high water marks is shown in Table 5-2. The "Dn Dist" column is the downstream distance from the current station and was used to further refine the location of the observed high water mark.

Table 5-2. Feasibility Model Computed WSEL vs. Hurricane Floyd High Water Marks

Future Without-Project Conditions Model

River	Station	Equivalent FIS Model Station	Dn Dist (ft)	FLOYD (ft)	Computed WSEL (ft)
Tar	181360	281986	800	52.83	52.67
Tar	169267	269891	0	51.6	51.63
Tar	167309	267935	1500	51.33	51.52
Tar	153347	253976	300	51.43	51.26
Tar	150375	N/A	300	51.33	51.16
Tar	146458	246458	0	50.93	50.93
Tar	145958	N/A	0	50.76	50.93
Tar	145182	245728	0	50.73	50.9
Tar	144946	245644	0	50.8	50.87
Tar	144744	N/A	0	50.83	50.86
Tar	142997	243633	100	50.73	50.61
Tar	142159	N/A	50	50.53	50.35
Tar	141831	242436	300	50.23	49.83
Tar	140680	N/A	250	48.93	49.68
Tar	129228	N/A	750	48.33	47.18
Tar	118914	219544	300	46.73	45.95
Tar	112157	210220	1300	45.83	45.03
Tar	108317	205613	0	45.25	44.77
Tar	91553	190400	700	41.93	41.44
Tar	19727	120353	2500	27.95	28.02
Tar	9971	110600	450	27.55	27.63
Tar	9437	109570	450	27.25	27.28
Tar	3960	104517	0	26.45	26.7
Tar	3694	104467	0	26.35	26.52

As noted in the calibration of the FIS Edgecombe County model, a steeper water surface slope is seen immediately downstream of the Highway 64 bridge structure. These lower WSELs when compared to observed water marks extended approximately 6 river miles downstream to the North Carolina Route 42 bridge structure. This drop was somewhat expect when compared to the FIS model due to lowering the Manning's n values of the outer floodplain from 1.0 to near .2. Approximately 3 river miles downstream of the NC 42 Bridge computed WSELs began to more closely match the observed high water marks for the remaining length of the model.

With-Project Conditions Model

The future without-project conditions model was modified by elevating and extended the existing Princeville levee per the selected plan alignment. The Levee Feature within HEC-RAS was used along with ineffective flow areas to depict the Town of Princeville staying dry up to the top of the proposed levee crest elevation. Resulting water surface elevations for the with-project conditions 1-D, steady flow model showed a nearly identical flood profile to that of the future without conditions model.

Exterior–Interior Relationship

As required in HEC-FDA, an exterior-interior relationship must be explicitly stated otherwise it is assumed the river and floodplain fill at an equivalent rate. This assumption may lead to overestimating inundation behind the levee when compared to the river WSEL. An exterior-interior relationship was developed for both future without and with-project conditions. It was determined through analysis of how Hurricane Floyd sequentially inundated the Town of Princeville that the assumption of an equivalent rate of rise in the exterior-interior relationship was inappropriate.

An exterior-interior relationship for the future without-project condition was developed by modeling Princeville as an elevation-volume curve storage area within HEC-RAS. A lateral structure that included all ungated culverts was created to connect the Tar River channel to the storage area depicting the Town of Princeville. The hydrograph of Hurricane Floyd was then routed through the model. The exterior-interior relationship curve for the future without-project conditions is shown in Figure 5-4. The curve validated the assumption that flood waters were already inundating the interior of Princeville prior to the levee being overtopped.

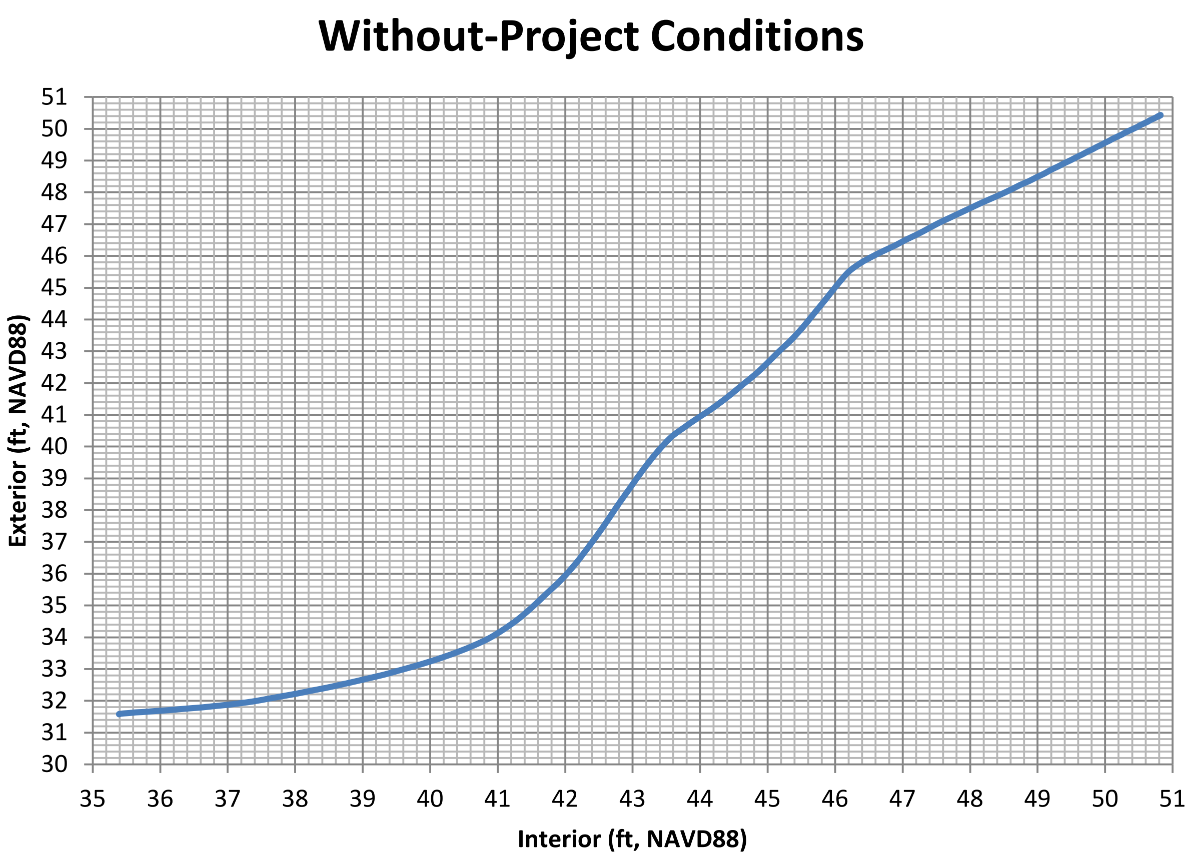

Figure 5-4. Exterior-Interior Relationship for Future Without-Project Conditions

The with-project conditions exterior-interior relationship was developed by elevating the lateral structure that connected the Tar River channel and the interior of Princeville to the selected plan design elevation. All previously ungated culverts were assumed to have flap gates installed that prevent positive flow into the town of Princeville. The curve, shown in Figure Figure 5-5, shows the levee acting as designed with the interior of Princeville not starting to fill until the levee is overtopped.

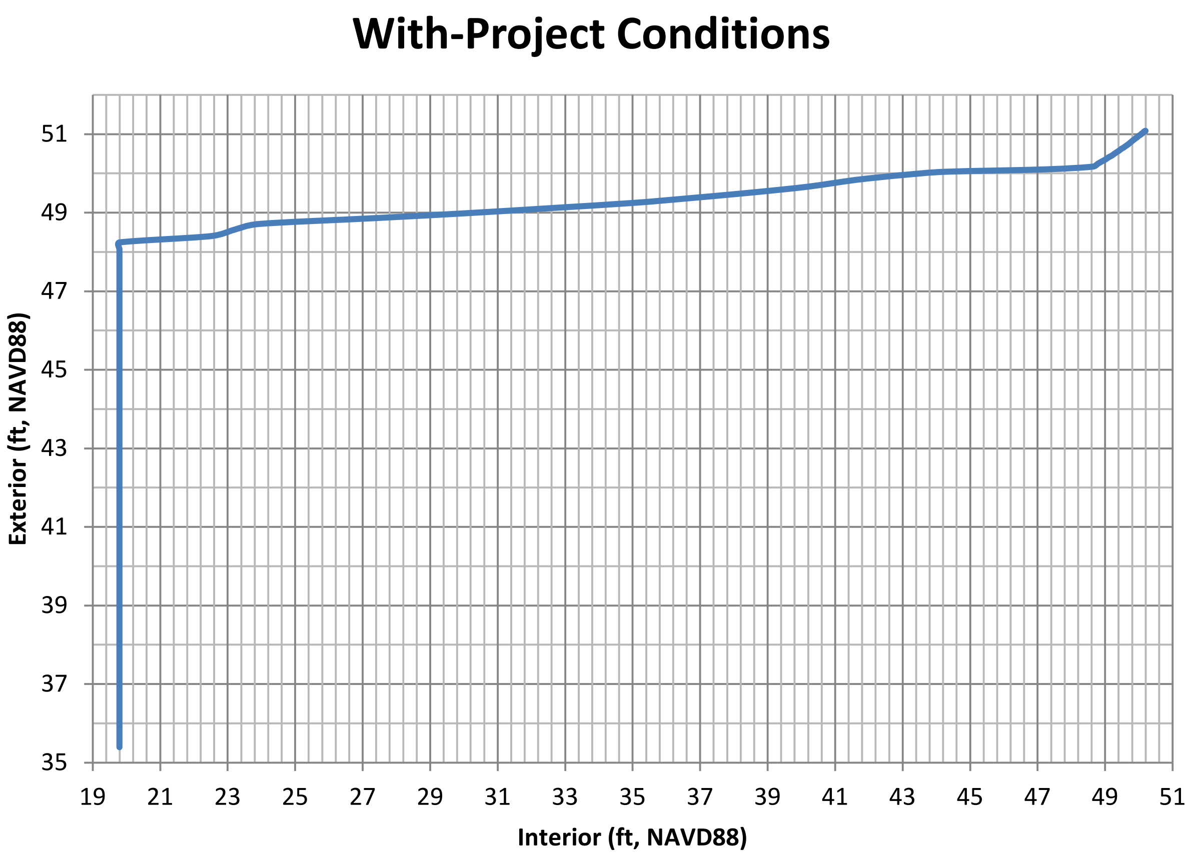

Figure 5-5. Exterior-Interior Relationship for With-Project Conditions

Chapter 6 **Flood Damage Reduction Measures**

The measures detailed in this chapter were initially screened from a larger list of measures; refer to Section 5 of the Main Report for more information on this master list. The following measures were selected due to the ability to adequately analyze their hydrologic and/or hydraulic effects by utilizing the project model discussed in Chapter 5. The objective of any evaluated measure is to provide flood risk reduction achieved by one of the following methods: **(1)** Reduce the level of the flood waters by enlargement of the channel, **(2)** by diversion, or temporary storage of excess water; **(3)** to prevent the water from entering the structure by construction of a barrier; **(4)** to remove the structures from the flood area by elevation, relocation, or by providing flood warning and evacuation information so that the inhabitants can evacuate safely; and **(5)** flood risk information and communication so that residual damages can be mitigated.

The following is a discussion on each of the evaluated measures as to why the measure did or did not accomplish the objective of providing an acceptable level of flood risk reduction. One of the major concerns of any action in the river flood plain is its impact on the flood elevation, especially if there is a possibility of increasing flood elevations on other parts of the flood plain. The objective, for evaluating flood reduction measures, is to ensure that there would not be a significant increase, no greater than 0.1 foot, in the 0.01 AEP event.

Management Measures/Alternatives

Backflow Devices

There are culverts located under US 64 on the north and south sides of Princeville that, as part of this project, would require backflow devices. Currently, the culverts are located such that flooding on the north side of US 64 will begin at roughly the 0.04 AEP event. Flooding starts to backflow through culvert south of Princeville at the approximate 0.03 AEP event. This alternative addresses the flooding that occurs at the 0.04 AEP event by preventing backflow through the culverts into the town of Princeville. There is a total of 12 ungated culverts along the perimeter of the Princeville area that need flap gates in order to provide protection against backwater flooding. Refer to Exhibit 3-3 for the approximate location of pipes needing backflow devices. This alternative will be required regardless of which levee extension is selected. Exhibit 6-1 indicates the extent of the flooding by the 0.04 and 0.01333 AEP events. The area flooded by the 0.01333 AEP event on the north side is due to a culvert (no backflow device) under Shiloh Farm Rd. and the low area over the culvert. This area would not allow enough water to cross the road on the 0.01333 or the 0.01 AEP events to permit the water to reach Princeville in a large enough quantity to produce extensive flooding, generally sheet flow. Exhibit 6-1 also indicates that most of the initial flooding would come from the culverts on the south side, under US 64. The beginning flood elevation may not be controlled by the pipe invert elevation. High ground exists between a number of these culverts and the Tar River.

The alternative levee alignment “I” crosses over an existing culvert which handles discharge under route NC 111 (Greenwood Blvd). Shown in Exhibit 6-2, this 27-inch double-barrel reinforced concrete pipe drains to the north and would require a backflow prevention device.

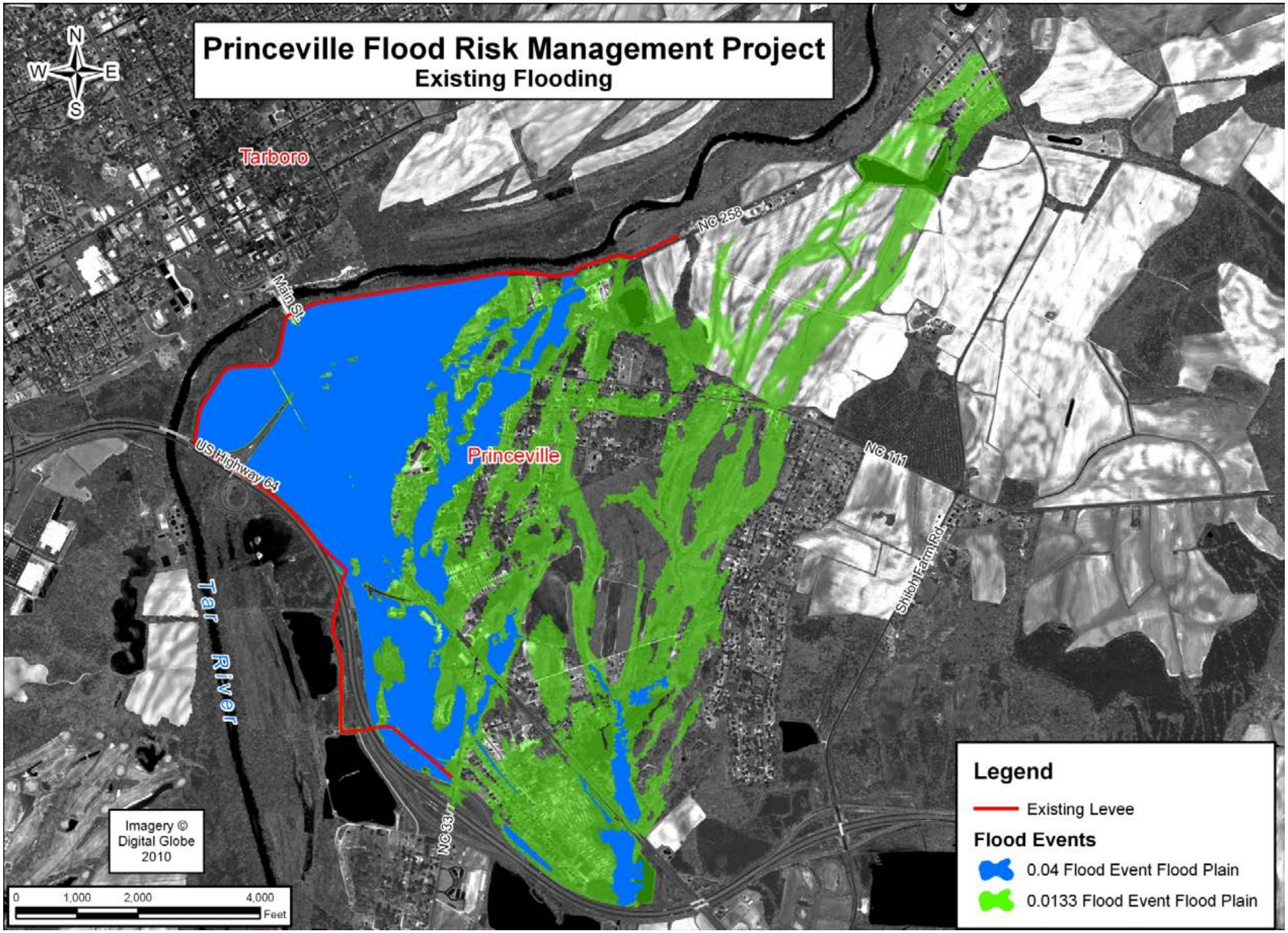

Exhibit 6-1. 0.04 & 0.01333 AEP Event Inundation

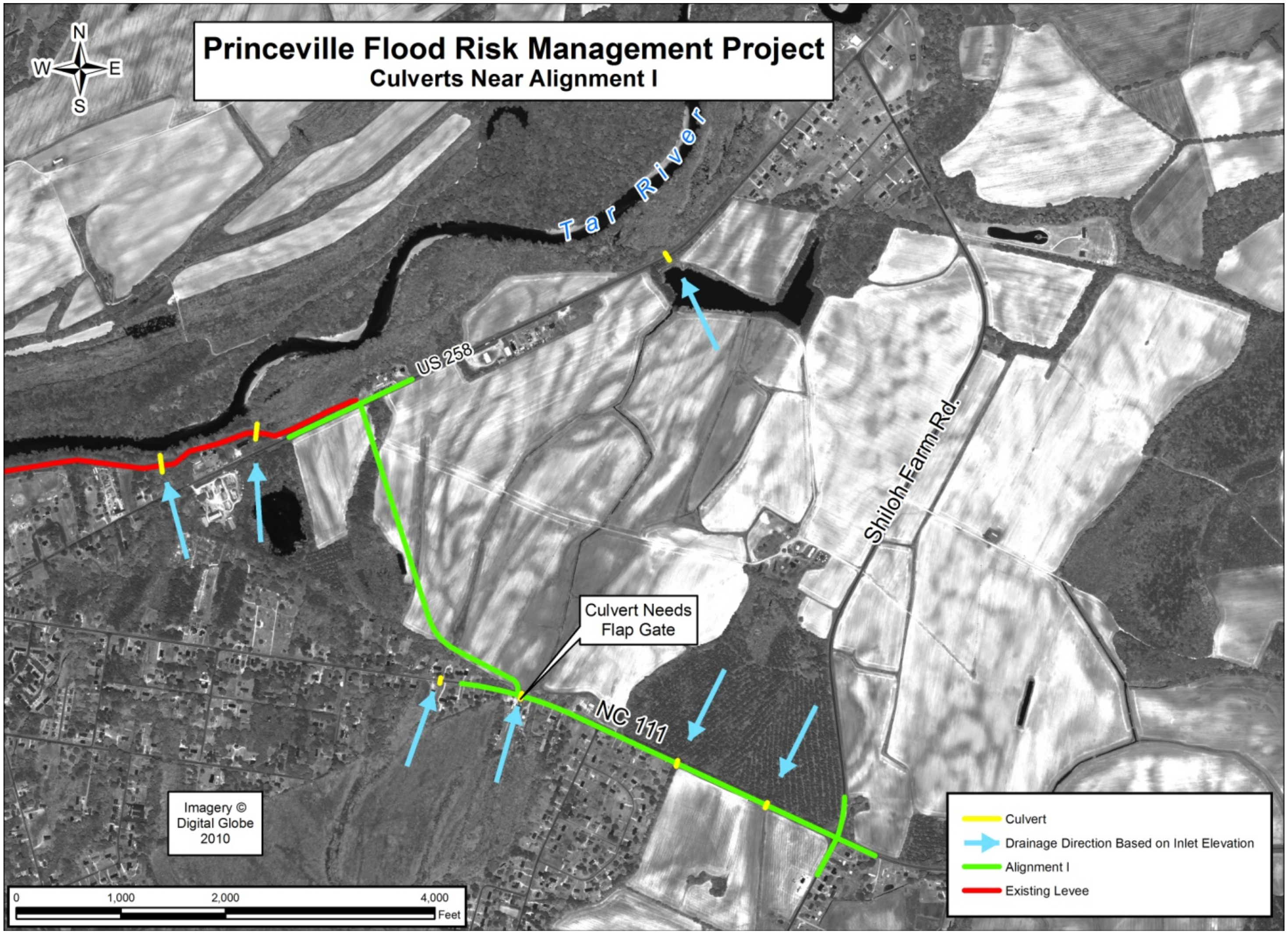

Exhibit 6-2. Culverts near Alignment I

Raise Existing Levee

Initial steady flow modeling of northern alignments, shown in Exhibit 6-3, indicated that raising the existing levee to increase the level of protection would also increase the elevation of any flood event in excess of the 0.00333 AEP event by roughly 0.4 ft. Raising the levee would take away the storage area behind the levee for events greater than the 0.00333 AEP event, forcing the flood waters to flow down the river channel and through the three bridges between Princeville and Tarboro. The increased flows, for these larger floods, passing thru the bridges will cause an increase in water surface elevation of 0.2 to 0.4 foot above Main Street Bridge and around 0.2 feet between US 64 and Main Street, if the levee was raised 4 feet. In the incremental analysis that was performed on the most cost effective alternative, two of the increments included the raising of the existing levee. These two increments would require the construction of a ring levee to provide a consistent level of protection on all sides of Princeville. Please refer to Chapter 9 for additional information on this topic.

Levee Extensions

These alternatives would be constructed without raising the existing levee. All of these solutions will be located outside of the existing flood way. The area of the flood plain that will be used to construct these alignments is considered to be a non-effective flow area. This is an area in the flood plain that does not contribute to the flow-carrying capacity of the flood plain due to a blockage downstream; in this case it is the existing levee and the road-fill for US 64. This creates a large pond when the water flows around the upper end of the levee. The construction of a levee in this area does not change how the water flows as it is already blocked from flowing beyond the US 64 road fill. For the 0.2, 0.1, 0.02, 0.01, and 0.002 AEP event profiles, there are no significant changes in the modeled water surface elevations. However, there are some changes in the 0.005 and 0.00333 AEP events due to the loss of storage caused by the extension of the levee on the north end. This loss of storage will cause an increase, between 0.1 and 0.3 feet, in the river water surface profiles for the area between the lower end of the levee and the upper end of the levee. The levee extension layouts can be seen in Exhibit 6-3.

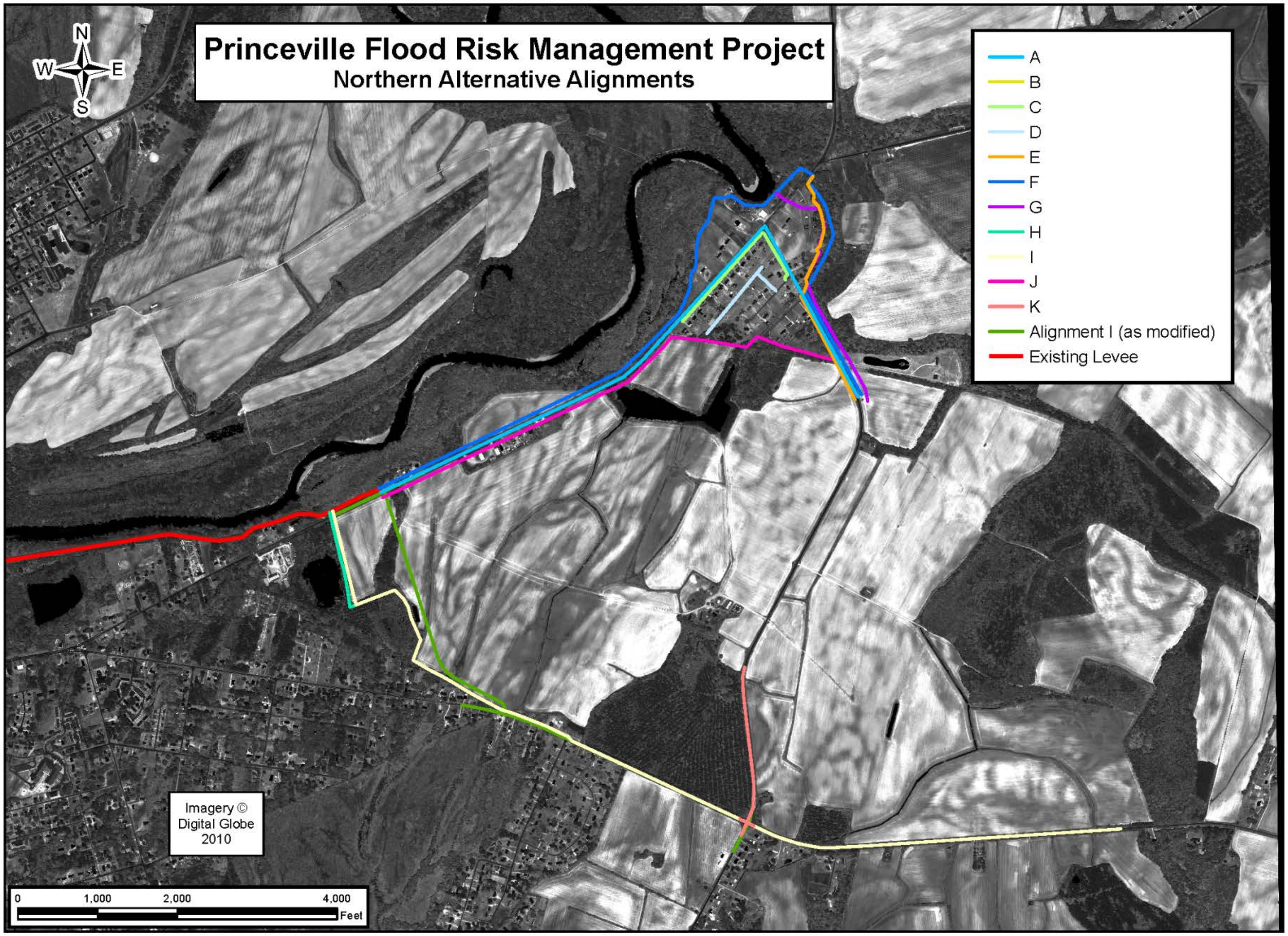

Exhibit 6-3. Levee Extension Alternatives

Interior Drainage

The extension of the existing levee will create situations that will either require the interior drainage system to be expanded or redirected. There is an area on the eastern side of Princeville and on the west and southeast side that will require a drainage plan be developed, as the outlets will require the addition of backflow devices to protect the area from being flooded from the river. The storage area on the west side has significant capacity. The storage area on the southeast only has minimal storage capacity without having to relocate residential structures. The eastern area is addressed below:

Alternative levee alignment "I" would effectively divide the existing drainage area that tries to drain under NC 111. The resulting interior drainage basins are shown in Exhibit 6-4. As mentioned above, the installation of a backflow prevention device on an existing drainage structure partially addresses the artificial basin divide created by the levee alignment. The other partial solution to this division would involve redirecting interior flow in a northwesterly direction to pool into an existing detention pond. Water in this pond eventually drains under Highway 258 and is introduced back into the Tar River by an existing culvert (with backflow prevention device) which is part of the original Princeville levee project. Water would initially be directed towards the pond through the use of an interior drainage ditch next to the landside levee embankment.

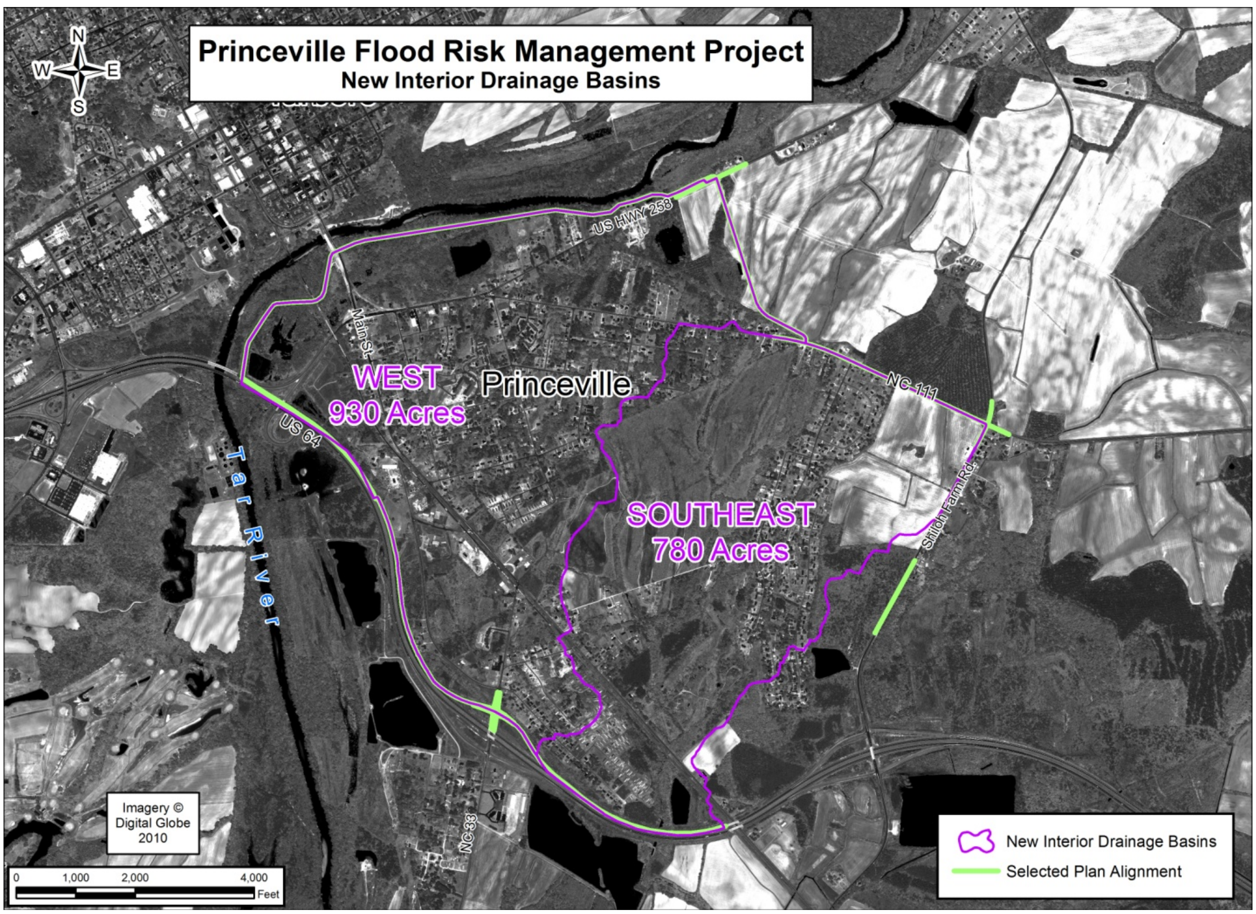

Exhibit 6-4. New West and Southeast Interior Drainage Basins

Surveyed invert elevations of the two culvert inlets suggest that nearby drainage is encouraged to flow north under NC 111. This flow direction is most likely true only under minor rainfall events. The LiDAR-based topography suggests that rainfall will eventually drain to the south where historic dendritic drainage patterns are visible on aerial photography. Exhibit 6-5 shows the primary drainage paths for the west and southeast interior drainage basins as well as the interior area that initially drains to the north. The lack of relief in topography makes it difficult to identify the exact location of a breakline between flow to the north/south during minor rainfall events.

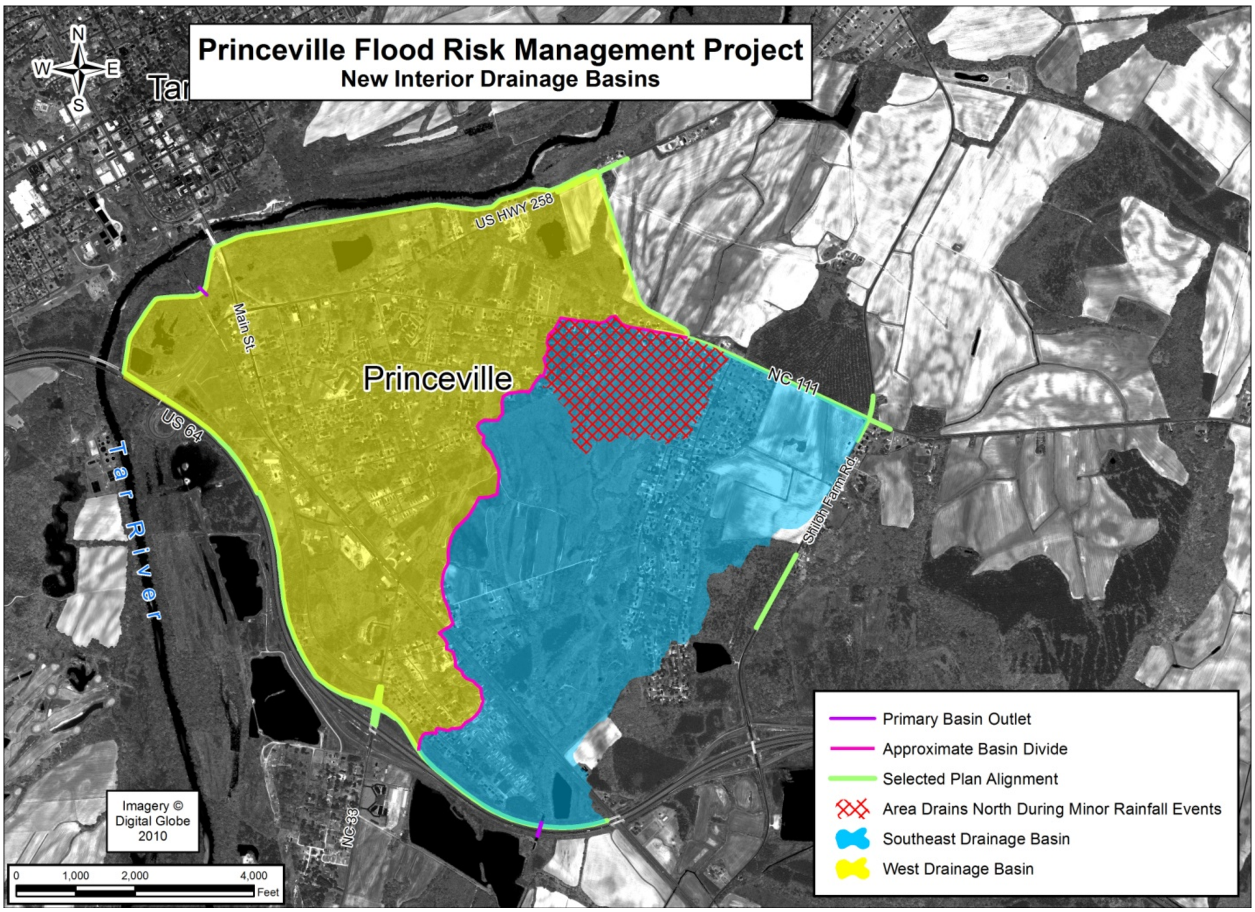

Exhibit 6-5. New Interior Drainage Basin Layout

A portion of the existing floodplain northeast of Princeville currently drains in a southwest direction towards the existing detention pond mentioned in the above paragraph. As shown in Exhibit 6-6, in order to avoid water pooling along the riverside toe of the levee embankment near HWY 258, it is proposed that water be redirected to another existing detention pond 0.5 miles south of the HWY 258/Shiloh Farm Rd. intersection. Again, a ditching system along the riverside of the levee embankment near HWY 258 could be constructed to convey flow towards the existing adjacent farmland irrigation canal network. Flow would then be carried north towards the detention pond.

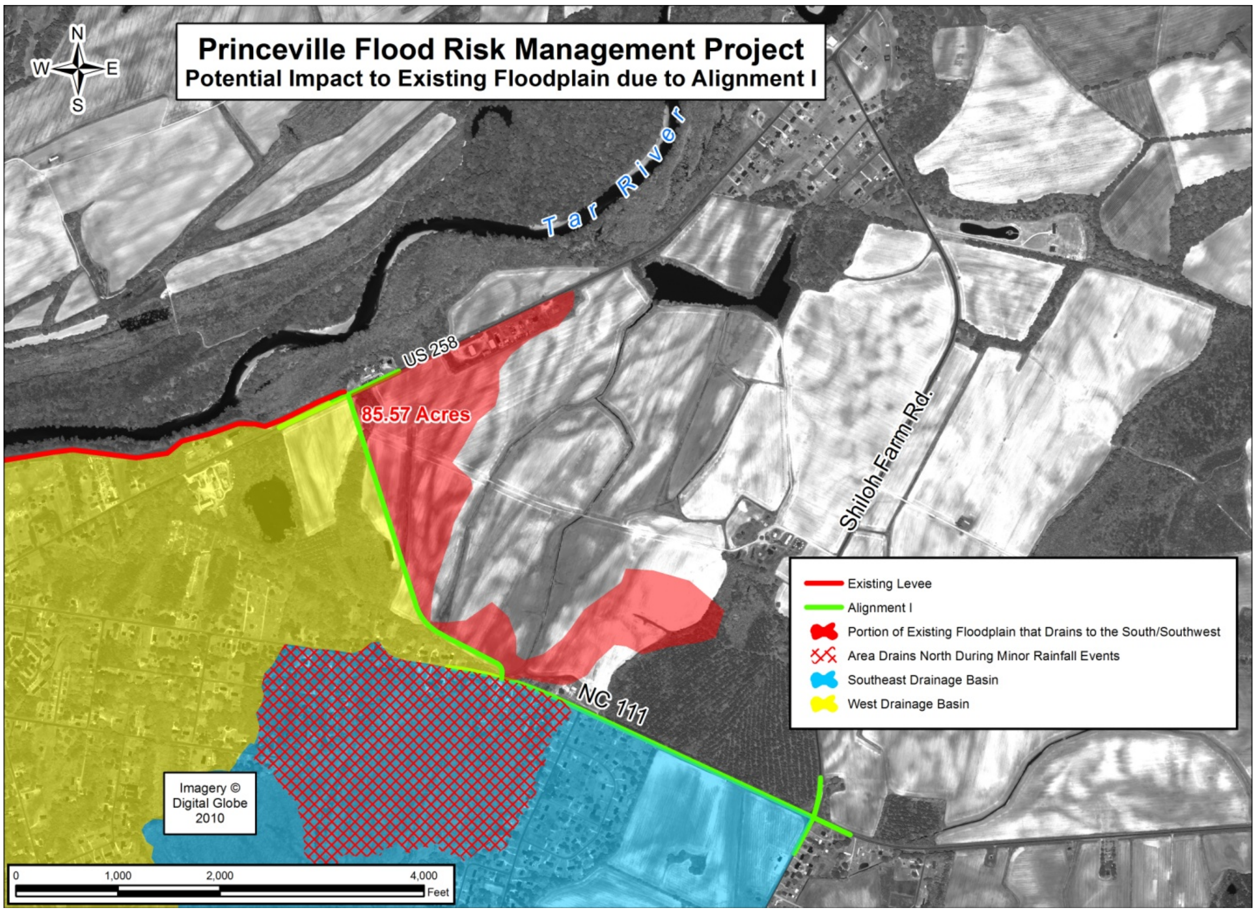

Exhibit 6-6. Potential Floodplain Impacts with Alignment I

Incremental Analysis of the Selected Plan

The incremental analysis approach for which formulates the selected plan was based on the Tarboro River and its complex flooding progression through the town of Princeville. As flood waters rise around Princeville, breach locations were identified, caused by either backflow through culverts/pipes, water overtopping a roadway/levee, or a combination of both. The trigger elevation for when the breach eventually occurred could then be traced back through a stage-discharge frequency relationship to a corresponding AEP event. Breach locations associated with lesser-magnitude events were addressed first. The proposed protection features were then carried forward and considered a step in developing the existing system. With these measures in place, breach locations associated with greater-magnitude events were sequentially addressed. This approach results in a system of measures that builds on itself to provide an acceptable level of protection which addresses all identified breach locations in Princeville. Table 6-1 is a simplified implementation of the recommended plan.

Table 6-1. Incremental Analysis

Increment					
1	2	3	4	5	6
Flap gates installed on the following culverts: #2, 7, 15a, 18a, 19, 22, 32, 33, 35, 36, 37.					
	The riverside shoulder of US 64 roadway along section of existing levee will be raised to equivalent protection elevation. **44'**	Raised an additional foot in elevation. **45'**	Raised an additional 2 feet in elevation. **47'**	Raised an additional 2 feet in elevation. **49'**	Raised an additional 2 feet in elevation. **51'**
	The US 64 westbound interchange ramps at NC 33 will be raised to an equivalent protection elevation to US 64 shoulder. **44'**	Ramps raised an additional foot to match US 64 shoulder elevation. **45'**	Ramps raised an additional 2 feet. **47'**	Ramps raised an additional 2 feet. **49'**	Ramps raised an additional 2 feet. **51'**
			New levee at north end, beginning on US 258 to the intersection at Shiloh Farm Rd. at elevations equivalent to existing levee, and eventually tying into natural high ground on Shiloh Farm Rd. **49'**	Levee raised an additional 2 feet in elevation. **51'**	Levee raised an additional 2 feet in elevation. **53'**
			2 sections of Shiloh Farm Rd., just north of NC 111 and just north of US 64, raised 1 foot in elevation. **49.3'**	Both sections of Shiloh Farm Rd. now form 1 large section after being raised an additional 2 feet in elevation. **51.3'**	After being raised an additional 2 feet in elevation, section now spans form US 258 at north end to US 64 interchange. **53.3'**

Levee Superiority and Planned Overtopping Design

The Selected Plan presents a step-down of levee crest elevations to closely match the natural slope of flow in the Tar River. Segment 4 consists of a starting levee crest elevation of 50 ft, dropping down to 49 ft at the transition point to Segment 3. Segment 3 consists of a constant levee crest elevation of 48 ft. Segment 2 consists of a constant levee crest elevation of 47 ft as does Segment 1.

The concept of levee superiority and planned overtopping is addressed by having the initial overtopping of the levee occur in Segments 3 and 4. Factors that determined this location include taking advantage of existing topography and attempting to direct flow away from high-risk damage centers. Overtopping flow in Segment 3 is naturally directed towards the low-lying ponding areas between US 258 and levee Segment 3. These ponding areas would be also be least hazardous locations considering current surrounding development. Overtopping in Segment 4 occurs at the levee crest elevation transition between 50 ft and 49 ft. This location would take advantage of the topography by directly flow towards a natural drainage path. This path extends south to the triple box culvert under US 64. A large portion of this drainage path is currently undeveloped.

Potential Wave Action, Overtopping, and Erosion

Potential failure modes for earthen levees associated with wind-generated wave action will be evaluated in the Preconstruction Engineering and Design phase. The most northern segment of proposed levee travels along NC 111 in a perpendicular orientation to the flow of the adjacent Tar River. This segment was identified during design review as a potential scenario of producing erosion on the levee as a result of water upon the river side face of the levee, traveling across predominately open agriculture land. This location will be evaluated using USACE's Coastal Engineering Design and Analysis System software to determine associated wind-generated wave action. Should modeling results indicate a high potential of levee failure due to this scenario, an appropriate erosion protection scheme will be developed, cost-evaluated, and implemented during Preconstruction, Engineering, and Design phase (PED).

Chapter 7 **Interior Drainage Analysis**

The intent of a levee is to reduce flood damages from a flooded river. However, the nature of the levee can create a different set of problems concerning interior drainage. Interior areas formally flooded by slowly rising flood waters from the river may now be subject to localized flooding that can occur more quickly. An analysis of the interior drainage areas in southeast Princeville was completed in accordance with EM 1110-2-1413, Hydrologic Analysis of Interior Areas. The analysis of interior flooding is based on a coincident analysis of exterior and interior stages. The exterior stage is developed through an analysis of river gage data and the interior stage is developed by routing a frequency storm event through the interior area. Under normal conditions for the Princeville area, interior flood waters are passed through the levee through gravity outlets (culverts) when the water levels are higher than the water levels of the exterior area (Tar River). The gravity outlets were analyzed with and without flap gates which will shut when the exterior stage is higher than the interior stage. Currently, some of the culverts do not have flap gates installed on them; future conditions would include their addition. All elevations refer to North American Vertical Datum of 1988.

As shown in Exhibit 7-1, Interior drainage inside the protected area of Princeville was divided into 2 primary basins: West and Southeast. The area that was initially analyzed was the Southeast Drainage Area in Princeville, an approximate 780 acre (1.22 sq mi) site that is a mixed use of residential and wooded/wetlands area. The area drains through three 6’x7’ box culverts on the south side of town, under the levee (US 64) just west of Main Street. Approximately 100 feet downstream of the box culverts is a dirt road with two round concrete culverts (36 inches). There are no other impediments in the channel before it reaches the Tar River. The existing condition assumes that the interior drainage is routed through the 36 inch culverts with respect to the discharge from the box culverts. The box culverts can pass more water through them than the round culverts. Under high discharge rates, this can cause the water to back up in the box culverts and into Princeville. Alternative 1 is the removal of the 36 inch culverts and no flap gates on the box culverts; Alternative 2 is the removal of the 36 inch culverts and flap gates installed on the box culverts. The West interior drainage basin is approximately 931 acres and discharges into the Tar River through a 60 inch corrugated metal pipe culvert downstream of the Main Street Bridge.

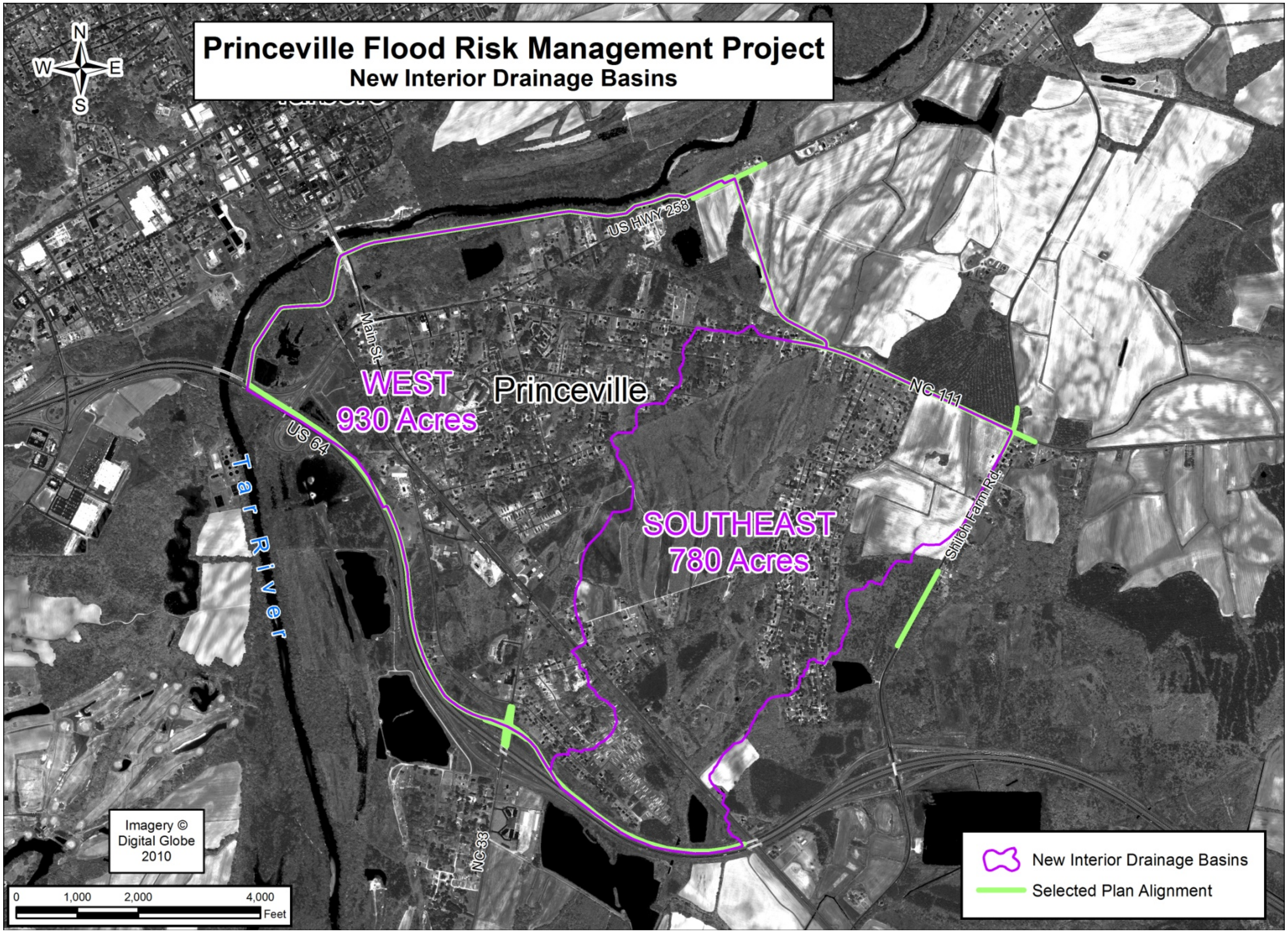

Exhibit 7-1. New Interior Drainage Basins

Hydrologic Analysis

Rainfall-runoff parameters were developed using the NRCS Curve Number Method. Based on land use, curve numbers (considered an average condition) of 78, 81, and 84 were estimated using the AMC III for the Southeast, West, and Northeast basins, respectively. The AMC III condition assumes that the soils are nearly saturated and would represent a worst-case scenario. This information formed the basis of the NRCS Unit Hydrograph, which was used to develop the 0.5, 0.2, 0.1, 0.04, 0.02 and 0.01 AEP event hydrographs. The volume of storage in the drainage area was estimated by using LIDAR data in ArcGIS ArcMap software. The 100-year ponding of the West basin with a max inundation volume of approximately 534 ac-ft was at elevation 34 ft. The 100-year ponding of the Southeast basin with a max inundation volume of approximately 239 ac-ft was at elevation 42 ft. The volume for each contour was computed, and a stage-storage function was developed using linear regression of the stage-storage parameters. The stage-storage function is used to determine the final interior stage based on the computed outflow.

Hydraulic Analysis

For the Southeast interior drainage basin, each frequency storm event was routed, according to TR-55 specifications, through the existing culvert conditions and subsequent alternatives using the equations for flow through a culvert based on the tailwater conditions. There are six different types of culvert equations based on the type of control (inlet or outlet) and the relative tailwater and headwater heights. Based on the inlet and outlet conditions (Type 3 – Tranquil Flow Throughout and Type 4 – Submerged Outlet) outflows were computed for each tailwater stage starting at elevation 35.5 ft and then computed for elevations 36 ft to 46 ft in one foot increments. If the headwater is less than the top of the road and the culvert is not flowing full then Type 3 flow exists. If the headwater is less than the top of the road and the culvert is flowing full, Type 4 flow exists. If the headwater elevation is greater than the top of the road, Type 4 flow is computed for the culverts plus the addition of weir flow over the road. In all cases, the existing culverts downstream of the box culverts are the controlling factor for interior water exiting Princeville. The existing interior stage was recomputed to take into account backwater effects from the box culverts. The approximate invert of the two 36 inch RCP is at elevation 35 ft. The invert for the three 6x7 box culverts 100 feet upstream of the RCP is also at elevation 35 ft.

The calculations were all completed using Microsoft Excel spreadsheets that were developed for the storm frequency events and culvert flow equations. There were 6 frequency storms routed through 12 different tailwater conditions for each culvert scenario. The outcome of each routing is the final interior peak stage. For the existing condition and each alternative scenario a graph was created with a suite of curves plotting the interior stage versus the return period for each exterior stage. This information will be used in the coincident probability worksheet outlined in the following paragraphs.

Exterior Stage Analysis

The USGS gage on the Tar River at the Main Street Bridge (USGS 02083500 Tar River at Tarboro, NC) is approximately 2 miles upstream from the area of interest and has been continuously recording daily discharge data since 1931 and gage height daily recordings since 1994. The gage height versus discharge value was plotted and a stage discharge relationship was developed. This information was used to interpolate gage values for the discharge readings from 1931 through 2008 which was the basis for this analysis. Data is divided into a wet season (Dec 1 through Mar 31) and a dry season (Apr 1 through Nov 31), which includes hurricane season. A stage-duration function (where duration is percent of time exceeded) was developed and divided into 10-percent increments. The middle value of each increment is taken as an index river stage. The increment interval for the duration represents the probability of the interval. The sum of the probabilities equals one. The wet and dry season stage-duration graphs are shown on Figure 7-1and Figure 7-2.

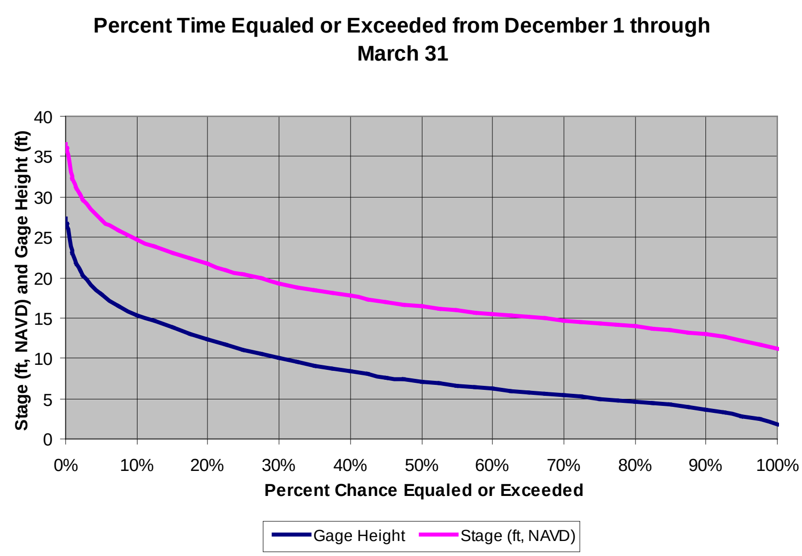

Figure 7-1. Stage-Duration Function for Wet Season

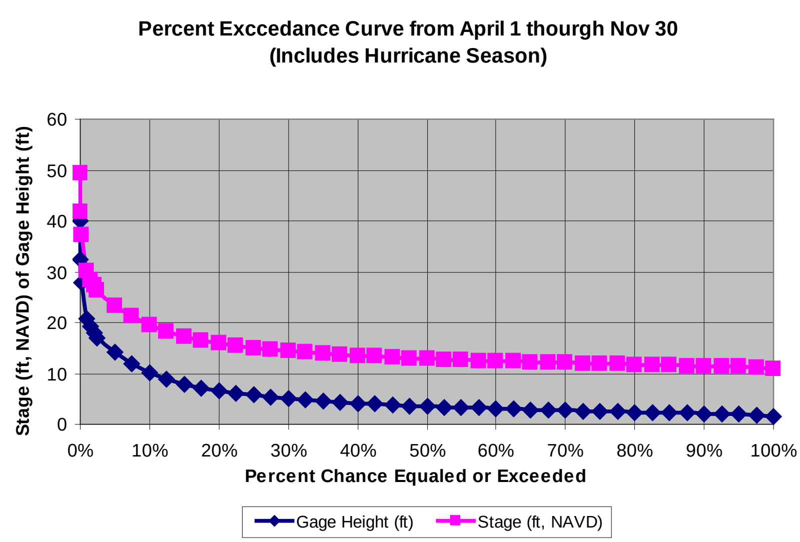

Figure 7-2. Stage-Duration Function for Dry Season

The coincident frequency analysis is performed to determine peak interior water surface elevations associated with the river index stages. Flood probability values for the interior given the probability of the river at a specified stage are then calculated. The method is repeated for each alternative. The probability plot is shown in Figure 7-3.

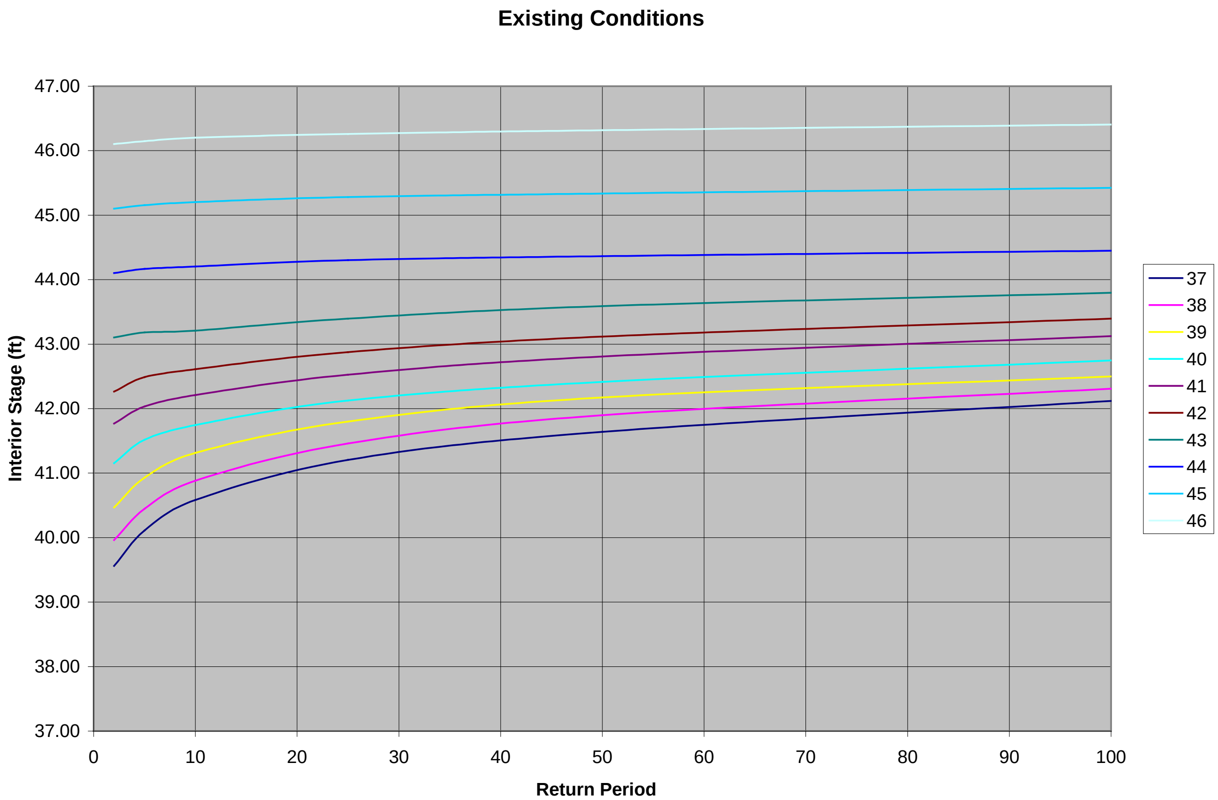

Figure 7-3. Coincident Frequency Probability for Existing Conditions

This information is complied in Table 7-1 and Table 7-2 to determine the coincident probability. Various tailwater elevations were evaluated, and the probability was determined from the gage analysis. The interior stage probability was determined from Figure 7-3. The weighted probability for interior elevations is the total sum of the exterior stage probability multiplied by the interior stage probability for each interior stage elevation.

Table 7-1. Max Interior Water Stage for Existing Conditions – Wet Season

	Maximum Interior Water Stage - Existing Condition - Wet Season (4 months)							
	Elev - Frequency Relationships							
Tailwater Stage	14	19	24	28	34	40	46	Wt. Probability for Interior Elevation
Stage Prob	0.5499	0.2	0.175	0.055	0.0199	0	0	
Interior Stage								
37	1	1	1	1	1	1	1	1
38	1	1	1	1	1	1	1	1
39	0.909	0.909	0.909	0.909	0.909	1	1	0.909
40	0.167	0.167	0.167	0.167	0.167	1	1	0.167
41	0.053	0.053	0.053	0.053	0.053	0.556	1	0.053
42	0.011	0.011	0.011	0.011	0.011	0.05	1	0.011
43	0	0	0	0	0	0	1	0
44	0	0	0	0	0	0	1	0
45	0	0	0	0	0	0	1	0
46	0	0	0	0	0	0	1	0

Table 7-2. Max Interior Water Stage for Existing Condition – Dry Season

	Maximum Interior Water Stage - Existing Condition - Dry Season (8 months)							
	Elev - Frequency Relationships							
Tailwater Stage	14	19	24	28	34	40	46	Wt. Probability for Interior Elevation
Stage Prob	0.5499	0.2	0.175	0.055	0.0199	0	0	
Interior Stage								
37	1	1	1	1	1	1	1	1
38	1	1	1	1	1	1	1	1
39	0.909	0.909	0.909	0.909	0.909	1	1	0.909
40	0.167	0.167	0.167	0.167	0.167	1	1	0.167
41	0.053	0.053	0.053	0.053	0.053	0.333	1	0.053
42	0.011	0.011	0.011	0.011	0.011	0.05	1	0.011
43	0	0	0	0	0	0	1	0
44	0	0	0	0	0	0	1	0
45	0	0	0	0	0	0	1	0
46	0	0	0	0	0	0	1	0

Table 7-3. Coincident Probability – Existing Conditions – Wet and Dry Season

8.2 Coincident Probability - Existing Conditions - Wet and Dry Season					
Interior Pond Elevation (A) (feet, NAVD)	Flood Season Wet Season Interior Pond Elevation Prob. $P_1(A)$	Nonflood Season Dry Season Elevation Prob. $P_2(A)$	Total Probability Interior Pond Elevation Prob. P(A)	Percent Chance Exceedance Frequency	Return Frequen (years)
37	0.3333	0.6665	0.9998	99.98	1
38	0.3333	0.6665	0.9998	99.98	1
39	0.3029	0.6059	0.9089	90.887	1.1
40	0.0556	0.1116	0.1672	16.717	6
41	0.0175	0.0357	0.0532	5.32	18.8
42	0.0037	0.0076	0.0113	1.131	88.4
43	0	0.0003	0.0003	0.027	3750
44	0	0.0003	0.0003	0.027	3750
45	0	0.0003	0.0003	0.027	3750
46	0	0.0003	0.0003	0.027	3750

The coincident probability for existing conditions in Table 7-3 shows that there is a high frequency that the interior stage elevation will be 37 (normal water surface elevation) to 39 feet. Table 7-4 shows a summary of the return frequencies for the existing and alternative conditions for each interior elevation. The table shows that removal of the existing culverts has a lower return frequency for interior flooding. The 100 year return frequency under existing conditions (inclusion of the two round culverts downstream of US 64) has an interior elevation just over 42 feet. The removal of the round culverts in Alternative 1 reduces the 100 year return frequency to an elevation of between 39 and 40 feet. Alternative 2 includes the removal of the existing round culverts and the addition of flap gates on the box culverts. Up to elevation 40, the return frequencies are similar. The top of the box culvert is at elevation 41 feet.

Based on the results of the interior drainage analysis of the southeastern drainage area in Princeville removing the existing round culverts downstream of the levee would provide approximately a two foot reduction in the 1% flood elevation.

Table 7-4. Summary of Coincident Probability

Interior Elevation (feet, NAVD)	Return Frequency Existing	Alt 1 Box Culverts	Alt 2 Flap Gates
37	1.0	1.0	1.0
38	1.0	16.4	16.3
39	1.1	58.0	57.2
40	6.0	2500.0	2307.7
41	18.8	3750.0	2212.4
42	88.4	3750.0	28571.4
43	3750.0	3750.0	194805.2
44	3750.0	3750.0	n/a
45	3750.0	3750.0	n/a
46	3750.0	3750.0	n/a

Pump Stations

The addition of pump stations were initially analyzed and determined to not be an effective incremental step in the alternative design process. Thereafter, pump stations were not considered in any design capacity.

Chapter 8 **Climate Preparedness and Resiliency**

The following chapter was prepared with assistance from the USACE's Institute of Water Resource's Climate Preparedness and Resiliency Community of Practice (IWR-CPR). Several tools were used to evaluate the potential impact do to climate change and their results are will be discussed in the following sections. The current USACE Screening-Level Climate Change Vulnerability Assessment (VA) Tool and the Engineering & Construction Bulletin (ECB) 2014-10 Tool was used in this analysis. In addition, the NonStationary Detection Tool, which will be incorporated into a new 2015/2016 Engineering Technical Letter, was also used in this analysis. This discussion will start at the broad regional scale and finish at the project level with the analysis.

To effectively incorporate climate change adaptation and to increase resiliency and decrease vulnerability of the Princeville Flood Risk Management Project, the first step is to identify where vulnerability exists. With the knowledge that climate information and understanding is constantly evolving, USACE developed the USACE Screening-Level Climate Vulnerability Assessment at the Watershed-Scale. The preliminary, screening-level nationwide analysis is built on existing, national-level tools and data that include indicators or processes to identify vulnerabilities in watersheds with respect to climate change.

The VA Tool identifies the top 20% of the watersheds vulnerable to climate change for eight business lines. Figure 8-1 shows results from the VA Tool looking at the Flood Risk Reduction Business Line. The figure breaks out the analysis with respect the climate models that are trending dry in the left column and the climate model results that are trending wet are in the right column. The upper row in Figure 1 show the climate model results centered around year 2050 and the bottom row centers on year 2085. As can be seen, under all four possible epics and scenarios, the Tar River watershed in North Carolina is not in the top 20% vulnerable to climate change for the Flood Risk Reduction Business Line.

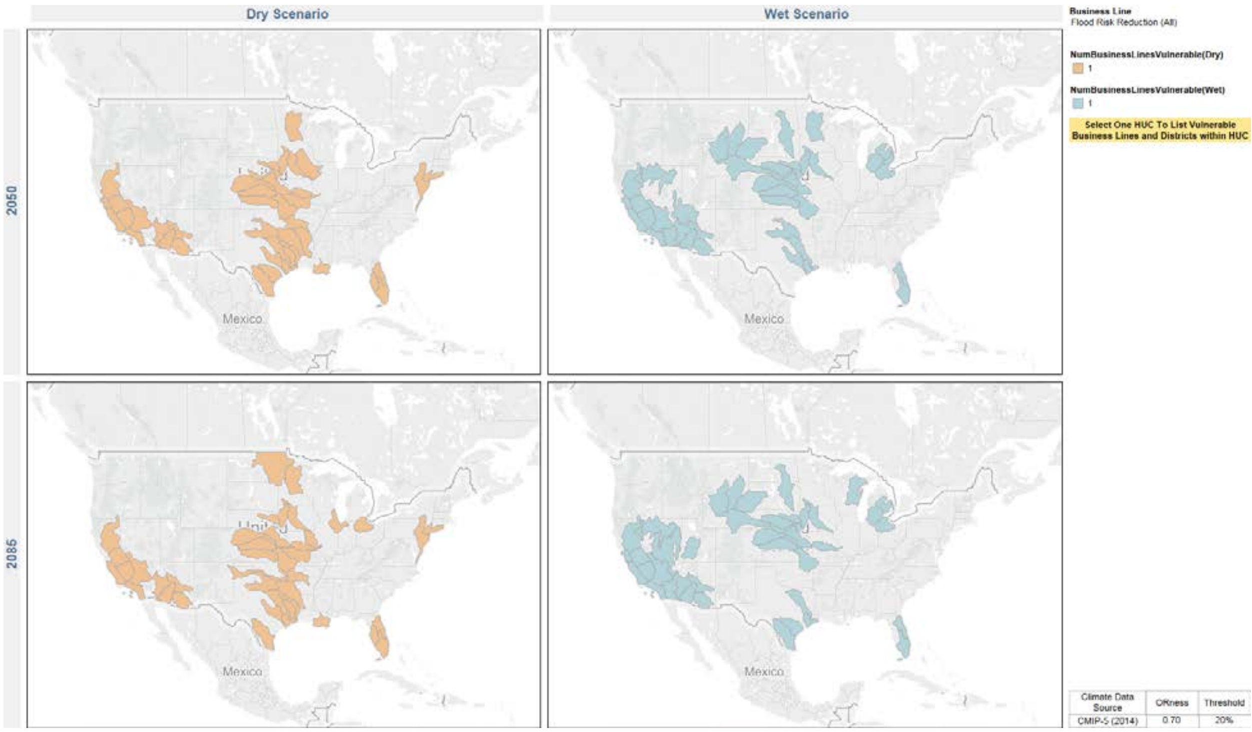

Figure 8-1. Scenario Comparison over Time

Figure 8-2 shows the vulnerability score overall vulnerability scores for all of the watersheds included in the Flood Risk Reduction business line for 30-year periods, or epochs, centered on year 2050 and year 2085 for the 47 climate models that trend dry. The bottom map shows the percent change in Weighted Order Weighted Average (WOWA) scores between year 2005 and year 2085. Areas that are darker red are projected to become more vulnerable to the Flood Risk Reduction-related indicators used in the analysis compared to areas that don't change much, shown in gray, or where vulnerability is projected to decrease, shown in green.

Note that in this screening assessment most HUCs are projected to become more vulnerable over time given these indicators and weights (red in the lower view). Some watersheds may even become less vulnerable over time. These are the ones shown in green in the bottom map. Vulnerability is based on the selected indicators, their weights, and the aggregation results. The Tar River watershed for this project is grey in Figure 8-2 which has a change over time near zero percent.

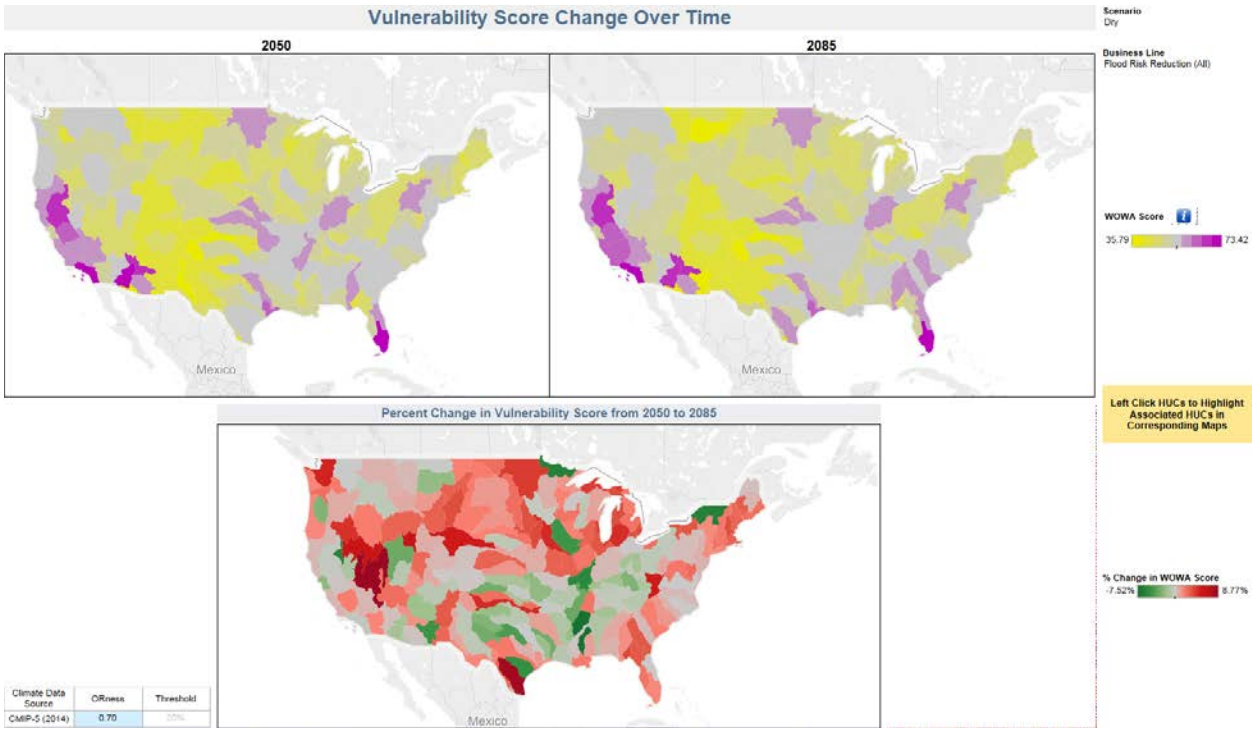

Figure 8-2. Vulnerability Score for Dry Scenario

Figure 8-3 shows the vulnerability score overall vulnerability scores for all of the watersheds included in the Flood Risk Reduction business line for 30-year periods, or epochs, centered on year 2050 and year 2085 for the 47 climate models that trend wet. The bottom map shows the percent change in WOWA scores between year 2005 and year 2085. Areas that are darker red are projected to become more vulnerable to the Flood Risk Reduction-related indicators used in the analysis compared to areas that don't change much, shown in gray, or where vulnerability is projected to decrease, shown in green. The watershed for this project is light red in Figure 8-3 which has a change over time near two percent increase in vulnerabilities.

While the VA tool identifies watersheds that may be vulnerable, it may not be appropriate to cascade those results to the project by default because projects are of finer spatial scales. To compensate for this, the ECB and ETL are employed to assess conditions at the finer spatial scales.

Figure 8-3 comes from the ECB 2014-10 tool and shows the annual maximum stream flow per year and the trend line associated with the annual values. The stream gage (USGS 02083500 Tar River at Tarboro, NC) had several years of missing data pre-1932 and those missing years are excluded from this plot. What this shows is that the trend of annual maximum stream flow is constant.

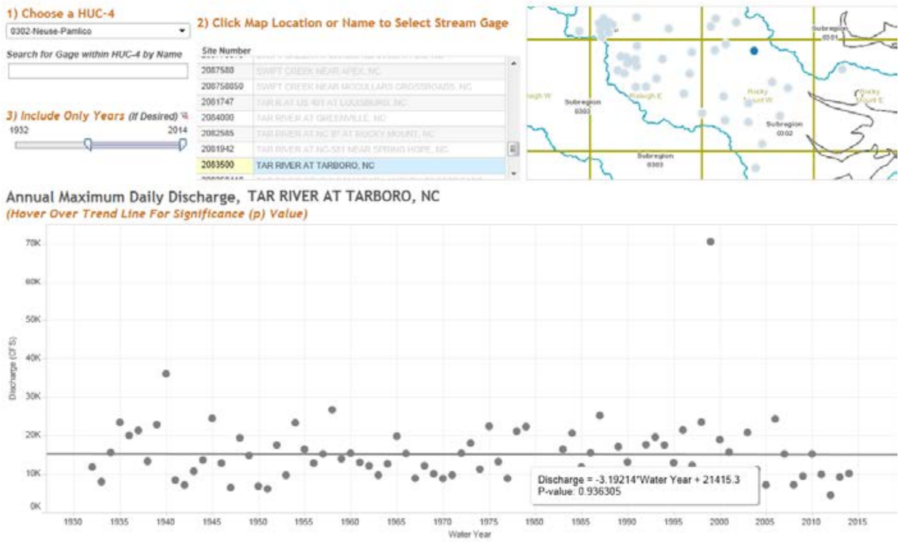

Figure 8-3. Annual Maximum Stream Flow for Tar River at Tarboro, NC

Figure 8-4 comes from the ECB 2014-10 Tool and show the projected climate data both the maximum and minimum values for the watershed along with the mean value for 93 climate ensembles through the year 2099. Looking at Figure 8-4, one can see that the variability of the maximum and minimum values are essentially consistent through time. Additionally the mean has a slight trend upward, but essentially represents a small change over 100 years.

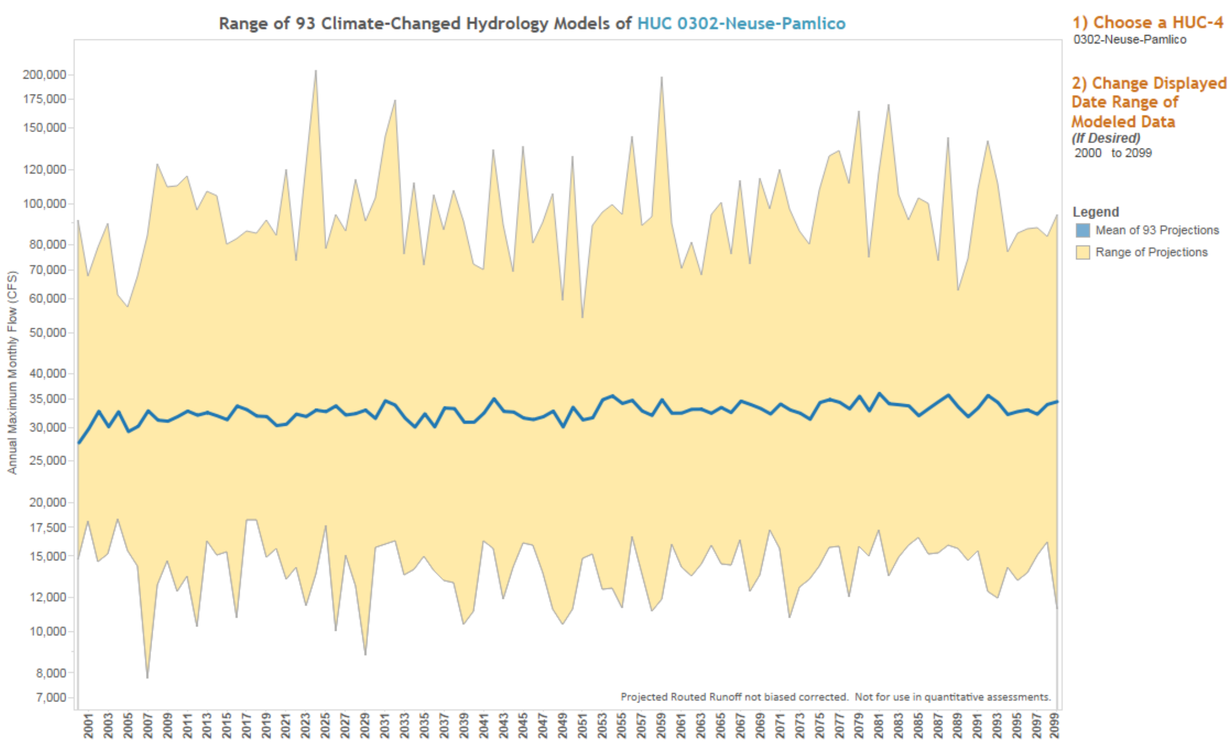

Figure 8-4. Climate Hydrology for the Neuse-Pamlico Watershed

The use of the Detection Tool is to analyze the condition close to the project to ascertain climate change impacts. USACE projects, programs, missions, and operations have generally proven to be robust enough to accommodate the range of natural climate variability over their operational life. Stationarity, or the assumption that the statistical characteristics of hydrologic time series data are constant through time, enabled the use of well-accepted statistical methods in water resources planning and design that rely primarily on the observed record. Recent scientific evidence shows, however, that in some places and for some impacts relevant to USACE operations, climate change and human modifications of the watersheds are undermining this fundamental assumption, resulting in nonstationarity. Changes in hydrologic processes can occur either abruptly or gradually depending on the characteristics of the non-stationary factors affecting physical processes. For example, changes in water regulation through the construction of a dam would abruptly change the streamflow patterns downstream. On the other hand, ongoing development within a watershed would gradually alter the shape of the resulting flood hydrograph over time. Statistical methods have been developed to detect both types of change in the tool.

Figure 8-5 show the results from the Detection Tool for the stream gage (Tar River at Tarboro, NC) after the multiple statistical checks have been performed. The upper plot represents the observed annual maximum data. The vertical bar represent times in the record that a statistical change and the observed record is detected. The middle graph is a heatmap of the methods that detected a change. As noted in the figure, there is not a large consensus (number of detections at the year) of detection methods indicating a strong likelihood of an actual change to the hydrology at the gage. The bottom figure plots the mean, standard deviation, and variance of the observed record showing the any change over time. Since the heatmap does not show a large number of detections for a year, it statically suggests that the hydrology has been stationary over time.

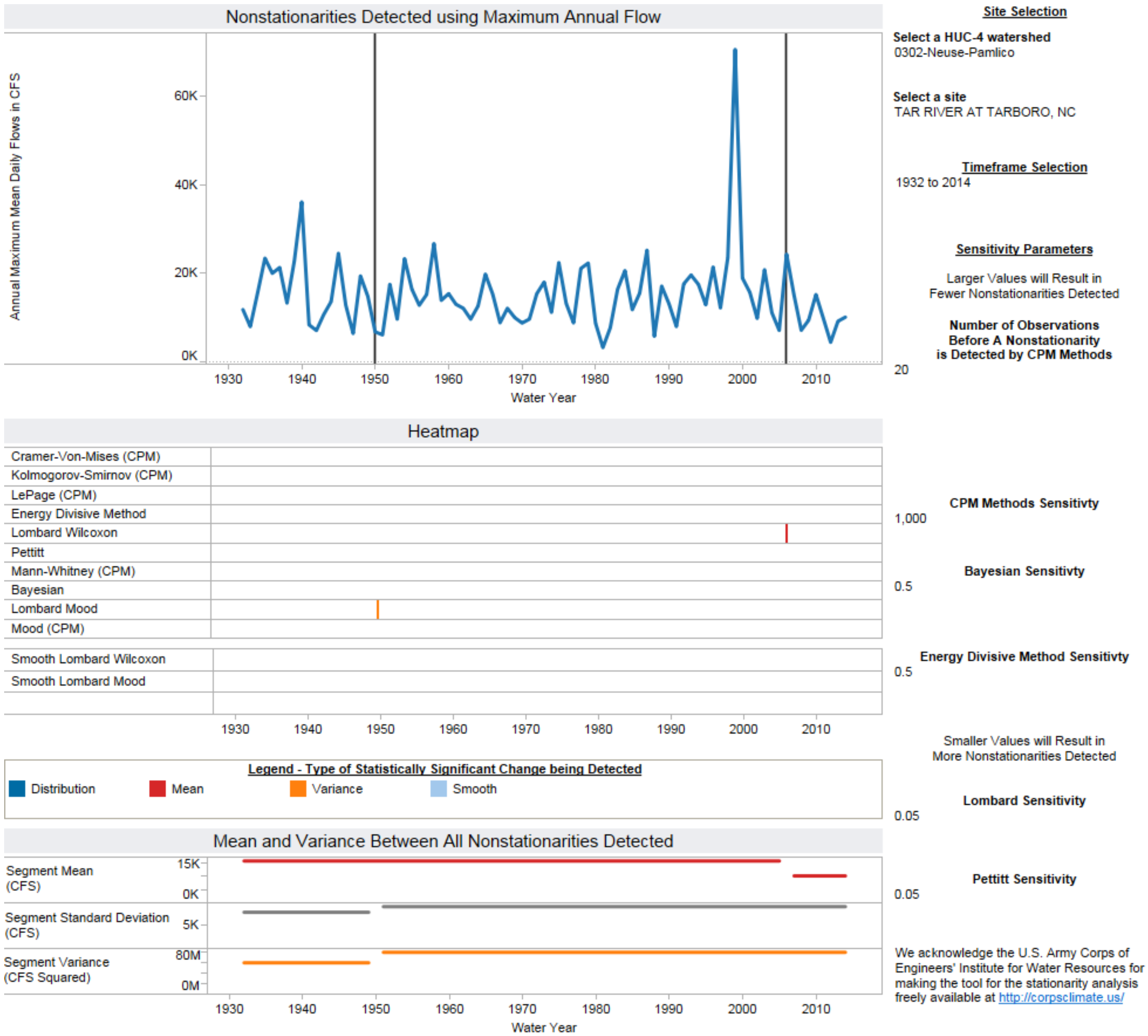

Figure 8-5. NonStationaries for Tar River at Tarboro, NC Gage

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Chapter 10 **White Paper Supplement**

Loss of flood plain Storage volume Effects

The following text is from White Paper: Analysis of Floodplain Storage and Losses of Floodplain Storage Volume and its Effect on Flood Heights Due to Levee Improvement, Town of Princeville, North Carolina. This paper was published early on in the project life and much of the final calculated values are not longer applicable to the final recommended plan. The following analysis assumed a recommendation plan with alignment A/B and K in place for the with-project conditions. These alignments can be seen in Exhibit 6-3.

Background:

The construction of alternative levee extensions at the Town of Princeville would prevent overflow from the Tar River during a certain range of flood events, and thus, could potentially increase flood heights within the confined channel by loss of that out-of-stream storage within the floodplain. This analysis was conducted to demonstrate without-project versus with-project flood heights (water surface elevations) within the confined reach of the Tar River, within the study area surrounding the Town of Princeville and vicinity, to determine if additional measures were required to mitigate for added flood heights.

Analysis of the situation:

The analysis of without- and with-project conditions required calculation of the amount of storage available, and what would be lost to overflow storage during the design event, the rate and elevation at which the floodflows would leave the river and flow into Princeville, and the effect this would have on downstream flood elevations on the river and adjacent levees.

Amount of flood overflow storage available:

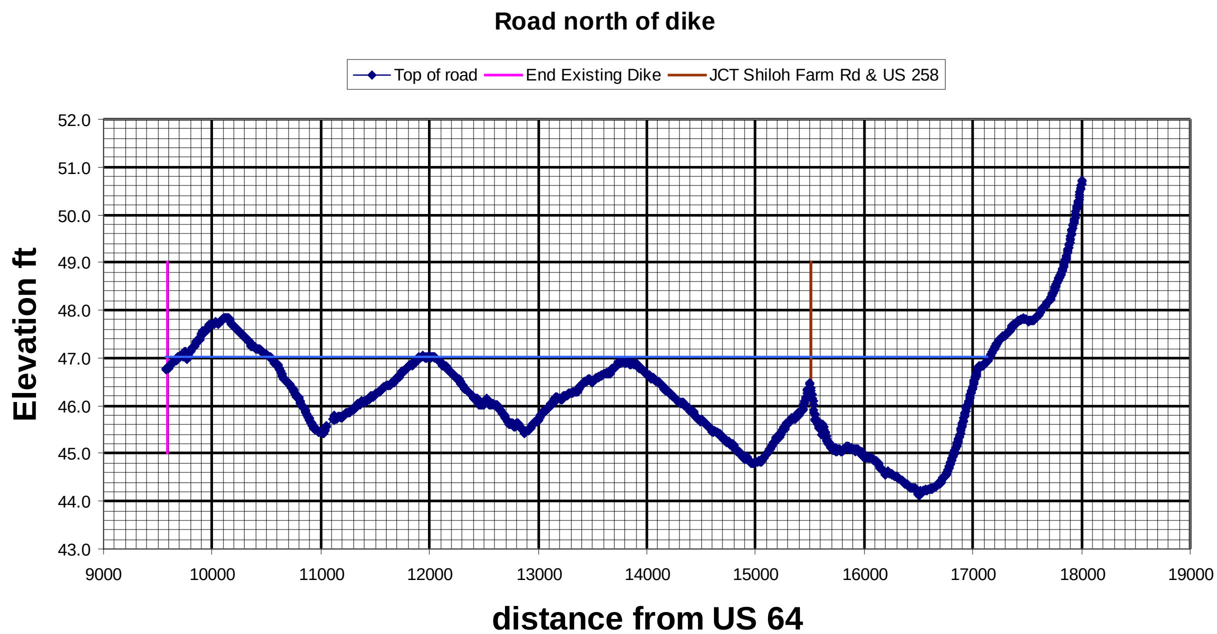
The volume of storage was determined by measuring the area at various elevations and using the “average double end method” to calculate the volume up to an elevation of 44 feet (i.e., the storage available below elevation 44 and behind the existing levee). Since the area confined by 46, 47’ and 48’ contours are very similar, the area contained within those elevations was considered to be equal to that of the 47’ contour. This summation of storage volumes at various elevations produced the following elevation-storage data:

Princeville Elevation Storage Relationship

ELEVATION	CUMULATIVE VOLUME million cu ft
44	276.0
46	499.0
47	575.2
48	651.4

Calculate the flow rate into the storage area:

There are two possible methods to determine the flow rate across the road into the storage area, **1**) Using the equation for flow over a broad crested weir, or **2**) Using Manning's normal flow equation. Each method was evaluated to determine which method produced results most reflective of reality. The primary overflow area consists of a reach upstream of the northern levee terminus extending from the intersection of US 258 and Shiloh Farm Road upstream to a point where Shiloh Farm Road reaches elevation 49 feet. The (exaggerated) profile of the road is shown below. The average depth for elevation 47 feet was computed by measuring the area between the elevation 47 foot line and the ground profile. By dividing the area by the top width you can calculate the equivalent depth of 0.9 feet. The depth at elevation 48 was computed the same way, as 1.8 feet. The depth at elevation 49 feet was determined by adding 1 foot to the depth for elevation 48, or 2.8 feet.



US 258 and Shiloh Farm Rd Profile

Weir equation:
Using the normal flow equation for weir flow:

Q=CLH^3/2, using C=2.5. The weir lengths for the weir equation are as follows:

elevation 47' - 7220 feet, elevation 48' - 8050 feet, and elevation 49' - 8250 feet.

This equation gives the following discharges for the associated river elevations:

Elevation 47' - 16,115 cfs, 48', - 48,600 cfs, and 49' - 96,635 cfs.

At the elevations 47', 48', and 49', then, the estimated time to fill the storage area was computed, using the equation:

Time (min)= storage(cu ft)/ (Q (cfs) X 60 sec/min);

This gives the following times to fill the storage area of:

Elevation 47' - 51.6 min, elevation 48' - 19.7 min, and elevation 49' - 11.2 min.

All of the times calculated using this methodology were far too short, and actually decreased with the larger floods, where the flood resulting from Hurricane Floyd took several days to fill the interior area. This methodology was determined to be inappropriate for this purpose.

Normal flow equation:
By using the Manning's equation for normal flow:

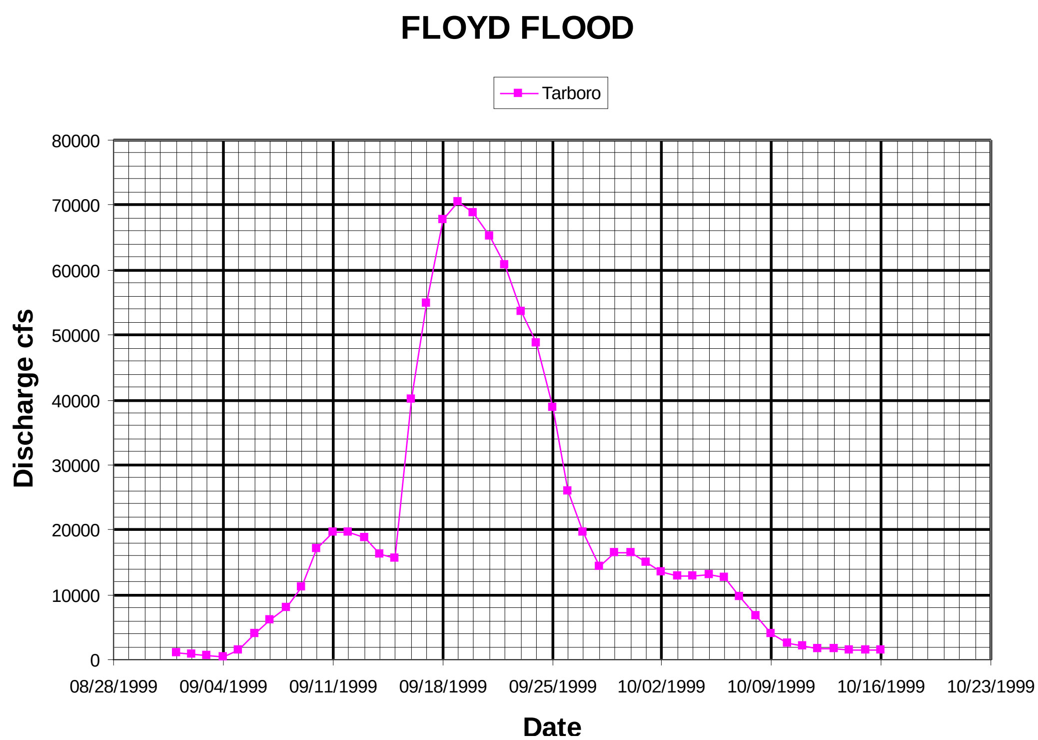
Q=(1.486/n) X AR^2/3 X S^1/2

Using S=0.000062 S^1/2=0.007874 and n=0.10.

Where R = Area divided by the wetted perimeter.

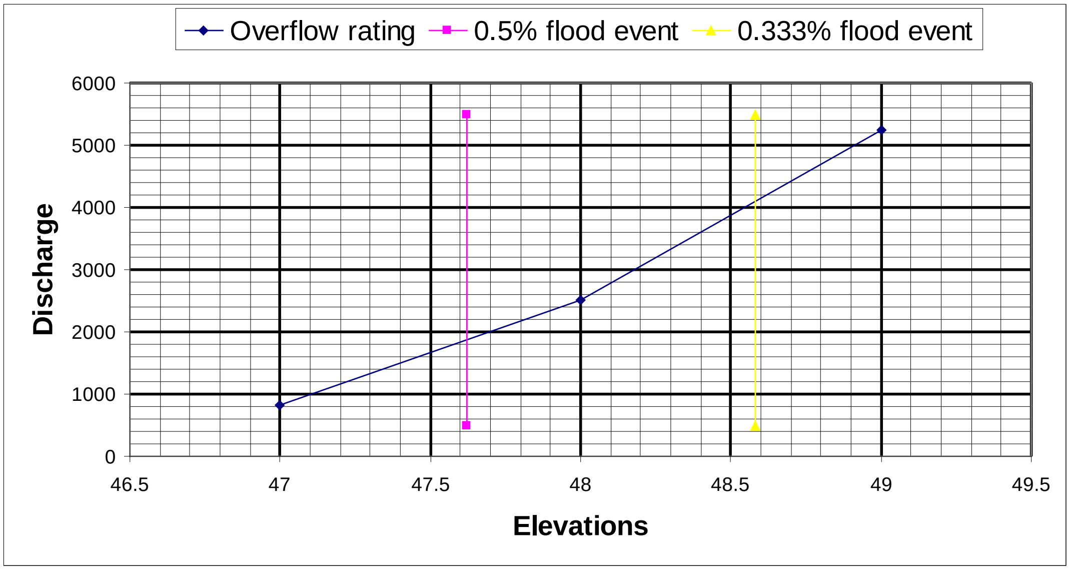
River elevation at 47': R=7220/7550=0.956
River elevation at 48': R=14500/8050=1.801
River elevation at 49': R=22450/8250=2.758
At 47' the Q= 820 cfs
At 48' the Q=2511 cfs
At 49' the Q=5237 cfs
Using the time equation from above we get:
At elev 47' Time = 17 Hrs
At elev 48' Time = 25 Hrs
At elev 49' Time = 29 Hrs

The rise of floodwaters using this methodology was determined to be approximately the same rate of rise as indicated by hydrograph for Hurricane Floyd, or about 3 feet per day.



Tarboro Gage Hurricane Floyd Flood Hydrograph

The use of Manning's equation, then, appeared to correlate better with the flood hydrograph than by use of the weir equation. Those calculated discharges, then, were used to adjust the HEC-RAS discharges. The figure shown below is a plot of the over flow discharges so that the correct overflow Q's can be selected for two comparative floods calculated using the HEC-RAS model; that of a 0.005 AEP, and that of a 0.00333 AEP event, for use in bracketing the design.

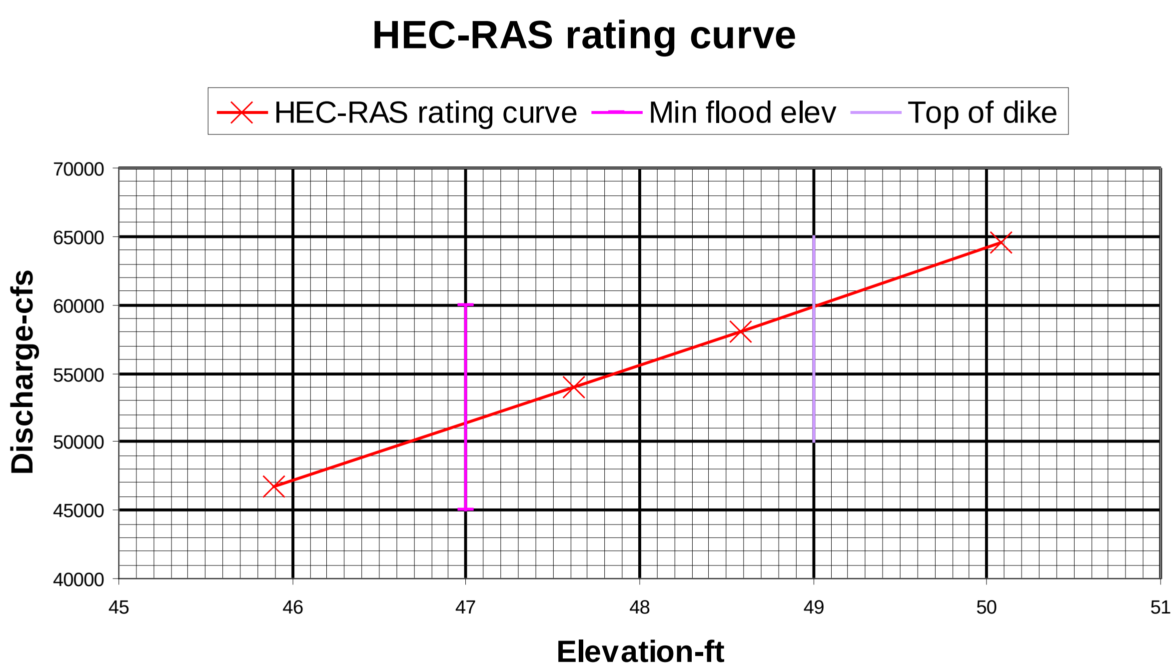


Overflow Area Rating Curve

HEC-RAS model adjustments:

Peak discharges were computed in the HEC-RAS model, a discharge - frequency curve was generated, and peak discharges for the 0.005 and the 0.00333 AEP events were determined. These two profiles were determined for the lower end and the upper end of the project area. The water surface elevations for the 0.01, 0.005, 0.00333, and the 0.002 AEP events, at section 253976, were plotted in order to determine the flow loss for each flood event. The figure below shows the relationship of the elevation to discharge for the river model at this section. This curve is only valid between the beginning flooding elevation and the top of the existing levee, since it would only be during events of that magnitude in which storage loss would occur. Using the figure, the elevation of the 0.005 AEP event would be 47.6 feet, and the 0.00333 AEP event elevation would be 48.6 feet. From Figure A-45, the overflow rating curve elevations, these elevations were used to select the overflow rate; 1,800 cfs for the 0.005 AEP event, and 4,100 cfs for the 0.00333 AEP event. The 0.005 and 0.00333 AEP events were re-run to see what effect loss of flow would have on floodflow elevations. The full flow discharges were used at the downstream end of the project since most of the lost flow would eventually return to the channel at this point. The first tables shown below gives water surface elevations for the 0.005 and 0.00333 AEP event profiles, and the second table gives WSELs for the 0.005 and 0.00333 AEP event profiles with the overflow discharge taken out.

The USGS measurements file for the Tarboro Gage was referenced for any data that was available in the 50,000 to 55,000 cfs range. There is only one reading taken in that range, it was during the rising side of the Hurricane Floyd flood. The elevation was about 0.4 feet higher than that of the RAS model.



HEC-RAS Rating Curve at Section 253976

Storage flow included with Peak Discharge

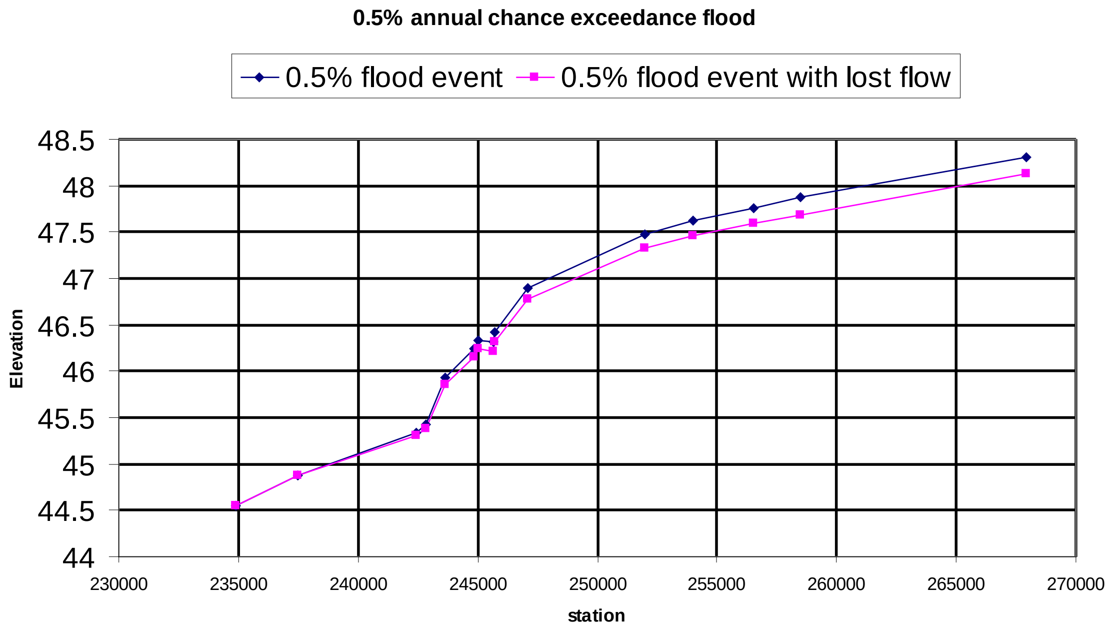
River Sta Feet from Mouth	Profile Return period Yrs	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)
267935	200-yr	54000	9.39	48.31
267935	300-yr	58000	9.39	49.27
258478	200-yr	54000	9.2	47.87
258478	300-yr	58000	9.2	48.83
256527	200-yr	54000	8.84	47.76
256527	300-yr	58000	8.84	48.71
253976	200-yr	54000	8.36	47.62
253976	300-yr	58000	8.36	48.58
252004	200-yr	54000	8.05	47.48
252004	300-yr	58000	8.05	48.44
247087	200-yr	54000	7.27	46.9
247087	300-yr	58000	7.27	47.84
245728	200-yr	54000	7.05	46.42
245728	300-yr	58000	7.05	47.31
245686		Bridge	MAIN ST	
245644	200-yr	54000	7.01	46.31
245644	300-yr	58000	7.01	47.25
245050	200-yr	54500	7.74	46.33
245050	300-yr	58500	7.74	47.28
244960		Bridge	RR	
244870	200-yr	54500	7.7	46.24
244870	300-yr	58500	7.7	47.18
243633	200-yr	54500	6.66	45.93
243633	300-yr	58500	6.66	46.86
242826	200-yr	54500	5.98	45.42
242826	300-yr	58500	5.98	46.31
242631		Bridge	US 64	
242436	200-yr	54500	6.22	45.34
242436	300-yr	58500	6.22	46.22
237457	200-yr	54500	7.2	44.88
237457	300-yr	58500	7.2	45.77
234886	200-yr	54500	7.01	44.55
234886	300-yr	58500	7.01	45.45

Storage Flow Not Included with Peak Discharge

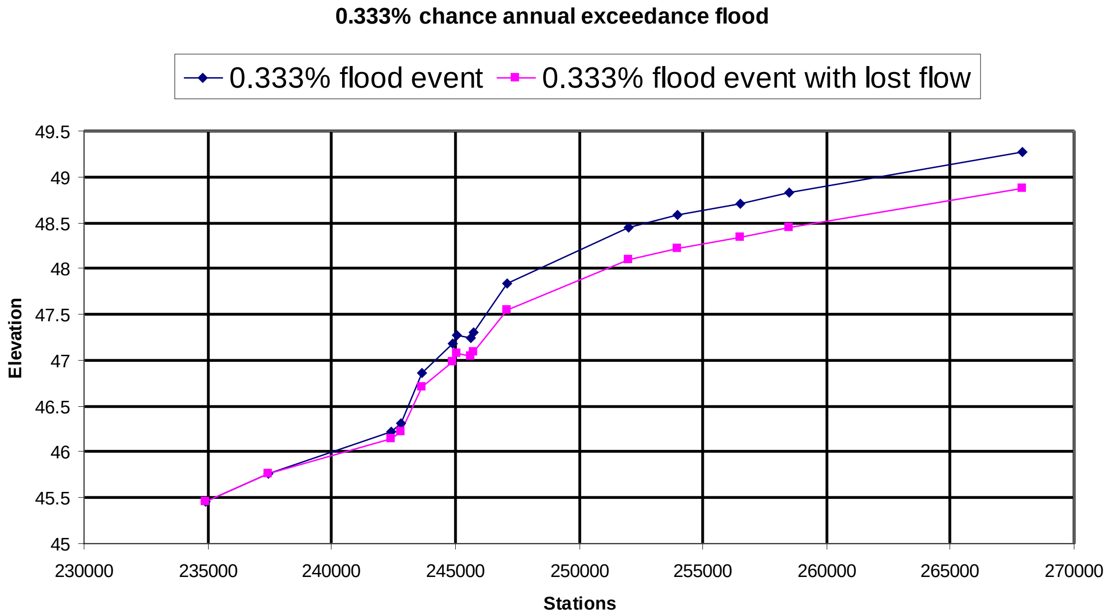
River Stage Feet from Mouth	Profile Return period Yrs	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)
267935	200-yr	54000	9.39	48.13
267935	300-yr	58000	9.39	48.87
258478	200-yr	52200	9.2	47.69
258478	300-yr	53900	9.2	48.44
256527	200-yr	52200	8.84	47.59
256527	300-yr	53900	8.84	48.34
253976	200-yr	52200	8.36	47.46
253976	300-yr	53900	8.36	48.22
252004	200-yr	52200	8.05	47.32
252004	300-yr	53900	8.05	48.09
247087	200-yr	52200	7.27	46.77
247087	300-yr	53900	7.27	47.55
245728	200-yr	52200	7.05	46.31
245728	300-yr	53900	7.05	47.09
245686		Bridge	Main St	
245644	200-yr	52200	7.01	46.22
245644	300-yr	53900	7.01	47.04
245050	200-yr	52700	7.74	46.24
245050	300-yr	54400	7.74	47.07
244960		Bridge	RR	
244870	200-yr	52700	7.7	46.15
244870	300-yr	54400	7.7	46.99
243633	200-yr	52700	6.66	45.86
243633	300-yr	54400	6.66	46.71
242826	200-yr	52700	5.98	45.38
242826	300-yr	54400	5.98	46.22
242631		Bridge	US 64	
242436	200-yr	52700	6.22	45.31
242436	300-yr	54400	6.22	46.15
237457	200-yr	52700	7.2	44.87
237457	300-yr	54400	7.2	45.76
234886	200-yr	54500	7.01	44.55
234886	300-yr	58500	7.01	45.45

The difference in the with- and without outflow storage scenarios produced elevation differences of approximately 0.1 foot for the 0.005 AEP event, to approximately 0.4 foot for the 0.00333 AEP event. (Water surface elevations at section 253976).

The first figure below shows the comparison of the 0.005 AEP event profile with and without the overflow. The second figure below shows the changes between the 0.00333 AEP event with and without the overflow.



With & Without Outflow Storage Scenarios 0.5% Chance Event



With & Without Outflow Storage Scenarios 0.333% Chance Event

Summary

The construction of a levee extension could cause, by removal of storage on the floodplain within the Town of Princeville, approximately 0.1 foot of increased floodflow height in the channel downstream of the existing upstream levee terminus, for an approximately 0.005 AEP event, which would also extend downstream through the project reach to that point at which floodflows exiting the floodplain would begin to re-enter the channel (i.e., the downstream end of the project). Construction of a higher design structure than that selected, for a 0.00333 AEP event, would potentially increase floodflow elevations within the channel by approximately 0.4 feet in that same reach.

This analysis indicated that the loss of storage caused by the extension of the existing levee would have a negligible effect on heights (water surface elevations) of floodflows within the channel. The change in the Conditional Non-exceedance probability (%CNP) for the 0.01 AEP flood would not change. The flood events between the 0.01 and 0.02 AEP flood events, % CNP would only change in the range of 4 to 5% and would not significantly affect project performance.

Addendum

The information described above in this white paper referred to a legacy alignment that cut off a larger portion of floodplain storage than that of the final recommended plan. The underlying hydraulic principal would still apply, though to a potentially smaller effect. The effects on floodplain storage "and potential induced flooding" related to the final recommended plan will be further analyzed during PED phase.

Chapter 11 **H&H Agency Technical Review**

Evaluation of Backwater model

The following are the comments from a review of the portion of the Tar River HEC-RAS model used for the Edgecombe County Flood Insurance Study. The review was performed by M. Gee of The Hydrologic Engineering Center in July 2007.

Background: The Wilmington District is considering a flood risk management project that requires the extension of a levee in the vicinity of the Town of Princeville, NC. A HEC-RAS model for the project has been developed by a contractor for the State of North Carolina Flood Plain Mapping Office in association with a new flood insurance study. One of the criteria being used by the Wilmington District in developing alternatives for the project is no increase in the flood impacts on neighboring communities. HEC has reviewed the RAS model with regard to its formulation, assumptions, implementation and internal consistency.

Basic Model Implementation

The HEC-RAS model reviewed simulates a single reach approximately 50 miles in length. The slope is about 0.00009 in the lower 20 miles and about 0.0002 in the upper 30 miles. This reach is modeled with 115 cross sections. There are 12 bridges and six lateral inflows. The RAS model is not georeferenced; therefore comparison of the computed inundated areas with the topography is not performed herein.

Downstream Starting Condition

Comment 1

The downstream boundary condition is important for rivers of this small of slope and was set in the RAS model based upon a known water surface elevation for the discharges simulated. The basis for these elevations should be documented. If there is any uncertainty in this elevation-flow relation, given the small slope of the study reach, an analysis of the sensitivity of the results to the downstream stage-flow relation should be considered.

Comment 1 Response:

Starting water surface elevations: These are defined because this model is a part of a larger model that went downstream below Greenville. To check the validity of the starting elevations, the model was started using the bottom slope at the lower end. The model was also started with a slope twice the bottom slope and with added cross sections in the first 600 feet. In both

Comment 2

The floodway analysis (RAS plan file “Tar River Floodway”) was done by increasing the starting (downstream) water surface for the 1% AEC flood event by 0.94 ft. What is the rationale for this?

Comment 2 Response:

The floodway elevation is .97 feet above the 100-year natural at the beginning cross section because they match the downstream profile.

Errors and warnings

Comment 3 Warnings and Errors:

HEC-RAS produces a set of errors and warnings that can be used to evaluate the computational accuracy of the solution. The warnings produced for this application are typical and in general do not indicate any need for model changes. There are, however, a number of divided flow indications for all of the profiles. The model should be checked in these areas to ascertain that flow paths exist so that flow can fill all parts of the cross section at these locations. Georeferencing the RAS model would assist with this determination.

Comment 3 Response:

The divided flow message appears to be because of a lot of low areas in the over banks, Most of them are minor and will not cause an increase in the water surface elevations if removed.

Selection of model parameters

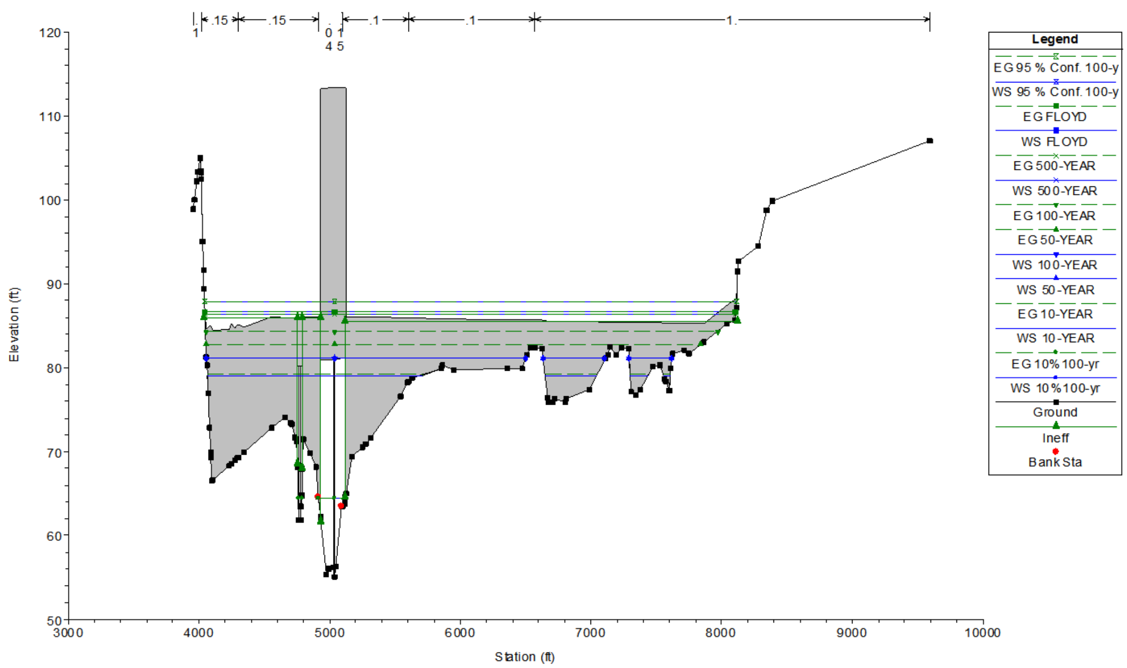
Model parameters for steady flow modeling consist primarily of roughness (*n*-values), cross section subdivision locations, and bridge modeling methods including approach and exit ineffective flow descriptions.

Comment 4   Roughness:

The *n*-values used produce a calibrated simulation and are reasonable in value. The *n*-values were varied transversely for each cross section based on the local anticipated distribution of roughness. This approach is appropriate and yields a more flexible model for simulation of modified future conditions.

Comment 5   Bridge modeling:

An example of concerns is the bridge deck at RS 451050 as shown below.

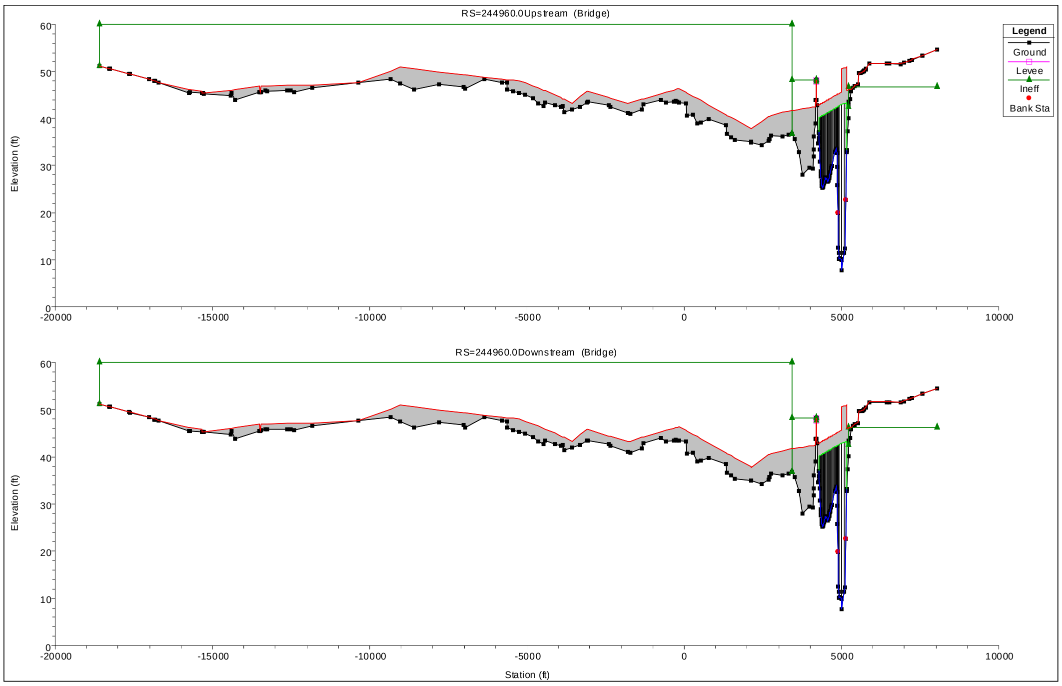


HEC-RAS Model Bridge Deck XS

The vertical blockage seems unrealistic although it may have little impact on the computed water surface elevations. The fill at the railroad bridge at Princeville is shown below. The fill profile does not seem realistic for a railroad crossing.

Comment 5 Response:

The rail road bridges were modeled with a portion, approximately the bottom 5 feet, of the bridge trestle written as solid. This is an attempt to account for debris blockage. The removal of this blockage on the rail road bridge at Princeville only dropped the 0.002 AEP event profile and the Floyd Flood profile by 0.03 and 0.06 feet, respectively. These profiles are so high that the effect on the economics and risk analysis is not noticeable.



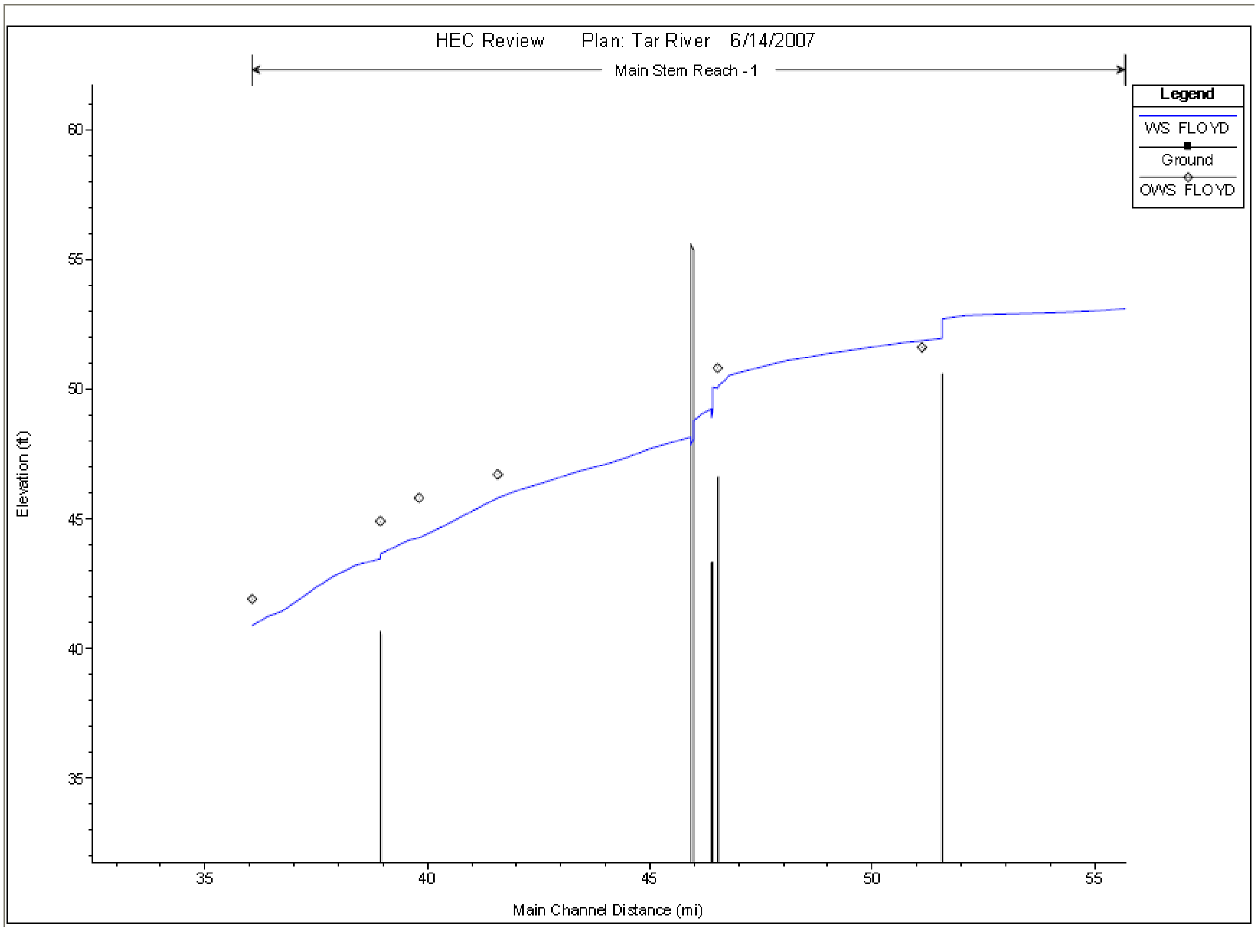
HEC-RAS Model Bridge Ineffective Flow Area

The entire left overbank is described as ineffective flow so the fill profile does not affect the computations, but the source of this fill profile should be explained in the notes for this structure.

Model calibration

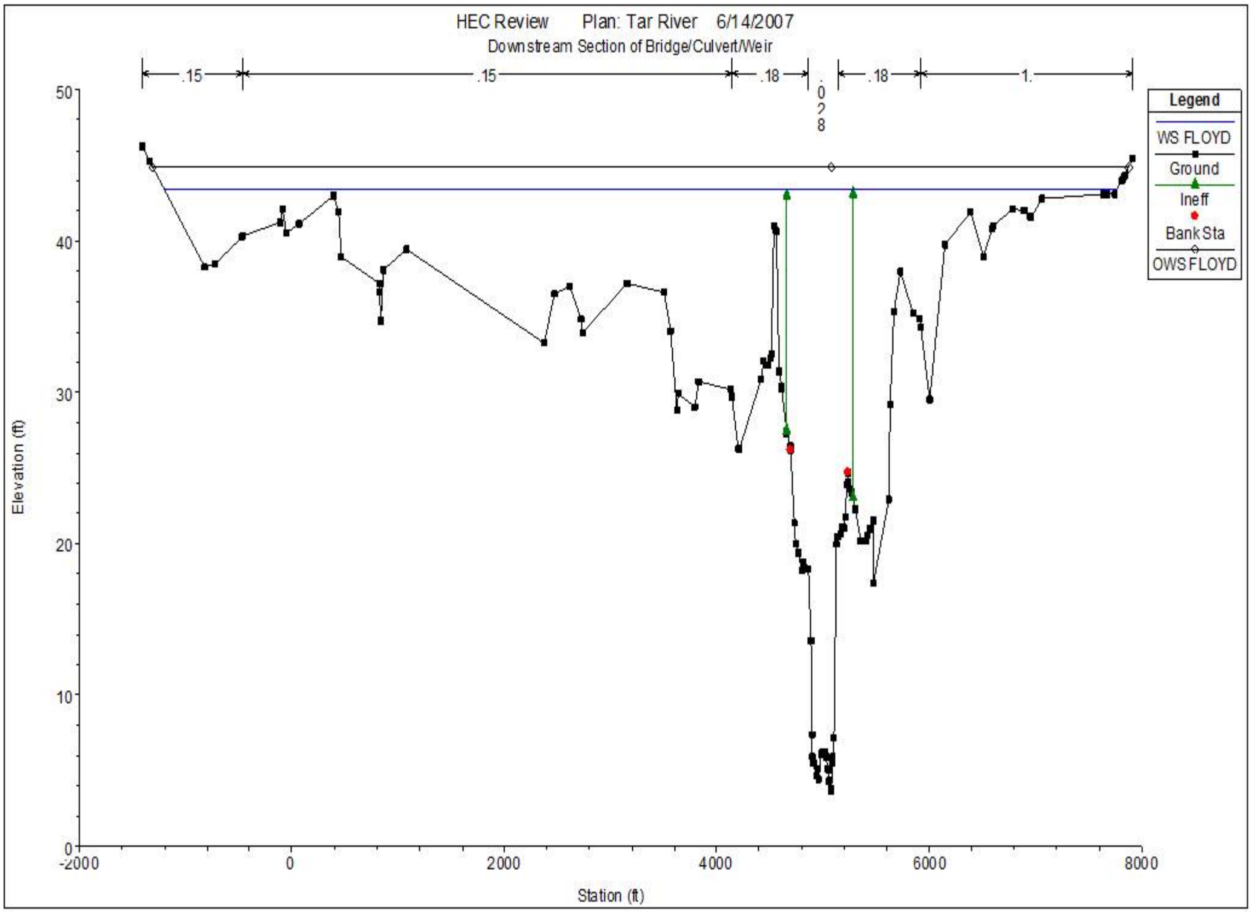
Comment 6

The observed data for hurricane Floyd were provided and compared to the simulations as shown below. The RAS profiles are lower than the observed data by about 1 foot in the lower 10 miles of the study reach.



Water Surface Elevation Comparison of Hurricane Floyd and HEC-RAS Model

The Manning's *n* values have been distributed across the cross sections in this area as shown below. The magnitudes of the *n*-values are realistic. The degree to which this profile is under predicted is not excessive; however this bias of the model should be acknowledged in its applications.



HEC-RAS Model Manning's n value Distribution

Georeferencing

Comment 7

The HEC-RAS model is not georeferenced; this capability is useful for presentation of inundation mapping and subsequent computation of economic consequences.

The following are additional comments on the M. Gee review of July 2007.

It should be remembered that this model, the Edgecombe County portion, is a portion of a larger model, part of it being downstream and part being upstream, in Pitt County and Nash County, respectively. This model was developed for a flood insurance study and had no requirement to be georeferenced. The calibration of the model was performed over the entire model, which covers Pitt, Edgecombe, and Nash Counties. The discharges were developed using a series of gages located in three counties. The lower end of the model may be predicting the Floyd elevations low because it does not include the possible effects of wind tide flooding from the Pamlico Sound.