FINAL REPORT

Master Stormwater Management Plan
Okaloosa County, Florida

Contract No.: C02-0634-PW2-55

Prepared By:

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Revision 1

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1.0 INTRODUCTION

1.1 PROJECT BACKGROUND

Okaloosa County is located in the northwest Florida panhandle. The County encompasses 995 square miles (60 square miles water, 935 square miles land) and includes a population of approximately 170,000 people. Okaloosa County is located along the Gulf of Mexico, extending north to the Alabama State line and contains two physiographic areas. Gently sloping plateaus at relatively higher elevations separated by lower, large stream valleys characterize the northern portion of the County. Lower elevations, barrier islands, lagoons, estuaries, and valleys characterize the southern portion of the County.

The streams and channels existing in Okaloosa County originate within the County as well as in Santa Rosa and Walton Counties in Florida and Escambia, Covington, Crenshaw, and Coffee Counties in Alabama. The Blackwater River, Yellow River, and Shoal River systems drain the majority of the County. The Blackwater River is located in the northwest portion of the County extending into Alabama to the north and Santa Rosa County to the west. The Yellow River flows from the Alabama state line to Eglin Air Force Base (AFB) at a northeast to southwest angle vertically through the center of the county. The Shoal River is located in the northeast portion of the County and extends into Alabama to the north and Walton County to the east. In addition to the three major river basins, two other principal watersheds exist in Okaloosa County, including the Choctawhatchee Bay and East Bay watersheds. These two watersheds are located in the southern part of the County and drain into the Choctawhatchee Bay and East Bay, respectively. The contributing drainage areas of all watersheds within Okaloosa County are shown in **Figure 1-1**.

Flooding periodically occurs along the streams and streets in Okaloosa County, with flood damage to streets, homes and businesses. As the County enjoys sustained growth through the years, runoff rates and flooding problems are likely to increase in many areas due to continued conversion of rural lands to urban uses.

Rainfall varies widely in Okaloosa County throughout the year. Data collected by Eglin AFB indicates that the monthly average is 5.1 inches and the yearly average is approximately 62 inches. The month experiencing the most rainfall is typically July followed by September, August, and June. The least rainfall occurs from October through February.

Urban development within a drainage area generally results in an increase in the percent impervious area, i.e., more hard surfaces, with a concurrent increase in runoff associated with any given storm event. Therefore, stream channels and culverts that were adequate prior to urbanization may become inadequate as the drainage area develops. This results in more

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Study Area Existing Features and Master Plan Elements

Figure 1-1

frequent stream channel flooding and backwater flooding from culverts unable to convey the higher discharges. Okaloosa County addresses these problems, as funds allow, through street and drainage improvement projects.

1.2 GOALS AND OBJECTIVES

This Master Stormwater Management Plan provides a framework describing stormwater processes in Okaloosa County. Specifically, the primary objectives of the study include the following:

1. Prepare calibrated large-scale hydrologic and hydraulic models for the main stems of the Blackwater, Yellow, and Shoal Rivers (Riverine Models).
2. Apply the Riverine Models, develop flood profiles along the main stems of the Blackwater, Yellow, and Shoal Rivers for the 2-, 10-, 25-, 50-, 100-, and 500-year return period storm events, considering both existing and future conditions.
3. Analyze eight areas identified by Okaloosa County for detailed study (Detailed Study Areas), including Foxwood Subdivision, Antioch Road, Meigs Drive, Commons Drive, Gap Creek, US 98 Box Culverts, Cimarron Outfall, and Lake Blake.
4. Conduct level of service (LOS) analyses at 67 structures (12 bridges and 55 culverts) identified throughout the County to provide an understanding of overall system performance.
5. Develop a pollutant loading model estimating the total annual pollutant loadings by sub-basin for four common pollutants, including Total Nitrogen (TN), Total Phosphorus (TP), Biochemical Oxygen Demand (BOD), and Total Suspended Solids (TSS).
6. Catalog all current repair and replacement projects.
7. Recommend improvements based on the results of the above analyses, provide cost estimates, and supplement the County's Capital Improvement Program (CIP).

Note that this Master Stormwater Management Plan addresses existing and projected flooding. Portions of the areas studied have been included in previous Federal Emergency Management Agency (FEMA) studies as shown in Figure 1-1. The Master Plan complements existing FEMA studies by using more developed source data, extending the modeling limits, and evaluating future development patterns.

1.3 RELATED COUNTY DOCUMENTS

1.3.1 National Pollutant Discharge Elimination System (NPDES) Program

Developed in two phases, the U.S. Environmental Protection Agency (EPA) federal NPDES stormwater permitting program, implemented by the Florida Department of Environmental Protection (DEP), regulates stormwater runoff from industrial activity, construction activity, and municipal separate storm sewer systems (MS4s). Promulgated in 1990, Phase I addresses discharges of stormwater runoff from industrial activity, "large" construction activity, and "medium" and "large" MS4s (i.e., those MS4s located in incorporated places and counties with populations of 100,000 or greater). Okaloosa County did not meet these requirements. Promulgated in 1999, Phase II addresses "small" construction activity and MS4s not regulated by Phase I that are classified as "urbanized" by the U.S. Bureau of the Census latest decennial data.

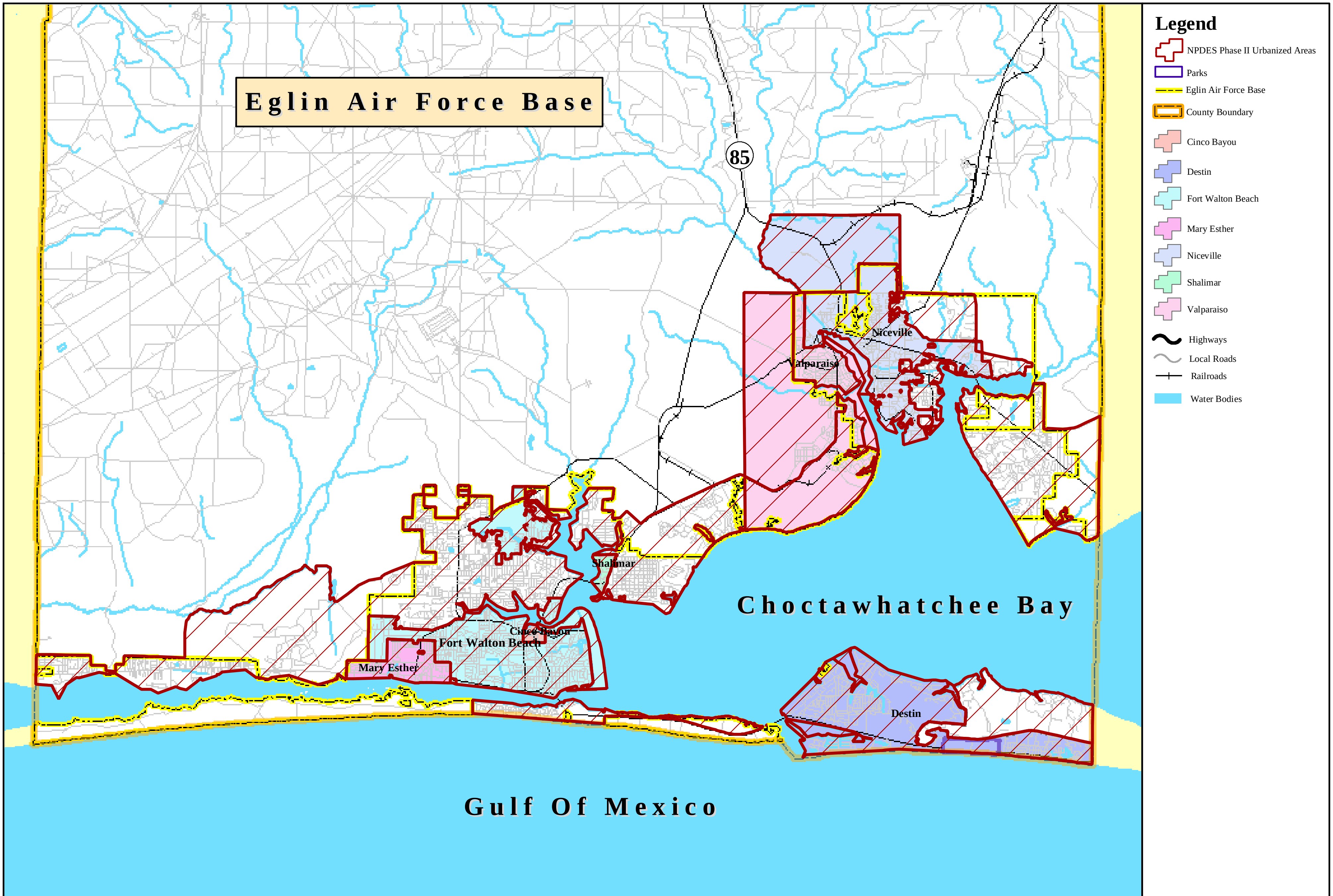
Figure 1-2 highlights the portion of unincorporated Okaloosa County classified as "urbanized" and therefore required to apply for an NPDES Phase II permit and implement a comprehensive stormwater management program to reduce the contamination of stormwater runoff and prohibit illicit discharges. Okaloosa County's draft Phase II MS4 Generic Permit Notice of Intent (NOI) was developed in connection with the Master Plan and is included in **Appendix A**.

1.3.2 Okaloosa County Comprehensive Plan

The Okaloosa County Year 2020 Comprehensive Plan outlines goals, objectives, and policies related to stormwater management, and indirectly addresses stormwater management through its land use, transportation, and coastal management sections. The goal of stormwater management based on the Comprehensive Plan is to "provide an environmentally safe and efficient stormwater management system." To achieve this goal the following objectives are outlined in the Comprehensive Plan:

<u>Objective 1</u> Correct existing stormwater management deficiencies by implementing improvements adopted in the 5-Year Schedule of Capital Improvements, developing and implementing a Stormwater Master Plan, and paving of roads according to adopted level of service standards.

<u>Objective 2</u> Coordinate the extension of or increase the capacity of stormwater management facilities to meet future needs. This shall be accomplished in part through enforcement of land development regulations that protect the quantity and quality of stormwater runoff and that ensure that the capacity of stormwater management structures for roads and other development are designed to meet facility needs.



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NPDES Phase II Urbanized Areas

Figure 1-2

Objective 3 The County shall protect natural functions of stormwater management features. This shall be accomplished in part through land development regulations and proper classification of land uses.

Objective 4 Discourage urban sprawl and maximize the use of existing stormwater management facilities through flexibility in the land development regulations to allow stormwater management facilities to serve more than one function and to promote the use of regional facilities where they will not contribute to urban sprawl.

1.3.3 Okaloosa County Land Development Code

The Okaloosa County Land Development Code (LDC) establishes regulations related to stormwater management primarily in Chapter 4, Consistency and Concurrency Determination; Chapter 5, Protected Area Standards; and Chapter 6, Development Design. The relevant stormwater sections of the LDC were revised within the same scope of work as this report.

Chapter 4 describes requirements and procedures designed to make proposed development projects consistent with the LDC and Comprehensive Plan. Section 4.02.05 specifically addresses stormwater and requires the following level of service standard:

> The level of service standard for stormwater on County roads shall be Level II – Street gutter systems are flowing full however ten to twelve feet of the road crown is not submerged and traffic can move at a slightly reduced speed. Stormwater swales and ditches are full with water overflowing the tops and edges in some locations. Water may be ponded eight (8) to ten (10) feet onto private property and yards. Inlets and culverts are flowing full to overfull slightly backing up water at entrances.

Section 5.02.05 Provision for Flood Hazard Reductions outlines the general and specific development standards in areas of special flood hazard.

Section 6.06.00 Stormwater Management contains performance objectives and design standards for stormwater management and is contained in **Appendix B**.

1.3.4 Local Mitigation Strategy

The Local Mitigation Strategy (LMS) provides guidance for both municipalities and unincorporated areas within the County in implementing several hazard mitigation initiatives. The LMS includes Goals, Objectives and Policies that support the following seven Guiding Principles and establishes a point-based system to judge the merits of proposed projects:

1. Protect human life and private property from the effects of disaster events.

2. Reduce public expenditures due to damage from disaster events.
3. Adopt land use regulations that support sustainable communities.
4. Protect environmentally sensitive areas.
5. Monitor and protect Natural Resources of Okaloosa County.
6. Mitigate potential losses through administrative measures.
7. Coordinate with private sector to mitigate losses.

The LMS Guiding Principles direct that the local governments establish policies and codes that support and implement both "structural and nonstructural" alternatives to reduce the risk disasters pose to life, public and private property and infrastructure. This document was also revised within the same scope of work as this report.

1.4 ORGANIZATION OF MASTER STORMWATER MANAGEMENT PLAN DOCUMENT

The Master Stormwater Management Plan is divided into nine main sections. Section 1 is the introduction. Section 2 outlines the methodologies used with regard to hydrologic model development, hydraulic model development, and the LOS analysis. Sections 3, 4, and 5 describe the three major riverine watersheds in Okaloosa County including the Blackwater River, Yellow River, and the Shoal River Watersheds. Each watershed description includes the general characteristics of the watershed, flood hydrology results, hydraulic model results, LOS analysis results, analyses of all detailed study areas identified for the watershed, and appropriate recommendations. Section 6 includes the coastal basins (i.e., Choctawhatchee Bay and East Bay). This section features analyses and discussions similar to those presented in connection with the three river basins. Section 7 summarizes the methodology and results of the pollutant loading model prepared as part of this study, and Section 8 summarizes the recommendations of Sections 3 – 7 and ranks the projects. Section 9 addresses the funding of drainage improvements and operations.

2.0 HYDROLOGIC AND HYDRAULIC DATA AND METHODOLOGIES

The following sections describe the data and methodologies used in this study.

2.1 DATA DEVELOPMENT

2.1.1 Physical Characteristics

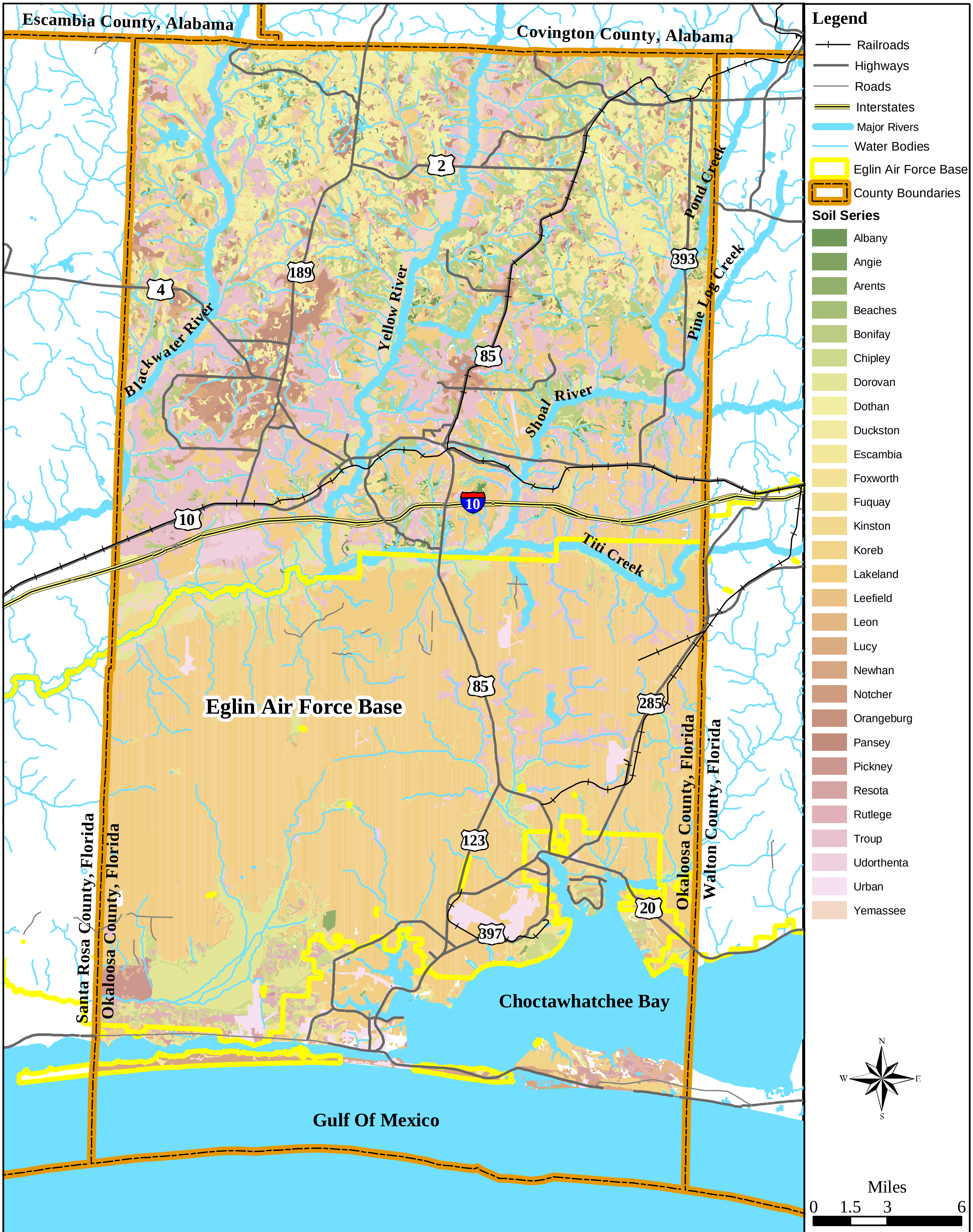
2.1.1.1 Topography

Topography of a drainage area refers to relief of the land surface, and is used to determine hydrologic and hydraulic input parameters relating to slope and elevation. Topography for the project originated from two principal data sets, including 30-meter digital elevation models (DEMs) from the United States Geological Survey (USGS) National Elevation Dataset (NED), and high-resolution triangular network (TIN) terrain models (County TINs), developed through photogrammetry and provided by Okaloosa County. The 30-meter DEMs cover all of the watersheds within the study area. These DEMs were used to delineate basins draining to the Blackwater, Yellow, and Shoal Rivers, and to compute initial hydrologic parameters such as lag time (which is based on slope). The County TINs cover the main river floodplains and the detailed study areas identified by the County. These TINs were used to develop stream cross sections, compute detention volumes, delineate basins impacting the detailed study areas, and map the limits of flooding based on model results.

2.1.1.2 Soil Types

Okaloosa County consists of three broad soil groups characterized by distinctive patterns of soils, relief, and drainage including soils of the upper coastal plain which are primarily located north of Eglin AFB, soils of the barrier islands and coastal plains which are located within Eglin AFB, and soils of the flatwoods, low knolls, and ridges which are located south of Eglin AFB. The primary difference between the broad soil groups is: the soils of the upper coastal plain exist in broad flat areas and on side slopes in the uplands, the soils of the barrier island and coastal plains exist on high dune ridges and in high upland areas, and the soils of the flatwoods, low knolls, and ridges exist in broad areas of flatwoods surrounded by poorly defined drainageways and depressions, and on low knolls and ridges. Each of these three broad groups is characterized by the soil series shown in **Figure 2-1**.

The types of soils present in a drainage area have a significant impact on the amount of runoff a given storm will produce. This impact is influenced primarily by the infiltration characteristics of the soil. One generalized measure of the infiltration characteristics of a soil commonly used



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Okaloosa County Soil Types

Figure 2-1

in developing hydrologic models is the Hydrologic Soil Group. This system categorizes soils into four groups based on expected rates of infiltration with Hydrologic Soil Group A representing well-drained soils and Hydrologic Soil Group D representing poorly drained soils.

Information on soil types and characteristics was obtained through the Soil Survey Geographic (SSURGO) and State Soil Geographic (STATSGO) databases. The SSURGO database is a digital version of the detailed, 1:24,000 scale soil survey maps created by the National Resource Conservation Service (NRCS, formerly the Soil Conservation Service (SCS)). The STATSGO database is a digital version of the 1:250,000 scale generalized soil maps also created by NRCS.

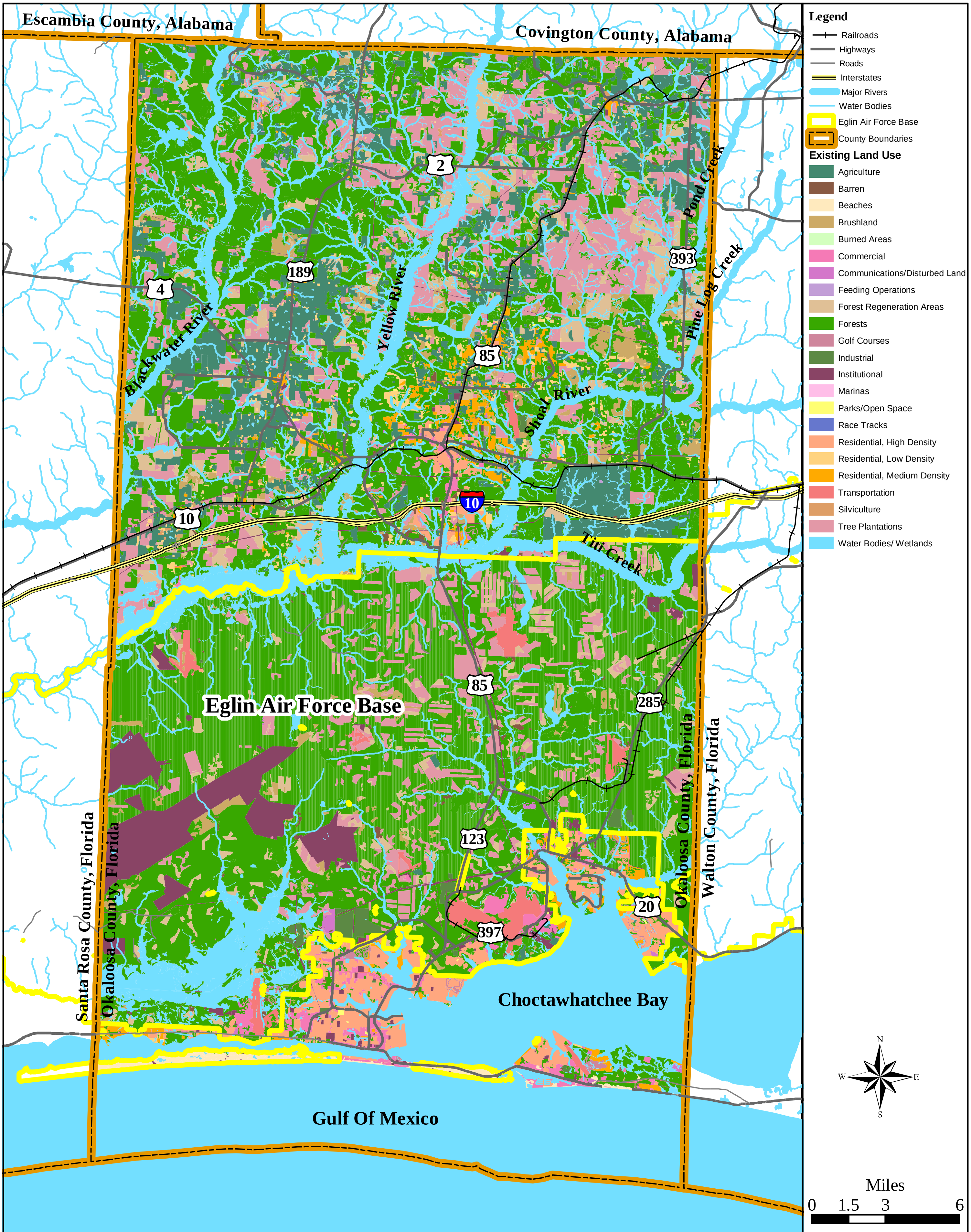
SSURGO soils data has not yet been compiled for the counties in Alabama contained within the project watersheds. For this reason, a merged database consisting of STATSGO data supplemented by SSURGO data, where available, was produced for the project containing the hydrologic soil types for the entire study area. Chapters 3 through 6 contain Figures illustrating the Hydrologic Soil Groups applied to each of the watersheds.

2.1.2 Land Use

Land use is a critical element for stormwater planning, impacting both the quantity and quality of runoff. The effect land use has on water quantity is generally linked to the amount of impervious area for a particular land use category. In general, an area with a higher percentage of impervious area will have a quicker time to peak (t_p) and a higher associated peak runoff rate (Q_p).

2.1.2.1 Existing Land Use

The existing land use data used for this study was initially prepared by the Northwest Florida Water Management District (NWFWMD) in 1995 using the Florida Land Use, Cover, Forms, and Classification System (FLUCCS). **Table 2.1** shows the FLUCCS codes and land use descriptions as grouped for the hydrologic and hydraulic (H&H) models. **Figure 2-2** provides a graphic representation of the information presented in Table 2.1.



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Okaloosa County Existing Land Use

Figure 2-2

Table 2.1
Existing Land Use Codes and Descriptions

FLUCCS Code	Land Use Description
2100, 2150, 2400	Agriculture
1600, 1610, 1620, 1660	Barren
3220, 7100, 7200	Beaches
3200	Brushland
7450	Burned Areas
1400, 1420, 1440, 1450, 1750	Commercial
1900, 7400, 8200, 8210, 8220, 8350	Communications/Disturbed Land
2300	Feeding Operations
4430	Forest Regeneration Areas
4100, 4130, 4200, 4340	Forests
1820	Golf Courses
1500, 1890, 8300, 8310, 8330, 8340	Industrial
1710, 1720, 1730, 1800, 8170, 8320	Institutional
1840	Marinas
1480, 1850, 1860, 1870, 2600	Parks/Open Space
1830	Race Tracks
1300, 1320	Residential, High Density
1100, 1120	Residential, Low Density
1200, 1220, 1760	Residential, Medium Density
2200	Silviculture
7300, 8100, 8110, 8140	Transportation
4400, 4410	Tree Plantations
2540, 5000-6900, 7500	Water Bodies/Wetlands

As shown in Figure 2-2 the County appears to be stratified into three distinct land use regions: the area south of Eglin AFB, Eglin AFB, and the area north of Eglin AFB. Those parts of the County bordering on Choctawhatchee Bay and the Gulf of Mexico, south of Eglin AFB, are heavily urbanized. Residential uses dominate, although commercial uses are common in town centers and along major roadways. Eglin AFB occupies the center of the County. Most of the land within Eglin AFB consists of upland forest or clearcuts in various stages of regeneration. Runway facilities are scattered around the Eglin AFB reservation, and several large cleared areas used for military testing are located in the western part of the base. North of Eglin AFB, the City of Crestview features mostly residential development, while silvicultural, agricultural and forest cover predominates throughout the rest of the northern region of Okaloosa County. Wetlands are largely located in the floodplains of the major river systems in the northern County, except for one large wetland system in the southwestern area of Eglin AFB, northwest of Fort Walton Beach. **Table 2.2** lists the existing land use classifications used for H&H models and the percentage of the County occupied by each land use.

Table 2.2
Okaloosa County Existing and Future Land Use Summary

Land Use Group	Existing %	Future %
Agriculture	4	2
Barren	<1	<1
Beaches	<1	<1
Brushland	<1	<1
Commercial	<1	<1
Communications/Disturbed Land	<1	<1
Forests	79	81
Forest Regeneration	1	1
Golf Courses	<1	<1
Industrial	<1	<1
Institutional	2	2
Marinas	<1	<1
Parks/Open Space	<1	<1
Residential, High Density	1	<1
Residential, Low Density	<1	<1
Residential, Medium Density	<1	<1
Transportation	<1	<1
Tree Plantations	3	3
Water Bodies/Wetlands	7	7
Total	**100**	**100**

2.1.2.2 Future Land Use

The future land use data used in this study was based on the County's future land use data as adopted in the Comprehensive Plan. However, the County's future land use map is very generalized, and it does not reflect the same level of detail shown in the existing land use data or that needed for the H&H models. Accordingly, the future land use database and existing land use database were overlaid using geographic information systems (GIS). Those areas that the existing land use data indicated were already urbanized were assigned a future land use equivalent to their existing land use. Non-urban areas were assigned the appropriate future land use designation from the County's future land use map. Although this technique does not account for the transition of urban land uses from a non-conforming use to a different urban land use type as indicated by the County's future land use map, it produces a more rational final product than if none of the existing land uses were assumed to persist into the future.

As the County's future land use map only includes areas within the County's jurisdiction an additional step was taken to include the municipalities' future land use. Using future land use maps obtained from municipal comprehensive plans, future land use categories were assigned to areas without a future land use already determined. **Figure 2-3** shows the future land use throughout Okaloosa County grouped by classifications used for the H&H models. Table 2.2 lists land use classifications and the percentage of the County occupied by each land use.

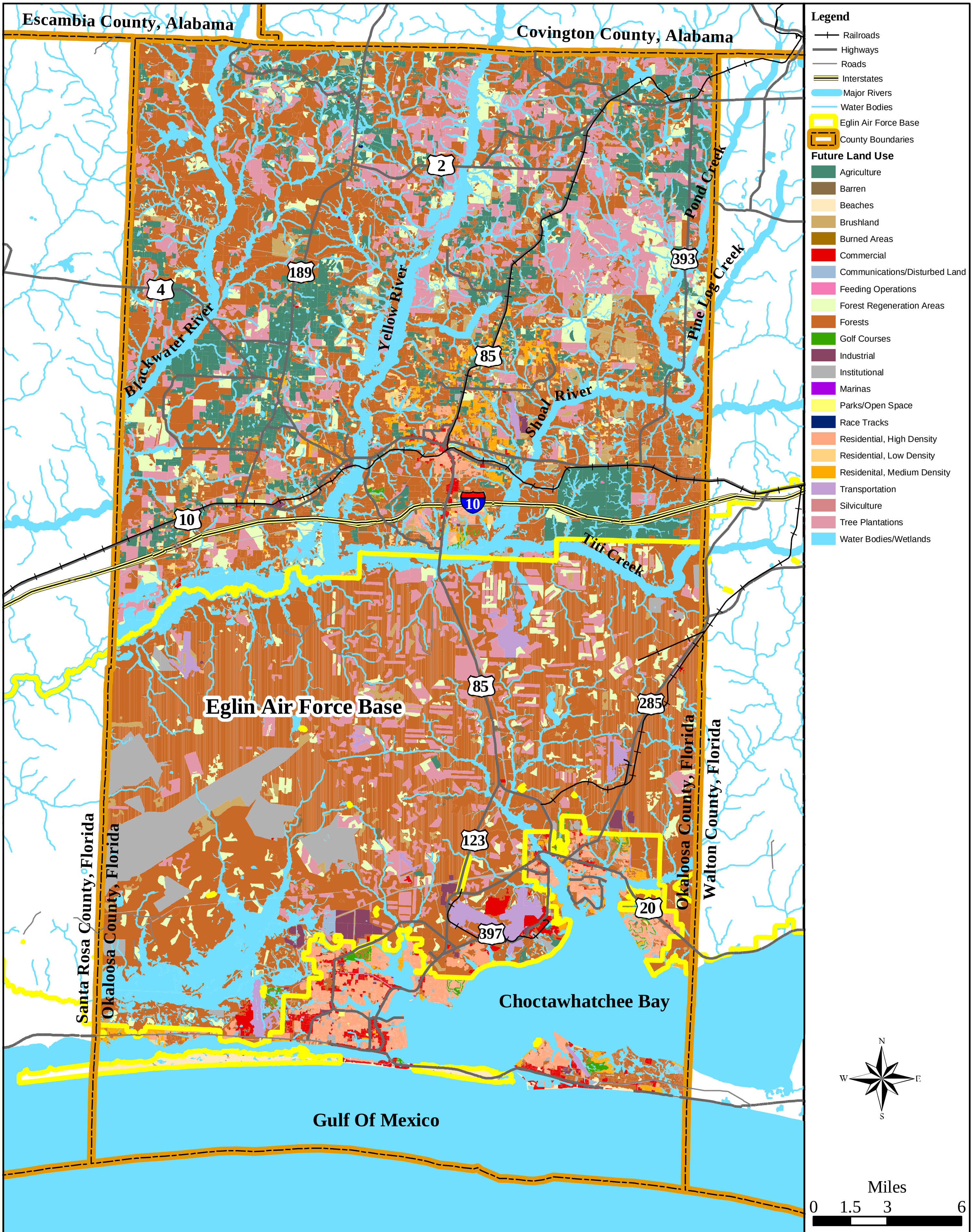
Comparison of Figures 2-2 and 2-3 and Table 2.2 reveal that the differences between the existing and future land use are minor. As the southern part of the County is already largely saturated with development, little change between the existing and future land use conditions exists. Most of the new urban acreage in the future land use map resulted from the conversion of forest lands to low density residential in the vicinity of Crestview, Laurel Hill, and the SR 4/SR 189 intersection. In addition to residential development, substantial future increases in commercial uses appeared along the I-10 and SR 85 corridors in Crestview as well as industrial development along I-10 and US 90 east of Crestview and in eastern Crestview itself. However, the overall quantitative distribution of land use types changed very little.

2.1.3 Historical Streamflow Data

Historical streamflow data has been collected by USGS for many streams throughout Florida. **Table 2.3** shows information related to the four gauging stations available in the study area. These stations provided information used to calibrate the hydrologic model to field conditions.

Table 2.3
Existing USGS Gauging Stations

USGS Number	Location	Period of Record
02370000	Blackwater River near Baker, FL	1951-2000
02368000	Yellow River at Milligan, FL	1939-1998
02368500	Shoal River near Mossy Head	1952-1989
02369000	Shoal River near Crestview	1939-1999



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Okaloosa County Future Land Use

Figure 2-3

HDR

2.1.4 Historical Stage-Discharge Data

Both USGS and NWFWMD have collected stage-discharge data within the Blackwater, Yellow, and Shoal River basins. **Table 2.4** identifies available data, which was used to calibrate the hydraulic model to field conditions.

Table 2.4
Historical Stage-Discharge Data

Agency	Number	Location	Period of Record
USGS	02370000	Blackwater River near Baker, FL	Recorded observations
USGS	02368000	Yellow River at Milligan, FL	Recorded observations
USGS	02368500	Shoal River near Mossy Head	Recorded observations
USGS	02369000	Shoal River near Crestview	Recorded observations
NWFWMD	365	Yellow River at SR 2	Rating curve
NWFWMD	511	Shoal River at US 90	Rating curve

2.1.5 Precipitation Data

2.1.5.1 Reconstituted Storm

Stormwater models are typically calibrated to a historical storm event allowing a comparison of predicted response to field observations. The calibrated model can then be used with hypothetical storms of the desired return frequencies.

On March 8, 1998 a weather system moving through southwest Alabama and northwest Florida produced significant flood stages in the Blackwater, Yellow, and Shoal Rivers. Precipitation was estimated between six and ten inches throughout the area. Heavy rains began in the early morning in Escambia County and moved east across Santa Rosa into Okaloosa County in the mid-morning. This storm was selected as the historical calibration event.

An investigation of available rain gage data for the March 8 storm and other events revealed that insufficient gage data exists for model development. As a solution, the calibration storm was reconstituted from radar reflectivity, hourly radar-estimated rainfall data, and upper air atmospheric soundings and surface observation data available from the National Weather Service (NWS). This process involved overlaying a GIS grid over the entire basin, applying an algorithm to raw reflectivity data that converts the radar data to precipitation, and calibrating the result to available gages. This technique produced a rainfall distribution that can be applied to each sub-basin within the watershed. A detailed description of the procedures applied appears in **Appendix C**.

It should be noted that other historical storms produced a larger flood response, including recent hurricanes. However, high winds degrade reflectivity data. Accordingly, the March 8 event was considered more appropriate due to a presumption of higher quality reflectivity data.

2.1.5.2 Design Storms

Design rainfall was developed from three sources including Hydro-35, TP 40, and TP 49 for the 2-, 10-, 25-, 50-, 100-, and 500- year frequencies, as described in **Table 2.5**. Note that the 500-year rainfall values were extrapolated by log regression.

Table 2.5
Okaloosa County Total Rainfall Depths (in)

Source	Frequency	Return Period					
		2-year	10-year	25-year	50-year	100-year	500-year[1]
HYDRO-35	5-min	0.55	0.67	0.75	0.82	0.88	1.01
HYDRO-35	15-min	1.20	1.49	1.68	1.84	1.99	2.30
HYDRO-35	1-hr	2.25	3.14	3.67	4.09	4.50	5.41
TP 40	2-hr	2.6	4.1	4.6	5.1	5.6	6.9
TP 40	3-hr	3.2	4.6	5.2	5.8	6.4	7.7
TP 40	6-hr	4.1	6.0	6.6	7.4	8.3	9.9
TP 40	12-hr	4.9	7.1	8.3	9.2	10.4	12.5
TP 40	24-hr	6.0	9.5	11.0	12.0	13.5	16.5
TP 49	48-hr	6.6	10.0	12.0	13.0	14.0	17.4
TP 49	96-hr	8.0	11.5	13.5	14.5	16.2	19.5

1. Extrapolated

2.2 HYDROLOGIC MODEL

The U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) Hydrologic Modeling System (HEC-HMS) was selected to simulate the precipitation-runoff processes of the Blackwater River, Yellow River, Shoal River, and three of the Detailed Study Areas (Gap Creek, Cimarron Outfall, and Commons Drive). HEC-HMS is designed to simulate the surface runoff response of a drainage basin to precipitation input.

2.2.1 Hydrologic Network

HEC-HMS represents a watershed as an interconnected system of hydrologic elements known as a hydrologic element network. Available hydrologic elements represent components of the watershed response and include sub-basins, reaches, reservoirs, junctions, diversions, sources, and sinks. Hydrologic elements are connected to form a node and reach system that collectively represents physical processes occurring in the watershed.

The most common hydrologic elements are sub-basins, junctions, and reaches. Sub-basins produce runoff for the model from meteorologic data, considering losses, baseflow, and the transformation of excess precipitation to runoff. Junctions represent discrete locations in the system where conservation of mass or continuity is maintained and are generally located:

1. At major structures (e.g., bridges, culverts etc.)
2. At ponds and lakes (specifically storage nodes)
3. At stream confluences
4. Coincident with the downstream boundary, and
5. Where major surface inflows to the conveyance system occur.

Reaches connect junctions and other nodes, typically represent rivers and streams, and include information regarding channel geometry, slope and roughness.

Applying these guidelines, a hydrologic network was created for each of the modeled watersheds. Schematic diagrams describing the hydrologic network constructed for each watershed appear in Hydrologic and Hydraulic Appendices.

2.2.2 Loss Rate

Precipitation falling on a pervious surface experiences losses due to infiltration. HEC-HMS features seven methodologies for computing losses including deficit and constant, Green and Ampt, gridded SCS curve number, gridded soil moisture accounting, initial and constant, SCS curve number, and soil moisture accounting. Precipitation adjusted for losses due to interception, transpiration, and infiltration is known as excess precipitation.

The SCS Curve Number method was selected to account for losses. This method estimates excess precipitation as a function of cumulative precipitation, soil cover, land use, and antecedent moisture using the following equations:

$$P_e = \frac{(P - 0.2S)^2}{P + 0.8S}$$

$$S = \frac{1000}{CN} - 10$$

where P_e is the excess precipitation, S is the maximum retention, and CN is the curve number. Applying these equations, HEC-HMS computes incremental excess precipitation using cumulative precipitation and cumulative excess precipitation at the end of each model time step.

The CN for a watershed can be estimated from land use, hydrologic soil group, and (AMC) moisture conditions, using published data. **Table 2.6** summarizes the CNs used during initial model development.

Note that the selected curve numbers were based on data presented in NRCS Technical Report 55 (TR-55).

2.2.3 Runoff Transforms

2.2.3.1 Selected Transforms

A runoff transform is a methodology used to convert excess precipitation into direct runoff. HEC-HMS provides six transform procedures including a kinematic wave model, the ModClark quasi-distributed linear transform, and four empirical unit hydrograph techniques including Clark, Snyder, SCS, and user specified. It is noted that transform methods are independent of loss methods such that the use of SCS methodology to compute losses does not require the use of the SCS empirical unit hydrograph for transform computations.

The SCS unit hydrograph technique was used for Gap Creek, Cimarron Outfall, and Commons Boulevard. However, the Clark unit hydrograph was selected for the Riverine Models. The use of the Clark unit hydrograph for the Riverine Models allowed better control of the hydrograph shape and a resulting closer match to observed streamflow data.

Table 2.6
Curve Numbers by Land Use and Hydrologic Soil Group

Land Use Description	Hydrologic Soil Group			
	A	B	C	D
Agriculture	67	78	85	89
Barren	0	0	0	0
Beaches	25	25	25	25
Brushland	30	48	65	73
Burned Areas	48	67	77	83
Commercial	89	92	94	95
Communications/Disturbed Land	77	86	91	94
Feeding Operations	59	74	82	86
Forest Regeneration Areas	57	73	82	86
Forests	36	60	73	79
Golf Courses	39	61	74	80
Industrial	81	88	91	93
Institutional	68	79	86	89
Marinas	95	95	95	95
Parks/Open Space	49	69	79	84
Race Tracks	70	80	85	87
Residential, High Density	77	85	90	92
Residential, Low Density	54	70	80	85
Residential, Medium Density	61	75	83	87
Silviculture	32	85	72	79
Transportation	98	98	98	98
Tree Plantations	43	65	76	82
Water Bodies/Wetlands	100	100	100	100

2.2.3.2 SCS Unit Hydrograph Procedure

The unit hydrograph is a commonly used empirical representation of the relationship between direct runoff and excess precipitation. The unit hydrograph expresses the basin outflow with respect to time. In this manner, the unit hydrograph "transforms" excess precipitation into a time-distributed representation of direct runoff.

The timing and shape of the SCS unit hydrograph depends upon the basin time to peak, t_p. The basin time to peak is defined as the time from the beginning of the rainfall event to the time at which the peak runoff rate is observed at the drainage area outlet. The time to peak can be estimated using the following empirical equation:

$$t_p = \frac{\Delta D}{2} + t_{lag}$$

where: t_p = time to peak, in hours
ΔD = duration of excess precipitation, in hours
t_{lag} = lag time, in hours

The lag time is defined as the time difference between the center of mass of the rainfall excess and the peak of the unit hydrograph. The Lag time is given by the following equations:

$$t_{lag} = \frac{L^{0.8}(S+1)^{0.7}}{1900\ Y^{0.5}}$$

$$S = \frac{1000}{CN} - 10$$

where: t_{lag} = lag time, in hours
L = greatest flow length, in feet
Y = average drainage area slope, in percent
CN = runoff curve number, based on land use, land treatment and soil type

2.2.3.3 Clark Unit Hydrograph Procedure

The Clark unit hydrograph method simulates the translation and attenuation of excess precipitation as it moves across the basin. The procedure utilizes a synthetic time-area histogram and time of concentration to represent translation, and a linear reservoir model to account for attenuation.

Application of the Clark unit hydrograph procedure requires input of the time of concentration t_c, and a storage coefficient R. The storage coefficient is an index of the temporary storage of precipitation excess in the watershed and has units of time. R is computed from observed data by dividing the flow at the inflection point on the falling limb of the observed streamflow hydrograph by the time derivative of flow.

2.2.4 Channel Routing

HEC-HMS provides six models to simulate the routing of a hydrograph through a channel reach, including Kinematic Wave, Lag, Modified Puls, Muskingum, Muskingum-Cunge Eight-point Section, and Muskingum-Cunge Standard Section. The Muskingum-Cunge Eight-point Section model was selected for this study.

The Muskingum-Cunge Eight-point section methodology requires the definition of a typical cross section for each channel reach, described by eight station-elevation coordinates. The procedure divides each cross-section into three parts, including left overbank, channel, and right overbank. A Manning's roughness coefficient is entered for each section based on channel roughness and floodplain roughness observed during field reviews.

2.2.5 Meteorologic Model

Meteorologic data is entered into HEC-HMS pursuant to one of six different historical and synthetic precipitation models. Historical data can be analyzed using gage weighting, inverse-distance gage weighting, gridded precipitation, or a user-specified hyetograph. Synthetic precipitation can be generated using the frequency storm approach , the SCS hypothetical storm, or the standard project storm included with HEC-HMS.

With regard to the Riverine Models, the user hyetograph method was used to describe the reconstituted storm. This procedure allowed the assignment of a separate hyetograph to each sub-basin, providing accurate input of the compiled radar reflectivity data. The frequency storm approach was used for the synthetic storms. Use of the frequency storm approach allowed control over storm centering, which provided flexibility during calibration.

With respect to the Gap Creek, Cimarron Outfall, and Commons Boulevard models, the SCS hypothetical storm was used to produce the synthetic storms. The reconstituted storm was not run in these models due to the lack of recorded response to the event.

2.2.6 CRWR-PrePro

CRWR-PrePro (PrePro) is a GIS preprocessor for HEC-HMS developed by the Center for Research and Water Resources (CRWR) at the University of Texas, Austin, under the supervision of Dr. David Maidment. PrePro is a GIS hydrologic data preprocessing tool used to summarize data from a GIS system for input to HEC-HMS. PrePro was used to develop the watershed basin components for the Blackwater, Yellow, and Shoal River Basins. Specifically, PrePro aided in basin delineation, the computation of lag times, and the assignment of curve numbers based on land use and soil type.

2.2.7 Model Calibration

The HEC-HMS model was calibrated to the known flood event of March 8, 1998. To begin this process, the rainfall time-intensity information, as obtained from NWS, was inserted into each of the respective Riverine hydrologic models. The resultant simulated hydrographs, as computed by HEC-HMS, were then compared to the recorded flood hydrographs from the appropriate USGS stream gages. Hydrograph reconstitution was judged on matching the observed peak discharge, time of peak discharge, and flood hydrograph volume. Adjustments to applicable model parameters were made as necessary to allow a closer match of each or all of these three features (peak, timing, and volume). For example, should the simulated hydrograph reflect less volume than the observed, the curve number for the sub-basins would be adjusted to a higher value in order to lower the losses and increase the predicted volume.

After successful storm reconstitution, each of the respective Riverine hydrologic models was then calibrated to the USGS stream gage information. To accomplish this, both peak discharge and volume-duration-frequency analyses were performed on streamflow data from the four USGS gages listed in Table 2.3 using the USACE Flood Frequency Analysis (FFA) Program Version 3.1. This produced computed peak flows and flood volumes statistically expected for each flood frequency. The Riverine hydrologic models were calibrated to the expected peak flow and volume-duration as predicted by FFA. Details regarding the calibration of each model, as well as an analysis of the goodness of fit appear in the following chapters.

It should be noted that no stream gages exist serving Gap Creek, Cimarron Outfall, or Commons Drive. Accordingly, these models were not calibrated to existing data.

2.3 HYDRAULIC MODEL

The steady flow component of the USACE River Analysis System (HEC-RAS) was selected to perform hydraulic simulations of the Blackwater River, the Yellow River, the Shoal River, Gap Creek, Commons Boulevard, and Cimarron Outfall. The steady flow component of HEC-RAS performs one-dimensional gradually-varied calculations for natural or constructed open channels, and produces water surface profiles. The component considers the effects of obstructions such as bridges, culverts, and weirs.

2.3.1 Model Development

2.3.1.1 Geometric Data

Much like HEC-HMS, HEC-RAS requires a river system schematic consisting of junctions and reaches. After the schematic is drawn, cross-section and hydraulic structure data is entered. Cross-section geometry was obtained from the County TINs using the USACE HEC-GeoRAS software, which electronically aids in the generation of HEC-RAS cross-section input files within ArcView. Hydraulic structure data was obtained from field survey performed by Okaloosa County personnel, and from as-built plans. All other hydraulic parameters, such as reach lengths and Manning's Roughness coefficients, were obtained from County GIS data, field observations, or aerial photography.

2.3.1.2 Flow Data

Flow data was input into the hydraulic models using results from the calibrated HEC-HMS models. Normal depth of flow was used as a boundary condition for all models except Gap Creek and Cimarron Outfall, which used mean high tide.

2.3.2 Computations and Calibration

2.3.2.1 Riverine Models

Using steady flow techniques, water surface profiles were computed for the 2-, 10-, 25-, 50-, 100-, and 500-year return period flood events. These profiles were calibrated to the historical stage-discharge data presented in Section 2.1.4 by tuning hydraulic parameters such as Manning's n coefficients.

2.3.2.2 Gap Creek

Using steady flow techniques, water surface profiles were computed for the 2-, 10-, 25-, 50-, and 100- year return period flood events. Although no recorded stage-discharge data exists for Gap Creek, Martin Luther King Boulevard was observed to overtop during Tropical Storm Isadora, which produced precipitation equivalent to a 10-year return period. This information was used to calibrate the upper part of the basin.

2.3.2.3 Cimarron Outfall

Using steady flow techniques, water surface profiles were computed for the 2-, 10-, 25-, 50-, and 100-year return period flood events. Although no recorded stage-discharge data exists for the Cimarron Outfall, Parish Road is know to overtop frequently, with a one- to two-year return period. This information was used to calibrate the Cimarron Outfall model.

2.4 LEVEL OF SERVICE ANALYSES

LOS analyses were conducted for 67 structures located throughout Okaloosa County as defined in **Figure 2-4**. This analysis serves as a screening of selected structures throughout the County to provide an understanding of overall system performance. Analysis of the culverts assumes inlet control to facilitate the screening process, and consisted of the following steps:

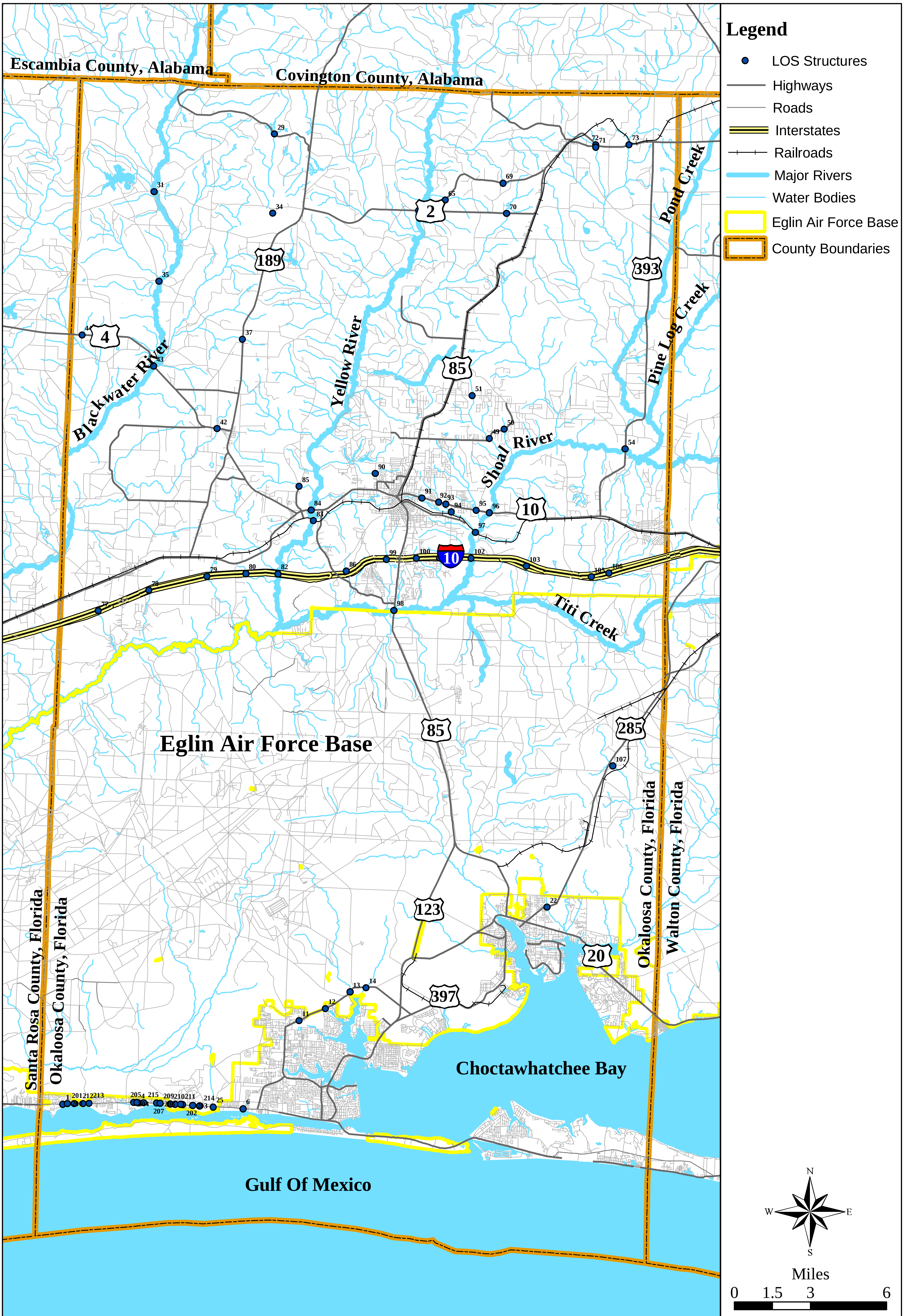
- Data Collection - Edge of pavement (EOP) elevations, flow line elevations, and culvert dimensions were collected by Okaloosa County using County personnel.

- Allowable Headwater – Using the data listed above the allowable headwater was determined for each structure. The allowable headwater was defined as the highest headwater condition that would not encroach on travel lanes.

- Allowable Discharge – Assuming inlet control, the allowable discharge was determined based on the allowable headwater, culvert dimensions, and inlet control nomographs included in *HDS-5, Hydraulic Design of Highway Culverts* (FHWA 1985).

- Discharge for Various Storm Frequencies - The discharges for 2-, 10-, 25-, 50-, 100-, and 500-year storms was calculated using either the Rational Method for drainage areas less than 590 acres, or the USGS Regression Equations for drainage areas greater than 590 acres.

- Comparison of Overtopping and Actual Discharges - If the analyses showed that the culvert overtopped more frequently than permitted by the criteria stated in the County's LDC and shown in **Table 2.7**, then a recommendation was made to alleviate the problem.



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Culvert and Bridge Structures

Figure 2-4

Table 2.7 Roadway Overtopping Design Storms	
Roadway Classification	**Design Storm**
Arterial	50
Collector	25
Local	10

3.0 BLACKWATER RIVER BASIN

3.1 GENERAL BASIN DESCRIPTION

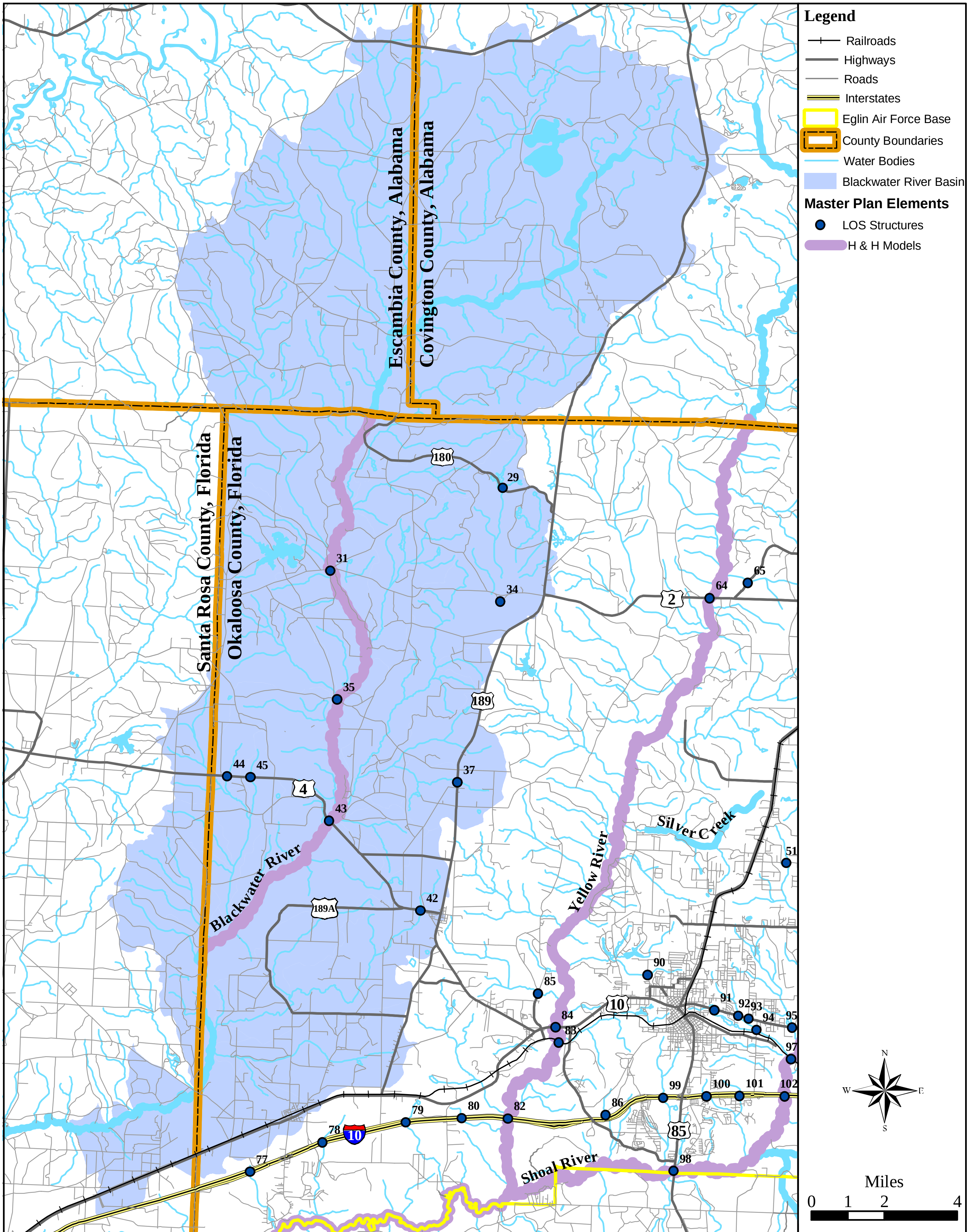
The Blackwater River Basin is located in the northwest portion of the County and is shown in **Figure 3-1**. The drainage basin measures approximately 286 square miles, of which 143 square miles are within the County boundary. Portions of the basin extend into Santa Rosa County and to the north into Alabama. The basin is roughly bounded by SR 189 to the east and I-10 to the south.

Elevations in the basin range from approximately 25 feet in Santa Rosa County to 340 feet in Alabama. Within Okaloosa County the elevations range from approximately 30 feet to 280 feet.

Table 3.1 shows the relative representation and general hydrologic characteristics for the soils found in the Blackwater River Basin within Okaloosa County. Within Okaloosa County, the Blackwater River Basin contains 20 different soil types, of which the Troup and the Dothan series account for close to 50 percent of the total basin area. The majority of the Troup series are located in the southern half of the basin while the Dothan series are primarily located in the northern half of the basin. The Kinston series, which accounts for approximately 10 percent of the basin area is primarily located along the river channel. For modeling purposes, the different soil types were grouped by NRCS hydrologic soil type as Type A, B, and C. Nearly the entire basin consists of Type B soils as depicted in **Figure 3-2**.

Land use classifications in the Blackwater River Basin range from forests to residential, with the majority of the basin classified as forest and agriculture land uses. The breakdown of existing land use (grouped by classifications used for the H&H models) within the Blackwater River Basin is shown in **Figure 3-3**.

Figure 3-4 shows the future land use (grouped by categories used for H&H models) within the Blackwater River Basin based on the County's future land use map, the municipalities' future land use maps, and existing land use data where necessary as discussed in Section 2.1.2.2. As shown the future land use is quite similar to the existing land use and there is not increase in I permeable land use. **Table 3.2** shows a comparison of the percentage of each land use classification for both existing and future conditions.



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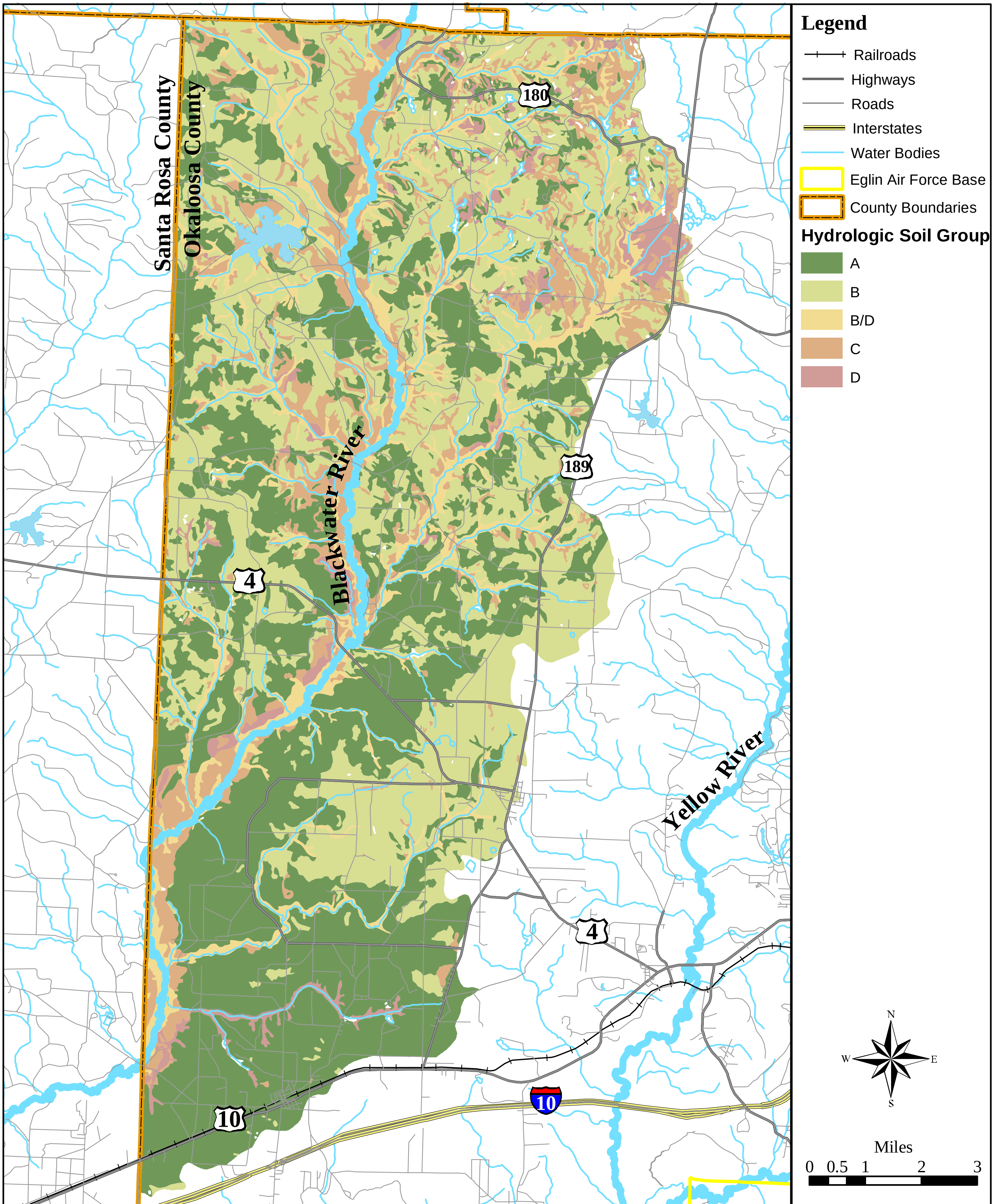
Blackwater River Basin

Figure 3-1

HDR



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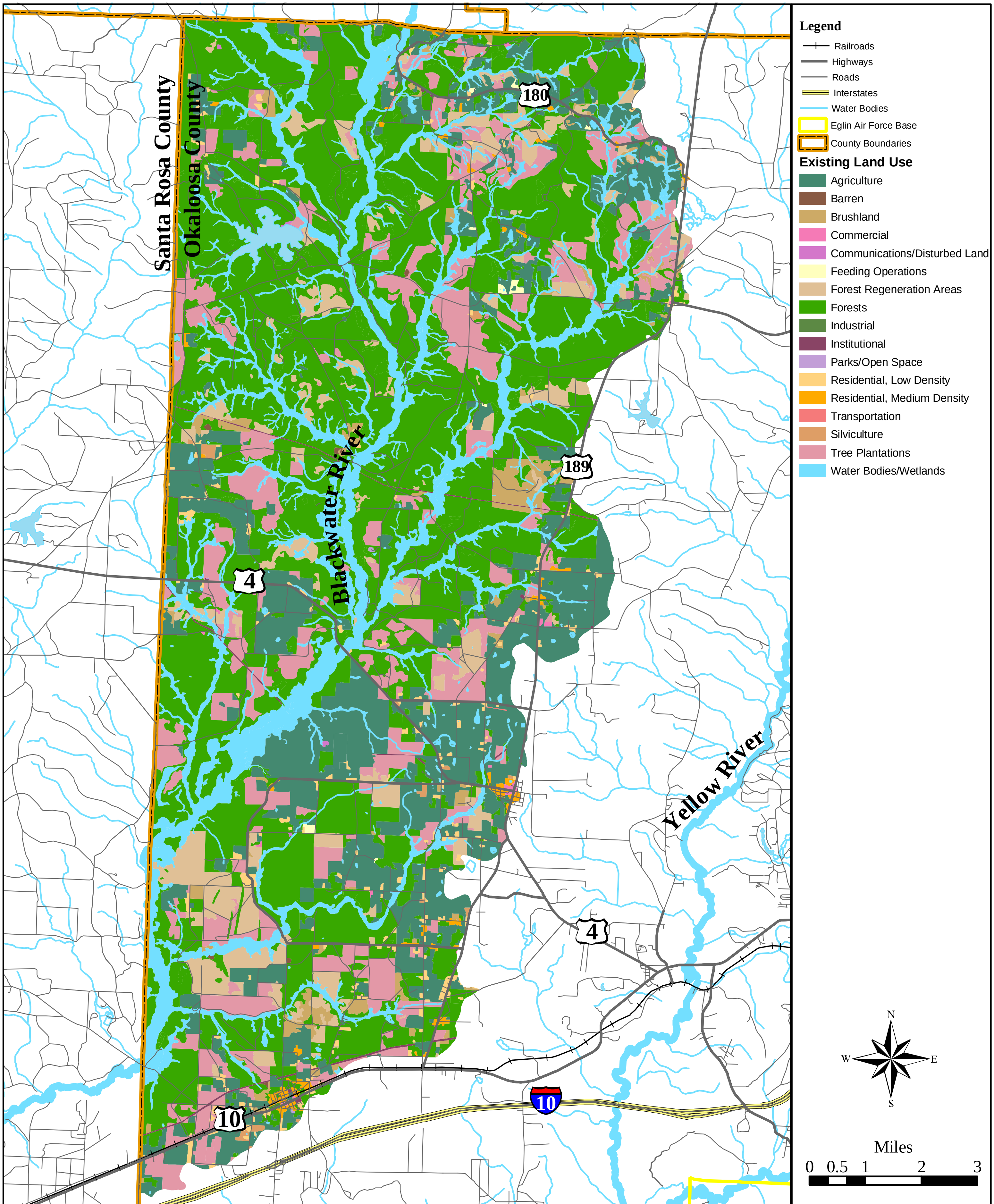
Blackwater River Basin NRCS Soil Classification (within Okaloosa County)

Figure 3-2

HDR



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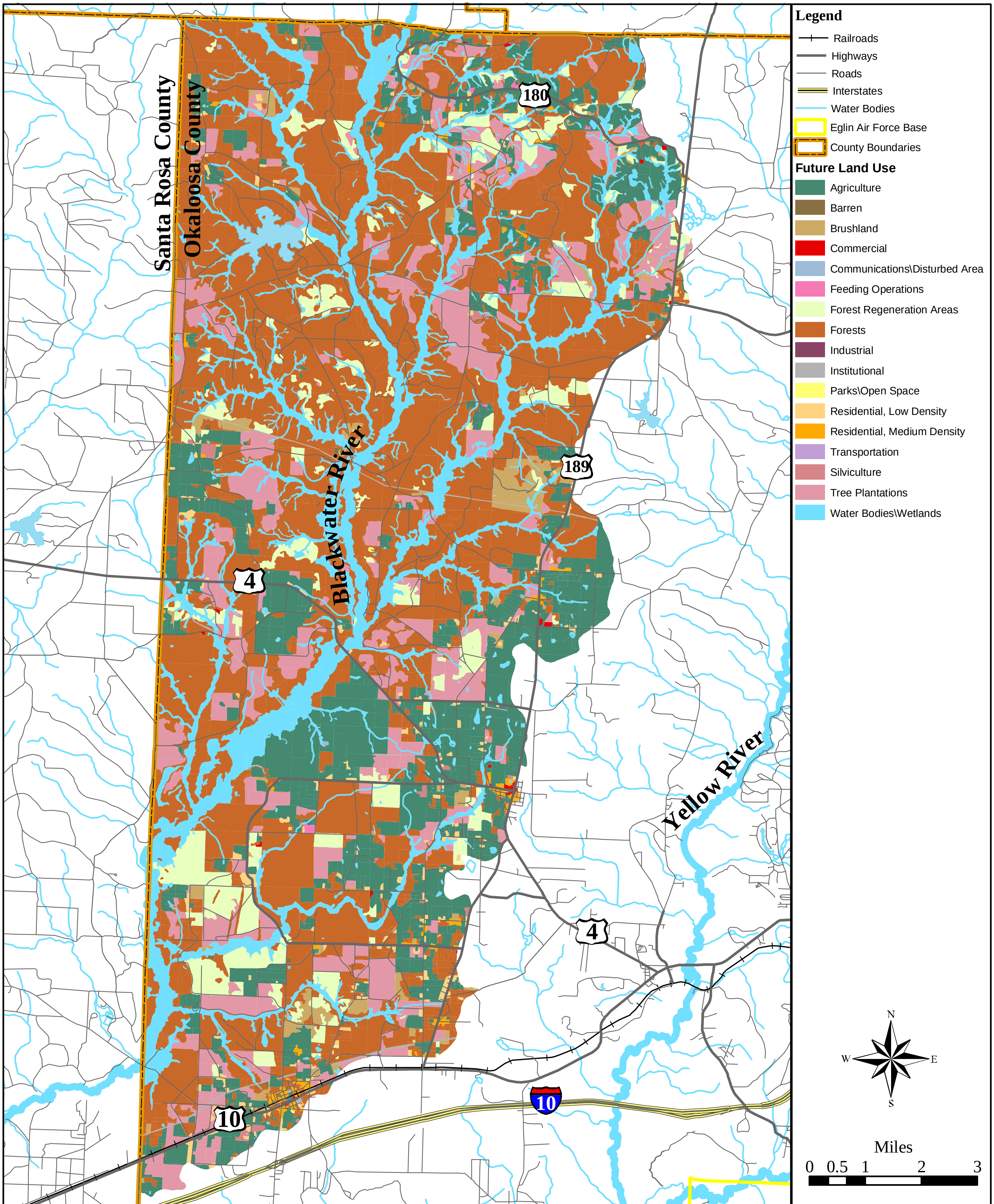
Blackwater River Basin Existing Land Use (within Okaloosa County)

Figure 3-3



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Blackwater River Basin Future Land Use (within Okaloosa County)

Figure 3-4

Table 3.1
Blackwater River Basin Soil Type Summary
(Okaloosa County)

Soil Series	General Hydrologic Characteristics	Texture	% Area
Bonifay	(0 to 8% slopes) Gently sloping well-drained soil on broad, nearly level to sloping ridges and side slopes. Moderate permeability with slow runoff.	Sand	6.3
Dothan	(0 to 8% slopes) Gently sloping well-drained soil on nearly level to sloping uplands. Moderate permeability with slow runoff.	Loamy Sand	14.9
Fuquay	(0 to 8% slopes) Gently sloping well-drained soil on broad, nearly level to sloping ridges and side slopes in the uplands. Slow permeability with slow runoff.	Loamy Fine Sand	7.7
Kinston	(0 to 5% slopes) Gently sloping poorly drained soil on nearly level floodplains along creeks, streams, and rivers on the Coastal Plain. Moderate permeability with slow runoff.	Silt Loam	8.8
Orangeburg	(0 to 12% slopes) Strongly sloping well-drained soil on nearly level to strongly sloping uplands. Moderate permeability with slow runoff.	Sandy Loam	8.2
Troup	(0 to 25%) Gently sloping well-drained soil on nearly level to steep uplands. Moderate permeability with slow runoff.	Sand	34.4
	Various soils, 10 soil types ranging from 0.01% to 4.1% area.		19.7
	Total Percent Area		100.0

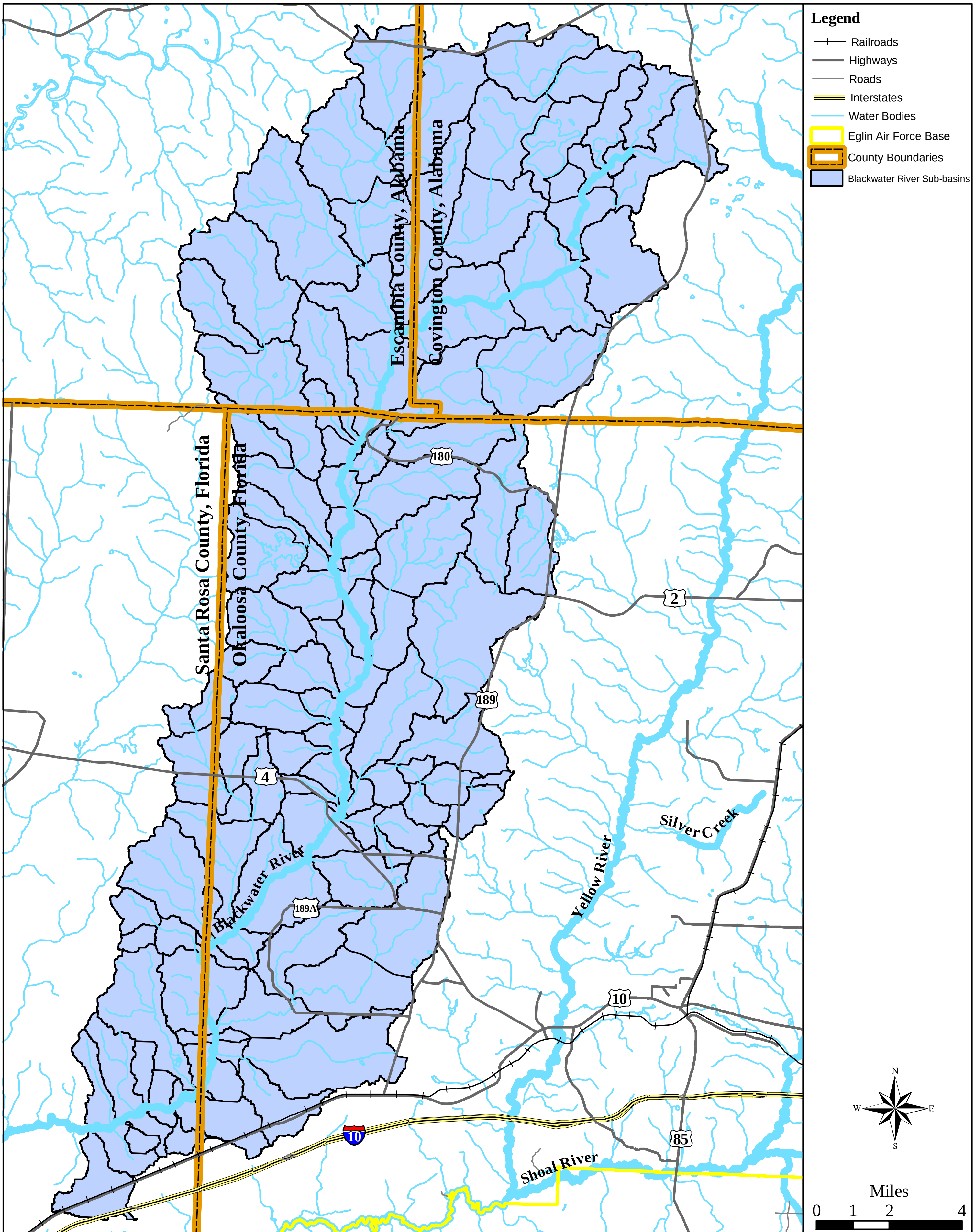
Source: Soil Survey of Okaloosa County, Florida; NRCS June 1995.

Table 3.2 Blackwater River Basin Existing and Future Land Use Summary (Okaloosa County)		
Land Use Group	**Existing**	**Future**
Agriculture	25	19
Barren	<1	<1
Brushland	1	2
Commercial	<1	<1
Communications/Disturbed Land	<1	<1
Feeding Operations	<1	<1
Forests	48	48
Forest Regeneration	5	6
Industrial	<1	<1
Institutional	<1	<1
Parks/Open Space	<1	<1
Residential, Low Density	<1	1
Residential, Medium Density	<1	<1
Transportation	<1	<1
Tree Plantations	10	12
Water Bodies/Wetlands	9	12
Total	**100**	**100**

3.2 FLOOD HYDROLOGY

The HEC-HMS model was used to compute peak runoff rates for the 2-, 10-, 25-, 50-, 100-, and 500-year storm events. Detailed input and output data appears in the Hydrologic and Hydraulic Appendices. **Figure 3-5** depicts the sub-basin delineation used during modeling.

The HEC-HMS model was calibrated to the known flood even of March 8, 1998. Initially, the peak flows and hydrographs produced by HEC-HMS did not match those measured at USGS Gage 02370000 for this storm event. In addition, a base flow of four cfs per square mile was observed at the gage. To more closely match the model results to the measured flows the transform method was changed from the SCS Unit Hydrograph method to the Clark's Method, and a baseflow of four cfs per square mile was added.



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Blackwater River HEC-HMS Sub-basin Delineation

Figure 3-5

After successful storm reconstitution, the hydrologic model was then calibrated to the USGS stream gage information. Originally SCS Type III Design Storms were selected for the meteorologic models of the 2-, 10-, 25-, 50-, 100-, and 500-year return period storm events. However, it was observed that these storms produced a hydrograph that rose too quickly. To allow more control over the hydrograph shape and timing, frequency storm events were substituted for the SCS Design Storms. The frequency storm events applied a maximum storm duration of four days, a peak center of 75 percent, and a storm area of 250 square miles (the approximate elliptical area upstream of the USGS gage was chosen).

The peak discharge results from HEC-HMS for the various return period storm events were compared with the HEC-FFA output that is based on the USGS Gage. The HEC-HMS peak flows were too high for the 2- and 10-year return period storm events and too low for the 500-year return period storm event. The volume of flood runoff for each of the simulated frequency storms was then checked with the respective volume-duration frequencies of the gage data. It was discovered that the 2- and 10-year predicted volumes were also higher than the HEC-FFA volume results.

To correct the inconsistency relating to the 2- and 10- year storm events, a second basin model was created for these events, which assumed an AMC I. This model resulted in 2- and 10-year peak discharges and volumes that more closely matched the respective HEC-FFA results.

Similarly, to correct the inconsistency relating to the 500-year event, and additional basin model was created for the 500-year return period storm event that had curve numbers ten percent higher than the original basin model. This third model allowed a closer approximation of predicted volume as compared to the HEC-FFA, and slightly closer approximation of predicted peak discharge as compared to the HEC-FFA for the 500-year event.

The original 25-, 50-, and 100-year return period storm model results were reasonably close to the HEC-FFA results, and were not altered.

Table 3.3 contains a summary of existing peak runoff rates, for selected storm events at key locations in the Blackwater River Basin applying existing conditions. Future development conditions were not considered, because the changes in curve number, due to small changes in land use, were slight and would not produce a significant difference in peak flows. A summary of the peak runoff rates for all sub-basin hydrographs can be found in the Hydrologic and Hydraulic Appendices.

Table 3.3
Blackwater River Drainage Basin
Peak Runoff Summary for Existing Drainage System Conditions

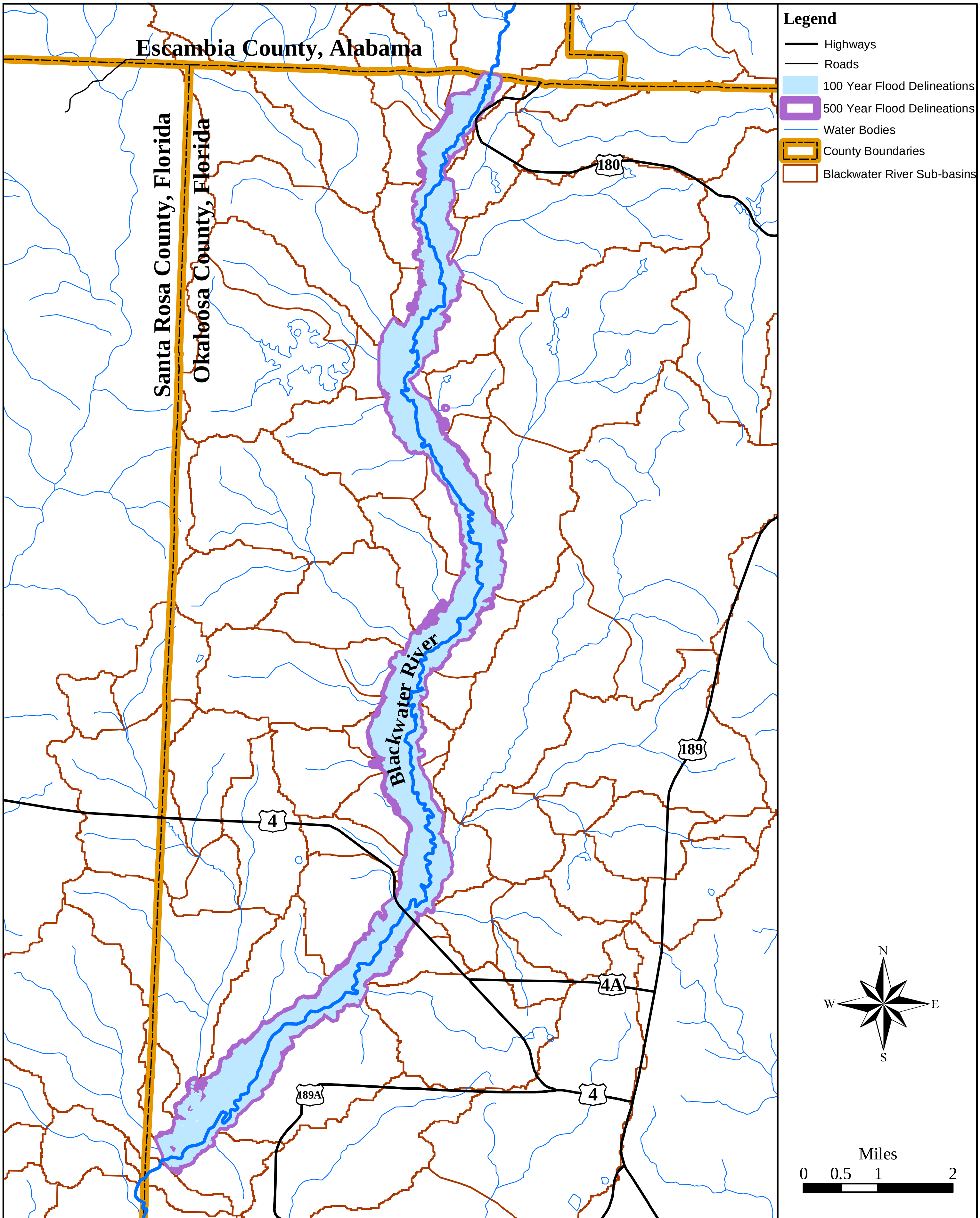
Structure Id. No.[1]	HEC-HMS Node	Location	Drainage Area (sq.mi.)	Peak Runoff Rate (cfs)[2, 3]					
				2-Year	10-Year	25-Year	50-Year	100-Year	500-Year
31	J28	Kennedy Bridge	140.2	2791	8512	19957	22361	26605	36563
35	J35	John Riley Barnhill Bridge	160.9	3215	9771	22862	25604	30425	41832
43	J51	Highway 4 Bridge	201.6	4168	12332	28242	31580	37413	51682

1. See Figure 3-1 for location of structure identification number.
2. Peak runoff rates based on existing land use condition.
3. Peak discharges reported are outflows from the specified nodes.

3.3 STREAM HYDRAULICS

HEC-RAS was utilized to determine the stream hydraulics of the channel and the bridges of the Blackwater River. In the modeling and mapping of the stream hydraulics, it was observed that the digital elevation model provided by the County had, in some locations, insufficient overbanks to allow for accurate mapping. These locations were primarily at confluences of tributaries and the Blackwater River mainstem, and at the John Riley Barnhill Bridge crossing. Where the digital elevation model was insufficient, the mapping was truncated at the limits of the TIN. Any further mapping of this area would require a digital elevation model with extended overbanks, especially for high flow events. **Figure 3-6** shows the flood delineations for the 100- and 500- year return period storm events and **Figure 3-7** illustrates the flood profiles for the 2-, 10-, 25-, 50-, 100-, and 500-year storm events.

Three bridge crossings exist over the main stem of the Blackwater River all of which were analyzed within the model. A summary of the hydraulic capacity for each of the crossings studied is presented in **Table 3.4** for the 2-, 10-, 25-, 50-, 100-, and 500-year storm events.



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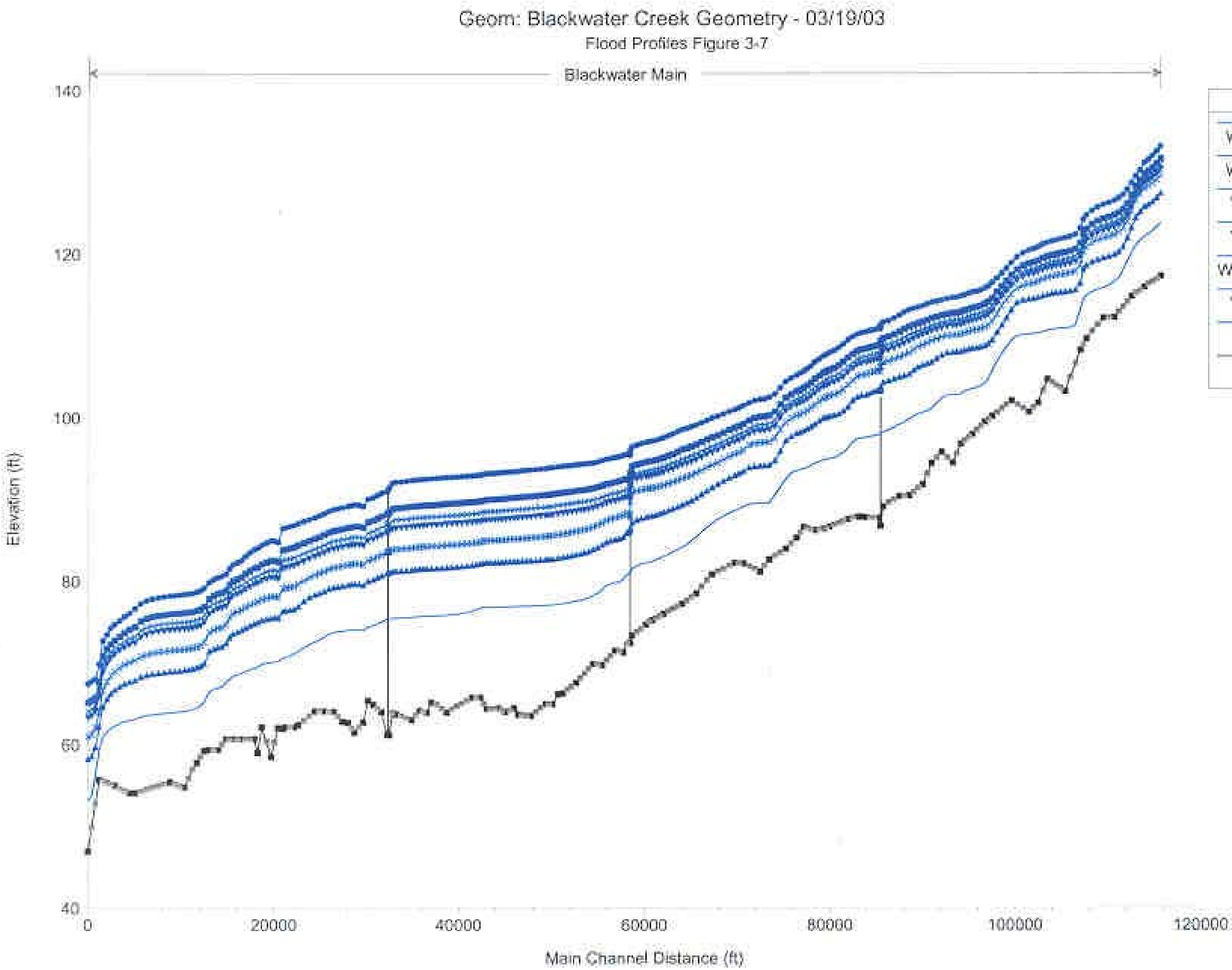
Blackwater River 100-Year and 500-Year Flood Delineations

Figure 3-6



Geom: Blackwater Creek Geometry - 03/19/03
Flood Profiles Figure 3-7
Blackwater Main
Legend
WS 500year
WS 100year
WS 50year
WS 25year
WS Calibrated
WS 10year
WS 2year
Ground
Elevation (ft)
140
120
100
80
60
40
0
20000
40000
60000
80000
100000
120000
Main Channel Distance (ft)

Table 3.4
Blackwater River Drainage Basin
Existing Hydraulic Capacity of Stream Crossings Summary

Structure Id. No.[1]	Location	Minimum Overtopping Elevation[2]	Depth of Overtopping (ft)[3]					
			2-Year	10-Year	25-Year	50-Year	100-Year	500-Year
31	Kennedy Bridge	104.75	-	-	3.0	3.6	4.6	6.6
35	John Riley Barnhill Bridge	87.62	-	-	3.4	4.1	5.4	8.1
43	Highway 4 Bridge	91.15	-	-	-	-	-	-

1. See Figure 3-1 for location of structure identification number.
2. Minimum overtopping depth elevation based on topographic survey, unless otherwise noted.
3. Depth of overtopping based on HEC-RAS analysis.

The standards/criteria for passing the design flood event without roadway overtopping were used to evaluate each crossing. A summary of the hydraulic capacity and return period for each of the crossings studied is presented in **Table 3.5**.

Table 3.5
Blackwater River Drainage Basin
Existing Hydraulic Capacity and Return Period of Stream Crossings Summary

Structure Id. No.[1]	Location	Existing Structure Type	Roadway Classification	Hydraulic Capacity Return Period	
				Required	Actual
31	Kennedy Bridge	Bridge	Local	10-Year	10-Year
35	John Riley Barnhill Bridge	Bridge	Local	10-Year	10-Year
43	Highway 4 Bridge	Bridge	Arterial	50-Year	500-Year

1. See Figure 3-1 for location of structure identification number.

3.4 LEVEL OF SERVICE ANALYSES

Table 3.6 summarizes the results of the culvert LOS analyses within the Blackwater River Basin. Within Table 3.6 the size of the existing culvert, storm frequency required by the LDC, overtopping frequency, and a recommendation are shown. All of the culverts were analyzed with the 25-year return period storm event. Based on the analysis it is recommended that structure 44 be desilted.

Table 3.6
Blackwater River Basin Culvert LOS Analysis Summary

Structure Id. No.[1]	Location	Existing Culvert	Storm Frequency	Overtopping Frequency	Recommendation
29	Hwy. 180, Panther Creek	2 – 8'x7'	25-year	>500	NA
34	Red Barrow Road, Panther Creek	3 – 10'x7'	25-year	45	NA
37	Hwy. 189, Pyron Spring Branch	2 – 9'x9'	25-year	>500	NA
42	SR 4, Penny Creek	2 – 5'x10'	25-year	>500	NA
44	Hwy 4, 1.2 mi west of Beaver Creek Hwy.	2 – 10'x10'	25-year	40[2]	Desilt Culvert
45	Hwy 4, 0.6 mi west of Beaver Creek Hwy.	2 – 5'x7'	25-year	>500	NA

1. See Figure 3-1 for location of structure identification number.
2. Without desilting.

3.5 DETAILED STUDY AREAS

No detailed study areas were identified by Okaloosa County within the Blackwater River Basin.

4.0 YELLOW RIVER BASIN

4.1 GENERAL DRAINAGE BASIN DESCRIPTION

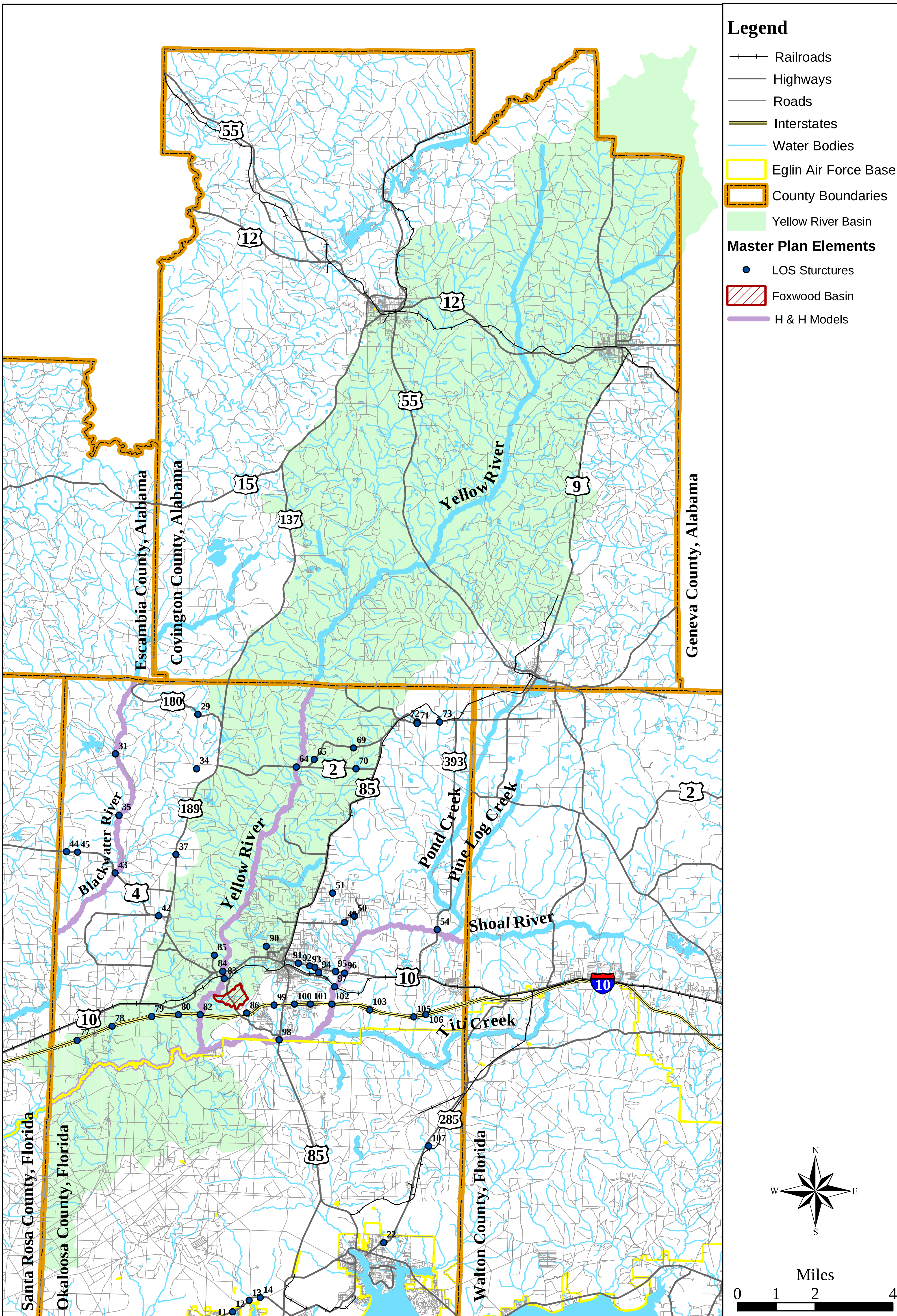
The Yellow River Basin travels through the middle of the County as shown in **Figure 4-1**. The drainage basin measures approximately 762 square miles, of which 263 square miles are within the County boundary. Portions of the basin extend into Santa Rosa County and the majority of the basin extends into Alabama. The basin in Okaloosa County is roughly bounded by SR 189 to the west, SR 85 to the east north of US 90, and follows no landmarks to the east south of US 90 or to the south.

Elevations in the basin range from approximately 20 feet near the confluence with the Shoal River to 500 feet in the northernmost area of the basin in Alabama. Within Okaloosa County the elevations range from approximately 20 feet to 320 feet.

Table 4.1 shows the relative representation and general hydrologic characteristics for the soils found in the Yellow River Basin within Okaloosa County. Within Okaloosa County, the Yellow River Basin contains 21 different soil types, of which the Lakeland series accounts for over 70

Table 4.1
Yellow River Basin Soil Type Summary
(Okaloosa County)

Soil Series	General Hydrologic Characteristics	Texture	% Area
Bonifay	(0 to 8% slopes) Gently sloping well-drained soil on broad, nearly level to sloping ridges and side slopes. Moderate permeability with slow runoff.	Sand	3.2
Dothan	(0 to 8% slopes) Gently sloping well-drained soil on nearly level to sloping uplands. Moderate permeability with slow runoff.	Loamy Sand	2.4
Kinston	(0 to 5% slopes) Gently sloping poorly drained soil on nearly level floodplains along creeks, streams, and rivers on the Coastal Plain. Moderate permeability with slow runoff.	Silt Loam	4.5
Lakeland	(0 to 30% slopes) Gently sloping excessively drained soil on nearly level to steep uplands. Rapidly permeable with slow runoff.	Sand	70.9
	Various soils, 16 soil types ranging from 0.01% to 1.4% area.		9.0
	Total Percent Area		100.0
Source: Soil Survey of Okaloosa County, Florida; NRCS June 1995.			



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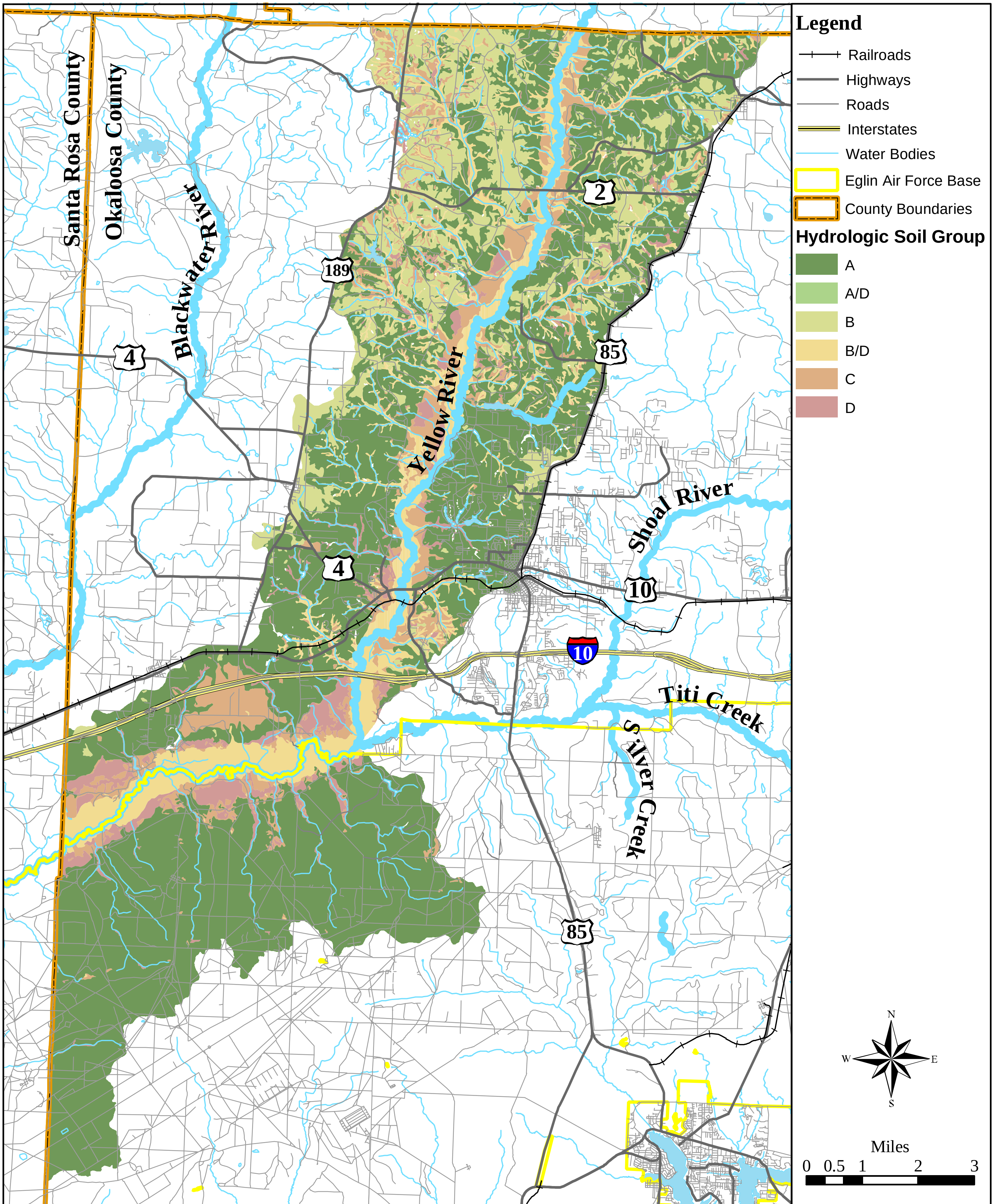
Yellow River Basin

Figure 4-1

percent of the total basin area. The Lakeland series is located throughout the basin comprising almost the entire area south of the Yellow River west of the confluence with the Shoal River. The soils located along the river channel primarily consist of the Kinston series, which accounts for approximately four percent of the basin area. For modeling purposes, the different soil types were grouped by NRCS hydrologic soil type as Type A, B, C, and D. Seventy percent of the basin consists of Type B soils as depicted in **Figure 4-2**.

Land use classifications in the Yellow River Basin range from forests to residential, with the majority of the basin classified as forest land. The breakdown of existing land use (grouped by classification used for the H&H models) within the Yellow River Basin is shown in **Figure 4-3**.

Figure 4-4 shows the future land use (grouped by categories used for H&H models) within the Yellow River Basin based on the County's future land use map, the municipalities' future land use maps, and existing land use data where necessary as discussed in Section 2.12.2. As shown the future land use is quite similar to the existing land use and there is no increase in impermeable land use. **Table 4.2** shows a comparison of the percentage of each land use classifications for both existing and future conditions.



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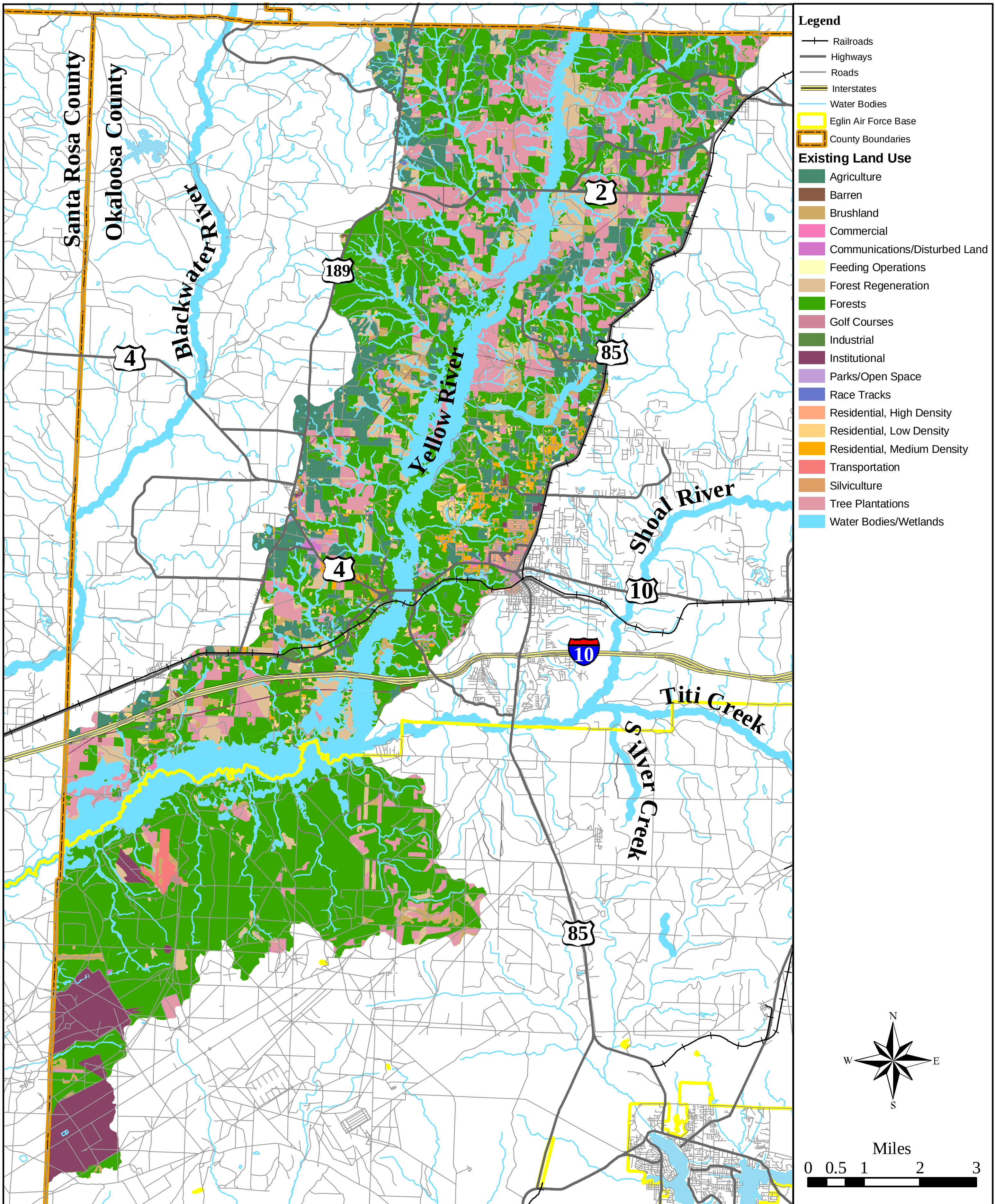
Yellow River Basin NRCS Soil Classification (within Okaloosa County)

Figure 4-2



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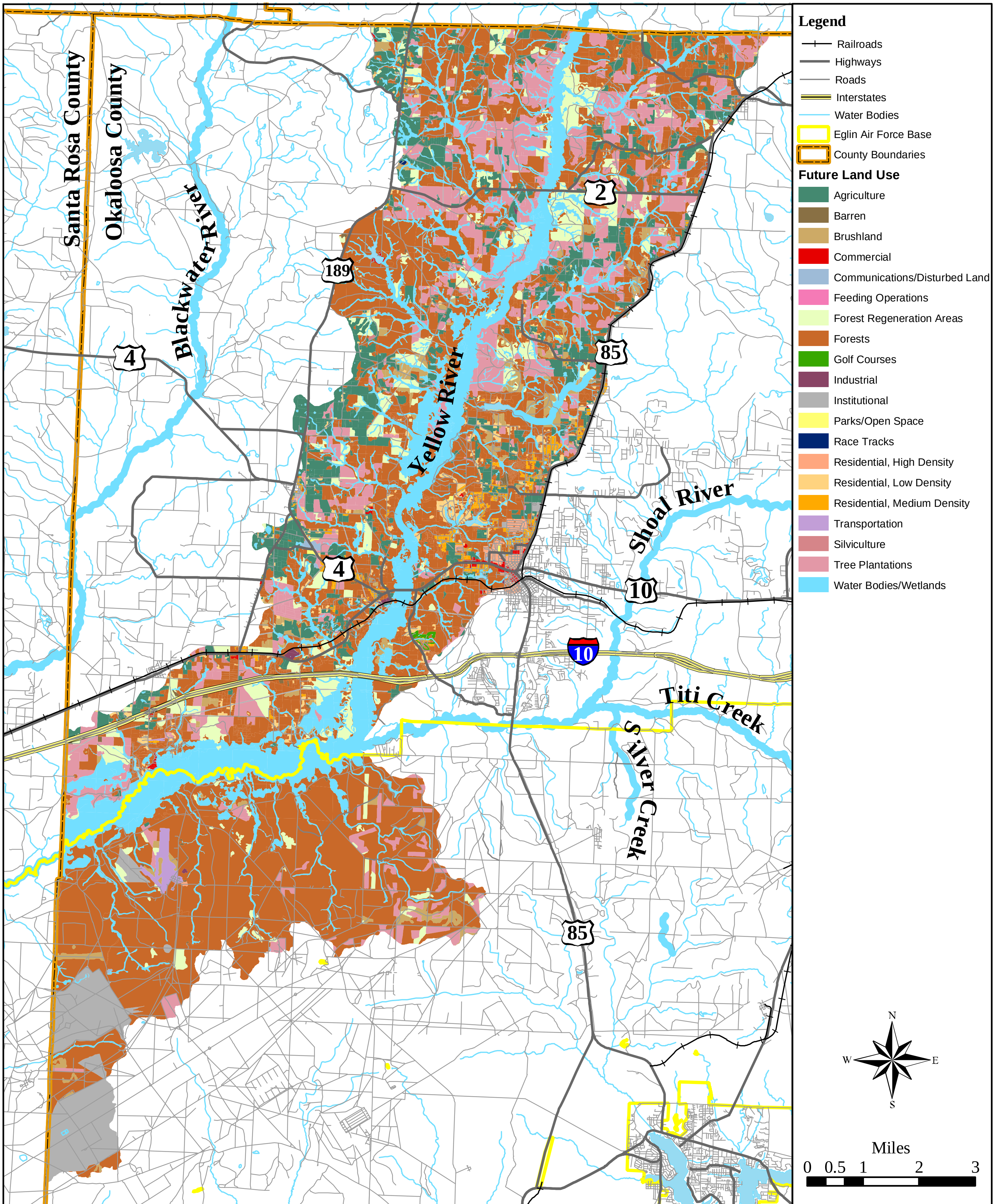
Yellow River Basin Existing Land Use (within Okaloosa County)

Figure 4-3



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Yellow River Basin Future Land Use (within Okaloosa County)

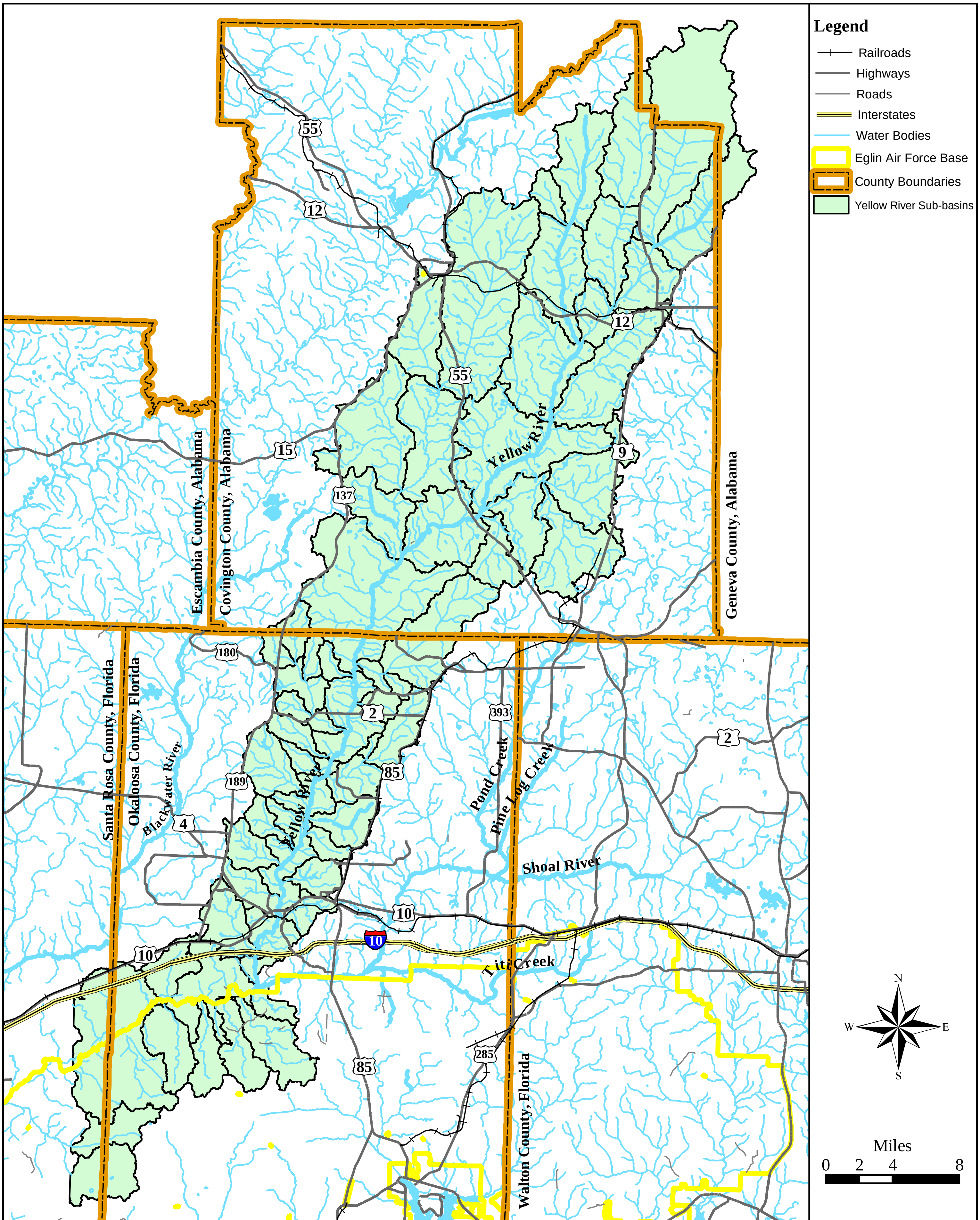
Figure 4-4

Table 4.2 Yellow River Basin Existing and Future Land Use Summary (Okaloosa County)		
Land Use Group	**Existing**	**Future**
Agriculture	6	3
Barren	<1	<1
Brushland	<1	<1
Commercial	<1	<1
Communications/Disturbed Land	<1	<1
Feeding Operations	<1	<1
Forests	79	81
Forest Regeneration	2	2
Golf Courses	<1	<1
Industrial	<1	<1
Institutional	2	2
Parks/Open Space	<1	<1
Race Tracks	<1	<1
Residential, High Density	<1	<1
Residential, Low Density	<1	<1
Residential, Medium Density	<1	<1
Transportation	<1	<1
Tree Plantations	5	4
Water Bodies/Wetlands	5	5
Total	**100**	**100**

4.2 FLOOD HYDROLOGY

The HEC-HMS model was used to compute peak runoff rates for the 2-, 10-, 25-, 50-, 100-, and 500-year storm events. Detailed input and output data appears in the Hydrologic and Hydraulic Appendices. **Figure 4-5** depicts the sub-basin delineation used during modeling.

The HEC-HMS model was calibrated to the known flood event of March 8, 1998. Initially, the peak flows and hydrographs produced by HEC-HMS did not match those measured at USGS Gage 02368000 for this storm event. In addition, a base flow of three cfs per square mile was observed at the gage. To more closely match the model results to the measured flows the transform method was changed from the SCS Unit Hydrograph method to the Clark's Method,



Master Stormwater Management Plan

Yellow River Basin HEC-HMS Sub-basin Delineation

Figure 4-5

and a baseflow of three cfs per square mile was added. In addition, antecedent moisture conditions were revised to reflect conditions believed to be present within the basin as reflected by the hydrograph recorded at USGS Gage 02368000.

Storm reconstitution efforts resulted in a computed peak discharge of approximately 47,800 cfs, versus a recorded peak discharge of 55,600 cfs. Time to peak from beginning of rainfall matched very closely, with less than a one-hour difference between computed and recorded values. Likewise, total runoff volumes were reconstituted well with a difference of approximately six percent between computed and observed values. The reconstitution efforts included raising SCS curve numbers to reflect antecedent moisture content higher than an AMC III value. The lower computed peak discharge relative to the observed value is likely a result of high antecedent moisture content and spatial variation of rainfall amounts within the basin.

After storm reconstitution, the hydrologic model was then calibrated to peak discharges for various design storm events as computed by a log-Pearson Type III analysis of USGS Gage 02368000, which includes 57 years of record. Due to the relatively long period of record for the gage, the log-Pearson Type III statistical analysis is considered to provide the best analysis available for predicting flow values for extreme events on the Yellow River. Precipitation depths for the design storm events were taken from TP40 and Hydro-35. The frequency storm events applied a maximum storm duration of four days, a peak center of 75 percent, and a storm area of 400 square miles.

The peak discharge results from HEC-HMS for the various return period storm events were compared with the log-Pearson Type III analysis, which was completed using HEC-FFA. The initial HEC-HMS simulations were completed assuming an antecedent moisture condition of AMC II. The HEC-HMS peak flows were too high for the 2-, 10-, and 25-year return period storm events and too low for the 100- and 500-year return period storm events. The volume of flood runoff for each of the simulated frequency storms was then checked with the respective volume-duration frequencies of the gage data. The 2-, 10-, and 25-year predicted volumes were also higher than the HEC-FFA volume results.

HEC-HMS simulated discharges for design storms were calibrated to the HEC-FFA computed discharges by varying the antecedent moisture condition for the various design storm events. Lower return period storms were adjusted by decreasing the antecedent moisture content, while higher return period storms were adjusted by increasing the antecedent moisture conditions. This process facilitated the development of HEC-HMS models for the various design storms that reasonably reproduce the computed design storm discharges predicted by the HEC-FFA gage analysis. HEC-HMS simulated peak discharges for the 2- through 100-year storms reproduced the computed design discharges to within ten percent. The 25-, 50-, and 100-year events were reproduced to within five percent. The 500-year event was simulated using curve numbers equivalent to 1.15 times an AMC III.

The original 50-year return period storm model results were reasonably close to the HEC-FFA results and were not altered.

The HEC-HMS models for the Yellow River were then finalized by adding the Shoal River HEC-HMS output hydrograph at the confluence location downstream of the Louisiana and Nashville Railroad bridge crossing (HEC-HMS node J80).

Table 4.3 provides a summary of existing peak runoff rates for selected storm events at key locations in the Yellow River Basin. Future development conditions were not considered, because the changes in curve number, due to small changes in land use, were slight and would not produce a significant difference in peak flows. A summary of the peak runoff rates for all sub-basin hydrographs can be found in the Hydrologic and Hydraulic Appendices.

Table 4.3
Yellow River Drainage Basin
Peak Runoff Summary for Existing Drainage System Conditions

Structure Id. No.[1]	HEC-HMS Node	Location	Drainage Area (sq.mi.)	Peak Runoff Rate (cfs) [2, 3]					
				2-Year	10-Year	25-Year	50-Year	100-Year	500-Year
64	J48	S.H. 2 Bridge	522.6	10250	25320	42050	61240	76390	105650
82	J76	SH10/US90 Bridge	643.4	9910	24330	40900	59840	75450	105930
83	J76	L&N RR Bridge	643.4	9910	24330	40900	59840	75450	105930
84	J79	IH 10 Bridge	666.1	9650	24310	40880	59910	75670	106540
N/A	J80	Shoal Confluence	1,163	14420	33480	53760	77300	96700	160180

1. See Figure 4-1 for location of structure identification number.
2. Peak runoff rates based on existing land use condition.
3. Peak discharges reported are outflows from the specified nodes.

4.3 STREAM HYDRAULICS

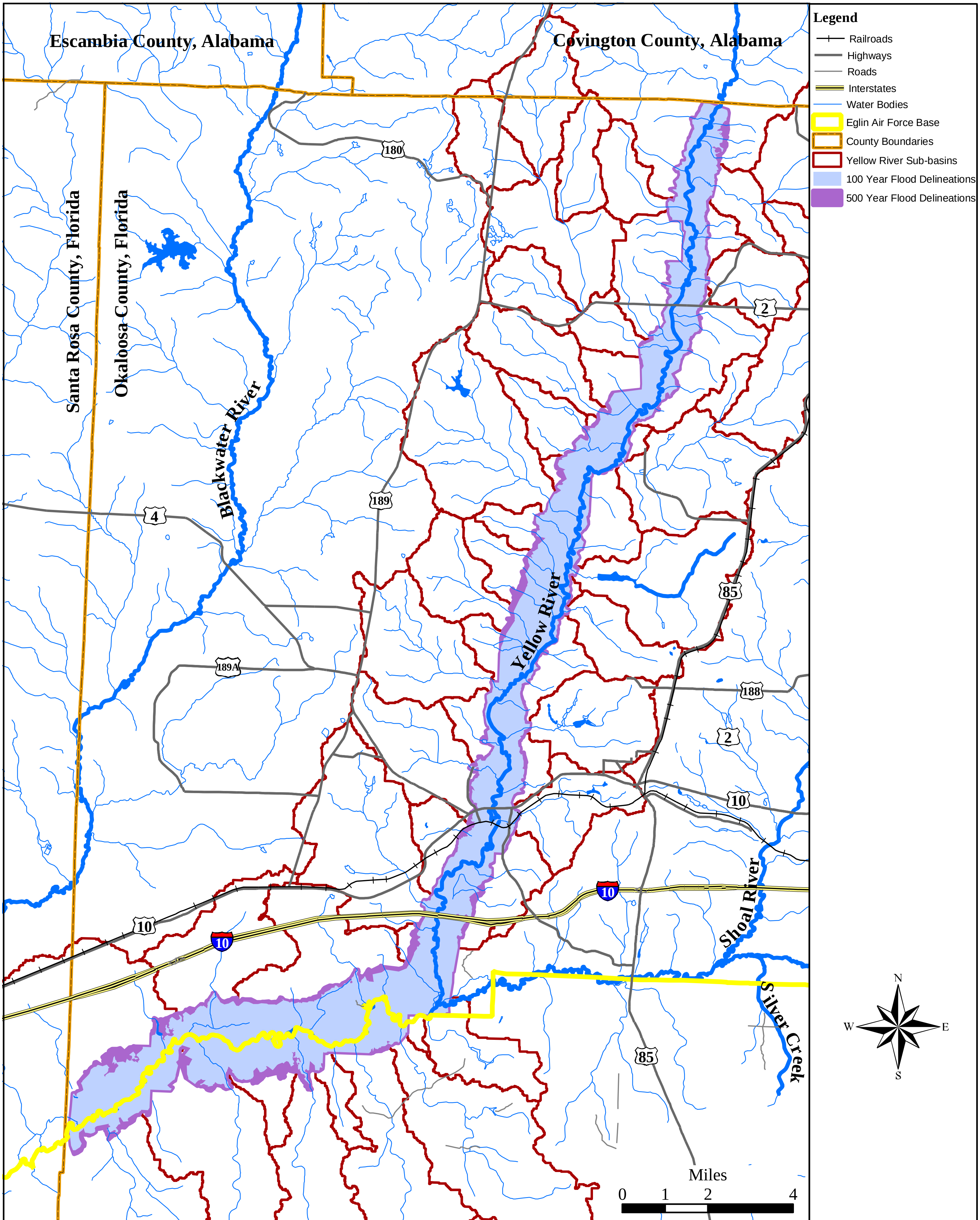
HEC-RAS was utilized to determine the stream hydraulics of the channel and the bridges of the Yellow River. In the modeling and mapping of the stream hydraulics, it was observed that the digital elevation model provided by the County had, in some locations, insufficient overbanks to allow for accurate mapping. Where the digital elevation model was insufficient, cross-sections were extended based on general observations of overbank slope as determined from USGS quadrangle maps. The HEC-RAS model was calibrated with stage-discharge data for USGS Gage number 02368000. The initial HEC-RAS model compared well with the gage data, requiring only a minor modification to overbank Manning's 'n' values, which were set to 0.18, the upper range of previous FEMA estimates. **Figure 4-6** shows the flood delineations for the 100- and 500- year return period storm events and **Figure 4-7** illustrates the flood profiles for the 2-, 10-, 25-, 50-, 100-, and 500-year storm events.

Four bridge crossings exist over the main stem of the Yellow River, all of which were analyzed within the model. A summary of the hydraulic capacity for each of the crossings studied is presented in **Table 4.4** for the 2-, 10-, 25-, 50-, 100-, and 500-year storm events.

Table 4.4
Yellow River Drainage Basin
Existing Hydraulic Capacity of Stream Crossings Summary

Structure Id. No.[1]	Location	Minimum Overtopping Elevation[2]	Depth of Overtopping (ft)[3]					
			2-Year	10-Year	25-Year	50-Year	100-Year	500-Year
64	S.H. 2 Bridge	117.5	-	-	-	-	-	-
84	SH10/US90 Bridge	64.7	-	-	1.9	4.2	5.7	8.2
83	L&N RR Bridge	61.5	-	2.6	3.9	5.5	6.7	9.0
82	IH 10 Bridge	68.5	-	-	-	-	-	-

1. See Figure 4-1 for location of structure identification number.
2. Minimum overtopping depth elevation based on topographic survey, unless otherwise noted.
3. Depth of overtopping based on HEC-RAS analysis.



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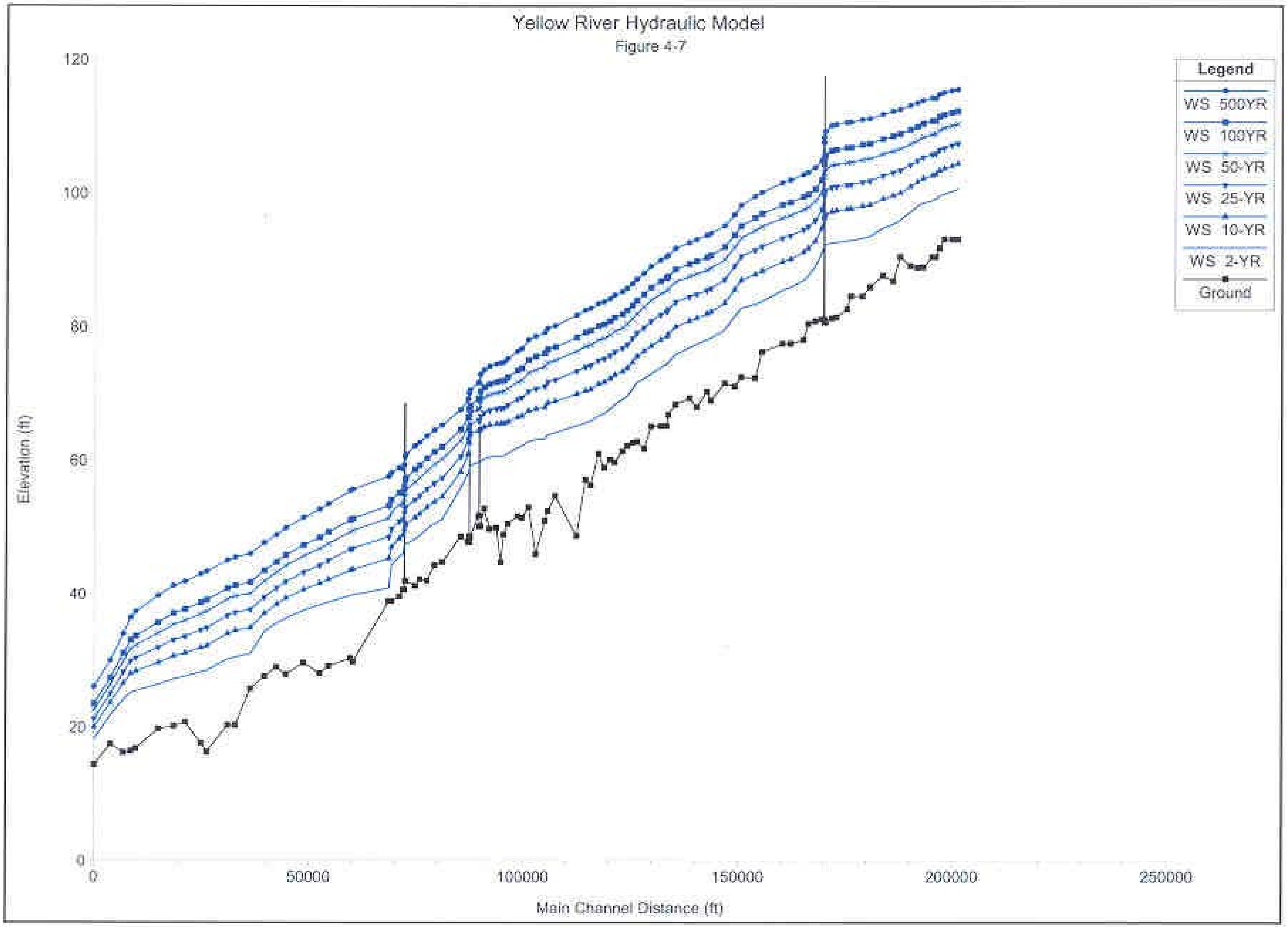
Yellow River 100-Year and 500-Year Flood Delineations

Figure 4-6



Yellow River Hydraulic Model
Figure 4-7
Legend
WS 500YR
WS 100YR
WS 50-YR
WS 25-YR
WS 10-YR
WS 2-YR
Ground
Elevation (ft)
0
20
40
60
80
100
120
Main Channel Distance (ft)
0
50000
100000
150000
200000
250000

The standards/criteria for passing the design flood event without roadway overtopping were used to evaluate each crossing. A summary of the hydraulic capacity and return period for each of the crossings studied is presented in **Table 4.5**.

Table 4.5
Yellow River Drainage Basin
Existing Hydraulic Capacity and Return Period of Stream Crossings Summary

Structure Id. No.[1]	Location	Existing Structure Type	Roadway Classification	Hydraulic Capacity Return Period	
				Required	Actual
64	S.H. 2 Bridge	Bridge	Arterial	50-Year	500-Year
82	SH10 / US90 Bridge	Bridge	Arterial	50-Year	10-Year
84	IH 10 Bridge	Bridge	Interstate	100-Year	500-Year

1. See Figure 4-1 for location of structure identification number.

4.4 LEVEL OF SERVICE ANALYSES

Table 4.6 summarizes the results of the culvert LOS analyses within the Yellow River Basin. Within Table 4.6 the size of the existing culvert, storm frequency required by the LDC, overtopping frequency, and a recommendation are shown. All of the culverts were analyzed with either the 25- or 50-year return period storm event. Based on the analysis it is recommended that structure 65 be desilted and 90 be replaced to increase the capacity to that required by the LDC. Structures 69, 70, 77-80, and 85 appear to have sufficient capacity.

Table 4.6 Yellow River Basin Culvert LOS Analysis Summary					
Structure Id. No.[1]	**Location**	**Existing Culvert**	**Storm Frequency**	**Overtopping Frequency**	**Recommendation**
65	Hwy 602, Mill Creek	3–8'x7'[2]	25-year	71[2] 6[2]	Desilt Culvert
69	Hwy 602, Big Creek Tributary	2–24"	25-year	25	NA
70	Hwy 2, Murder Creek	2–10'x6'	25-year	48	NA
77	I-10, Canoe Creek	10'x3'	50-year	>500	NA
78	I-10, Trewick Creek	2–12'x5' 10'x5'	50-year	>500	NA
79	I-10, Wilkerson Creek	2–9'x5'	50-year	>500[3]	NA
80	I-10, Yellow River Tributary	11'x4'	50-year	>500	NA
85	Old River Road, 0.2 mi. north of Garret Mill Road	3–10'x6'	25-year	40[3]	NA
90	Pandora Drive	102"	25-year	12	2-7'X6'

1. See Figure 4-1 for location of structure identification number.
2. After desilting.
3. Without desilting.

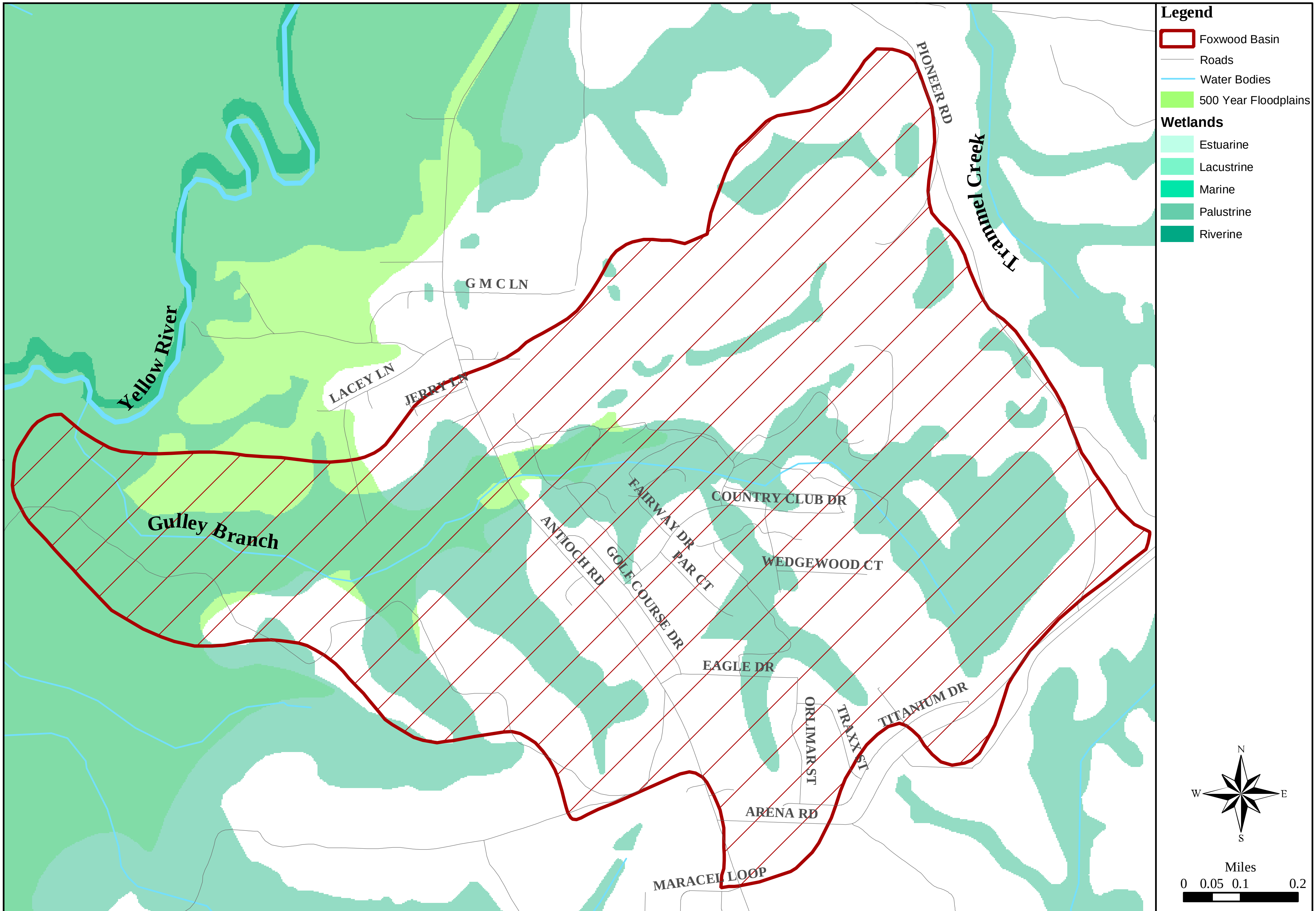
4.5 DETAILED STUDY AREAS

4.5.1 Foxwood Subdivision

4.5.1.1 Existing Conditions

Foxwood is a residential subdivision located off of Antioch Road north of I-10. The neighborhood forms part of a 215-acre drainage basin that discharges to Gulley Branch. The area features significant topographic relief, including rolling hills with slopes as steep as 12 percent. The roadway typical cross-section is concave, with a subsurface storm drain system and inlets located along the roadway centerline. A location map showing Foxwood and its associated drainage basin appears as **Figure 4-8** and **Figure 4-9** contains a picture of existing conditions.

Foxwood is located in an area with substantial coverage of Fuquay loamy fine sand and Bonifay sand. In both of these soils, water becomes perched above the subsoil during periods of heavy rainfall. This characteristic has manifested itself in Foxwood by saturating the roadway base and



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Foxwood Subdivision

Figure 4-8

the lawns of residents. In addition, the combination of perched water and steep slopes has created localized areas where the phreatic surface intersects existing ground, resulting in overland flow through yards to the storm drain system.

Figure 4-9
Foxwood Subdivsion Photograph

4.5.1.2 Alternative Solutions

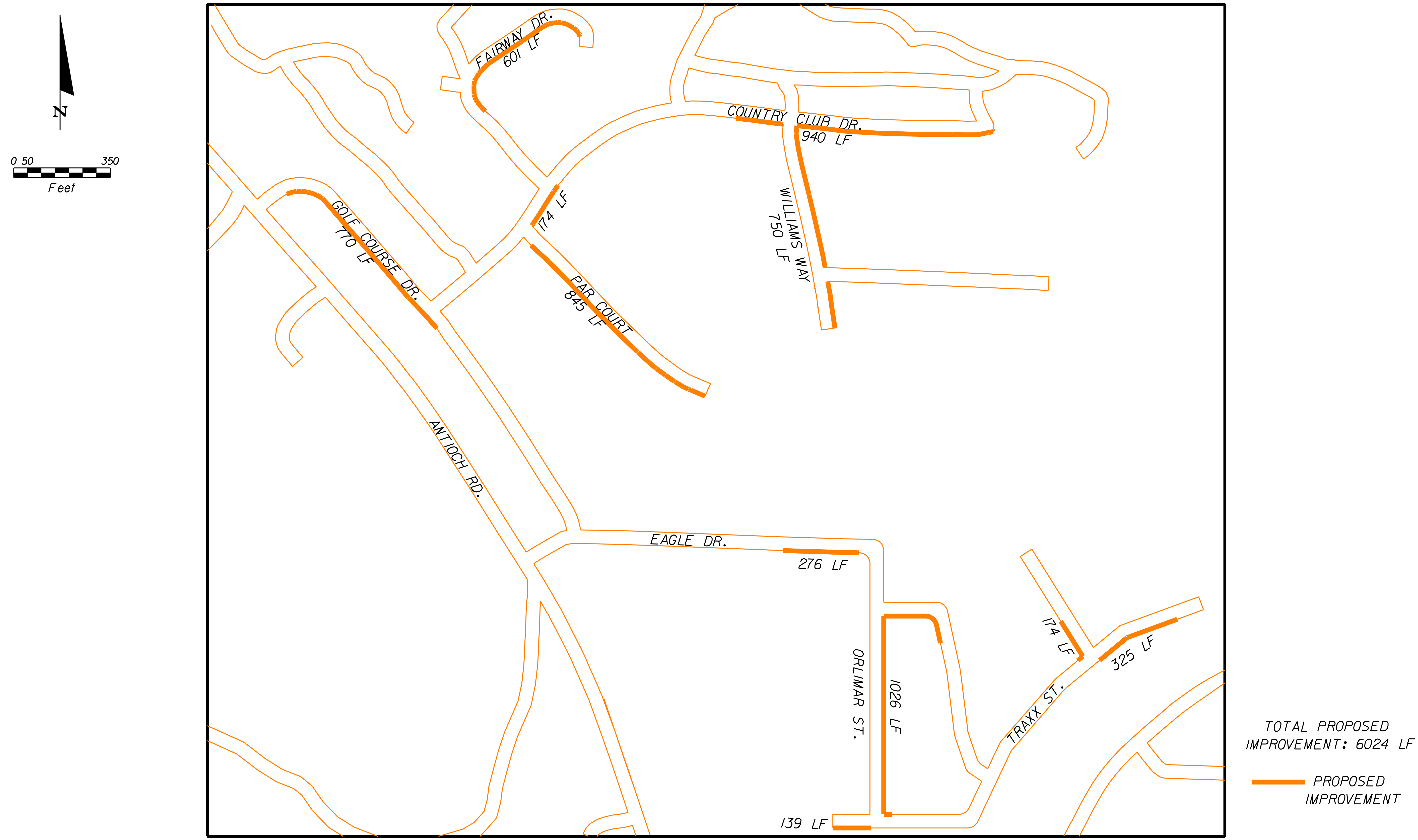
The extent of saturation created by perched water is highly dependent upon antecedent and current rainfall. Parts of the neighborhood that exhibit no problems during dry weather may produce springheads under wetter conditions. In other words, the location of all potential springs cannot be determined with certainty absent an extensive geotechnical investigation.

Instead of recommending a geotechnical investigation, it was assumed for purposes of this analysis that saturated conditions are most likely to appear in valleys, where the roadway interrupts steep slopes, and where previous spring activity has been observed during field visits. Applying this methodology, corrective measures are recommended at the locations shown in **Figure 4-10.**

Three alternatives were considered as appropriate corrective measures, including roadside ditches, underdrain and edge drain, which is a prefabricated strip drain installed in a trench adjacent to the roadway. The relative advantages and disadvantage of each system follows in **Table 4.7**

Table 4.7
Foxwood Subdivision Alternative Solutions

Alternative	Advantages	Disadvantages
Ditches	Lowest Cost Proven To Drain Base Easiest To Maintain	Requires Right-Of-Way Aesthetically Undesirable
Underdrains	Proven To Drain Base Aesthetically Desireable Can Be Built In Existing Right-of-Way	High Maintenance Requirements Can Clog With Sediment Most Expensive
Edge Drain	Moderate Cost Can Be Built In Existing Right-of-Way	New Technology Maintenance Costs Unknown



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FOXWOOD SUBDIVISION PROPOSED IMPROVEMENT LOCATIONS

Figure 4-10

Of the presented alternatives, underdrains are recommended due to their proven effectiveness, and because underdrains will not require additional right-of-way.

Note that the solutions presented are intended to improve the serviceability of the County roadway system within the neighborhood by draining the base and lowering the frequency of maintenance required. While surrounding property owners may experience improvements due to a general drawdown of groundwater, eliminating saturation in surrounding yards would require the extension of underdrain laterals into the yards.

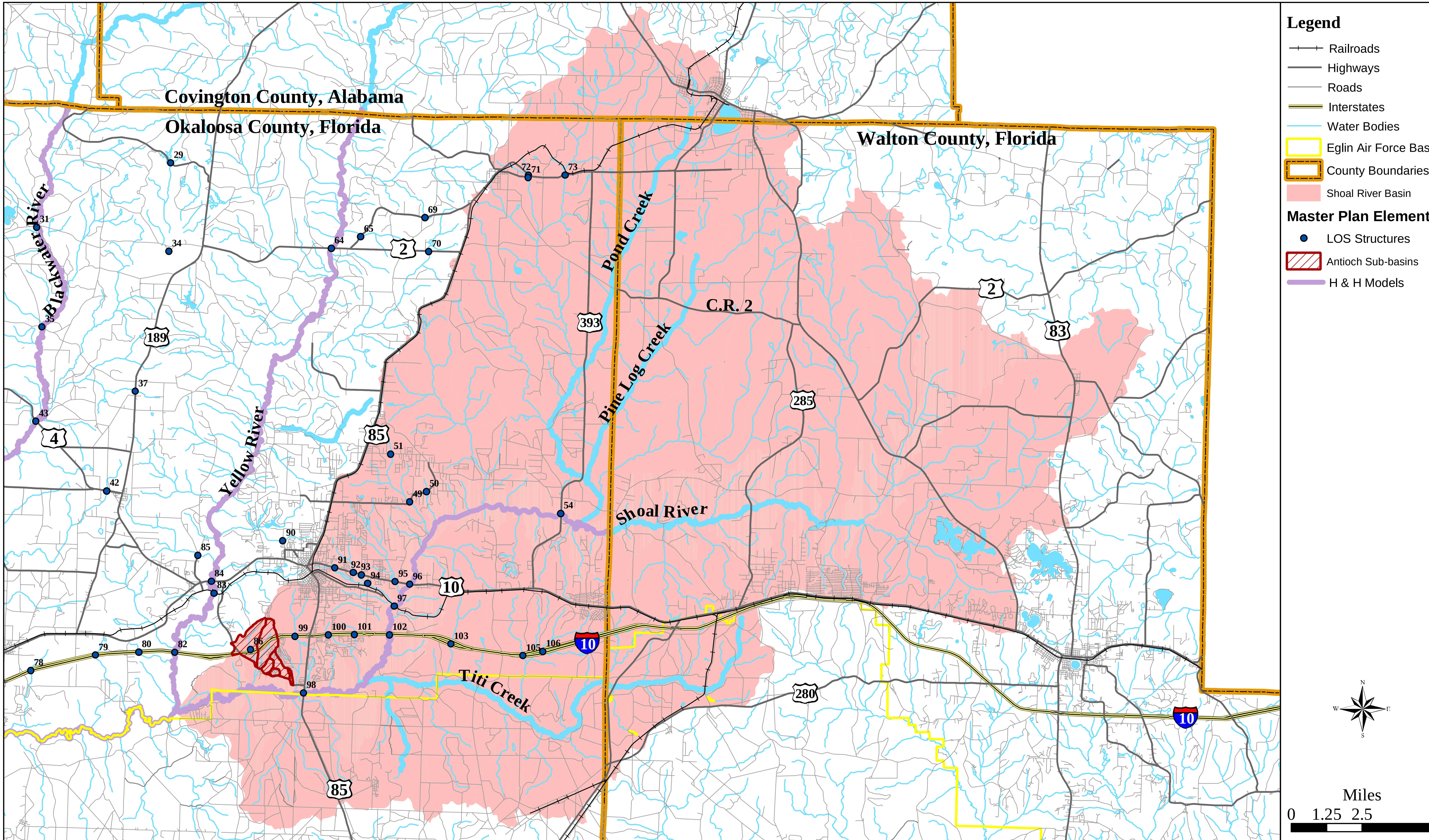
5.0 SHOAL RIVER BASIN

5.1 GENERAL DRAINAGE BASIN DESCRIPTION

The Shoal River Basin is located in the northeast portion of the County and is shown in **Figure 5-1**. The drainage basin measures approximately 498 square miles, of which 230 square miles are within the County boundary. Portions of the basin extend into Walton County and Alabama. The basin is roughly bounded by SR 85 to the west north of US 90 and follows no landmarks to the west south of US 90 or to the south.

Elevations in the basin range from approximately 20 feet near the confluence with the Yellow River to 345 feet in the northernmost area of the basin in Alabama. Within Okaloosa County the elevations range from approximately 20 feet to 325 feet.

Table 5.1 shows the relative representation and general hydrologic characteristics for the soils found in the Shoal River Basin within Okaloosa County. Within Okaloosa County, the Shoal River Basin contains 23 different soil types, of which the Lakeland series accounts for close to 75 percent of the total basin area. The majority of the Lakeland series is located in the southern two-thirds of the basin. The soils located along the river channel primarily consist of the Kinston series, which accounts for approximately four percent of the basin area. For modeling purposes, the different soil types were grouped by NRCS hydrologic soil type as Type A, B, C, and D. Eighty percent of the basin consists of Type A soils as depicted in **Figure 5-2**.



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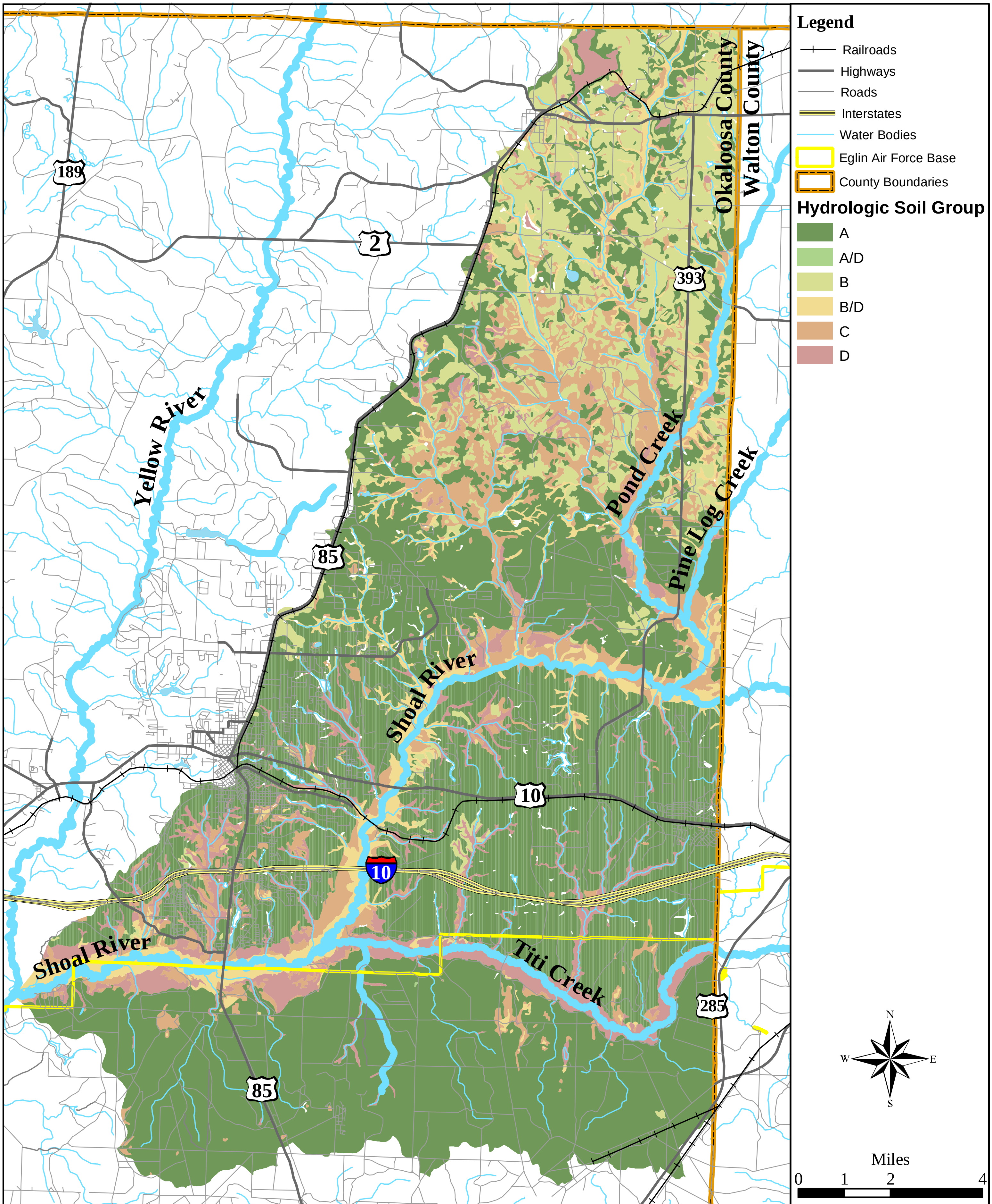
Shoal River Basin

Figure 5-1

HDR



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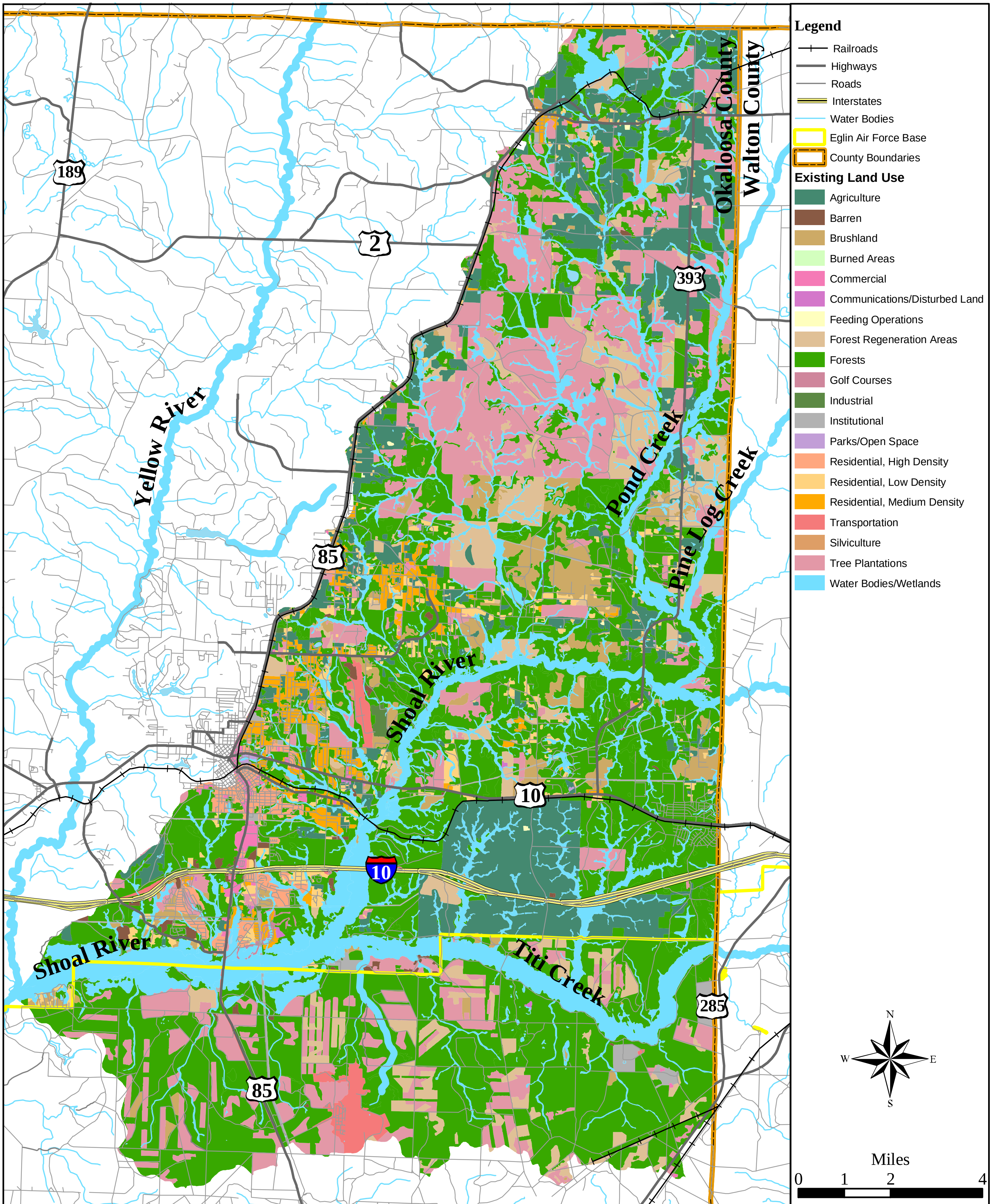
Shoal River NRCS Soil Classification (within Okaloosa County)

Figure 5-2

Table 5.1 Shoal River Basin Soil Type Summary (Okaloosa County)			
Soil Series	**General Hydrologic Characteristics**	**Texture**	**% Area**
Bonifay	(0 to 8% slopes) Gently sloping well-drained soil on broad, nearly level to sloping ridges and side slopes. Moderate permeability with slow runoff.	Sand	3.8
Dorovan	(<1% slopes) Level poorly drained soil on broad, nearly level flood plains along the major streams and in large hardwood swamps. Moderate permeability with slow runoff.	Muck	2.5
Dothan	(0 to 8% slopes) Gently sloping well-drained soil on nearly level to sloping uplands. Moderate permeability with slow runoff.	Loamy Sand	3.0
Kinston	(0 to 5% slopes) Gently sloping poorly drained soil on nearly level floodplains along creeks, streams, and rivers on the Coastal Plain. Moderate permeability with slow runoff.	Silt Loam	3.7
Lakeland	(0 to 30% slopes) Gently sloping excessively drained soil on nearly level to steep uplands. Rapidly permeable with slow runoff.	Sand	74.3
Troup	(0 to 25%) Gently sloping well-drained soil on nearly level to steep uplands. Moderate permeability with slow runoff.	Sand	5.3
	Various soils, 16 soil types ranging from 0.01% to 1.7% area.		7.4
	Total Percent Area		100.0
Source: Soil Survey of Okaloosa County, Florida; NRCS June 1995.			

Land use classifications in the Shoal River Basin range from forests to residential, with the majority of the basin classified as forest land. The breakdown of existing land use (grouped by classifications used for the H&H models) within the Shoal River Basin is shown in **Figure 5-3**.

Figure 5-4 shows the future land use (grouped by categories used for H&H models) within the Shoal River Basin based on the County's future land use map, the municipalities' future land use maps, and existing land use data where necessary as discussed in Section 2.1.2.2. As shown the future land use is quite similar to the existing land use and there is no increase in impermeable land use. **Table 5.2** shows a comparison of the percentage of each land use classifications for both existing and future conditions.



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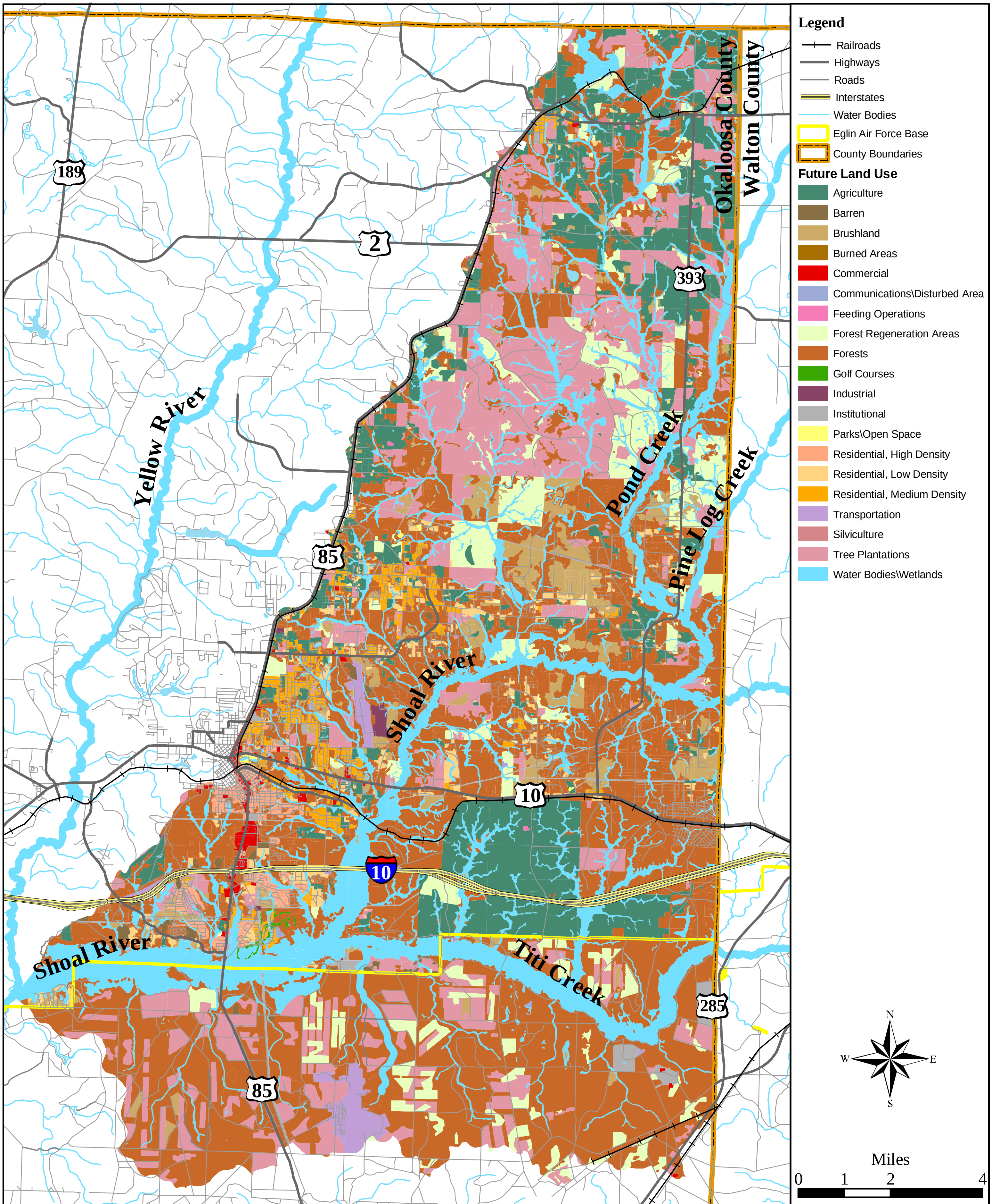
Shoal River Existing Land Use (within Okaloosa County)

Figure 5-3



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Shoal River Future Land Use (within Okaloosa County)

Figure 5-4

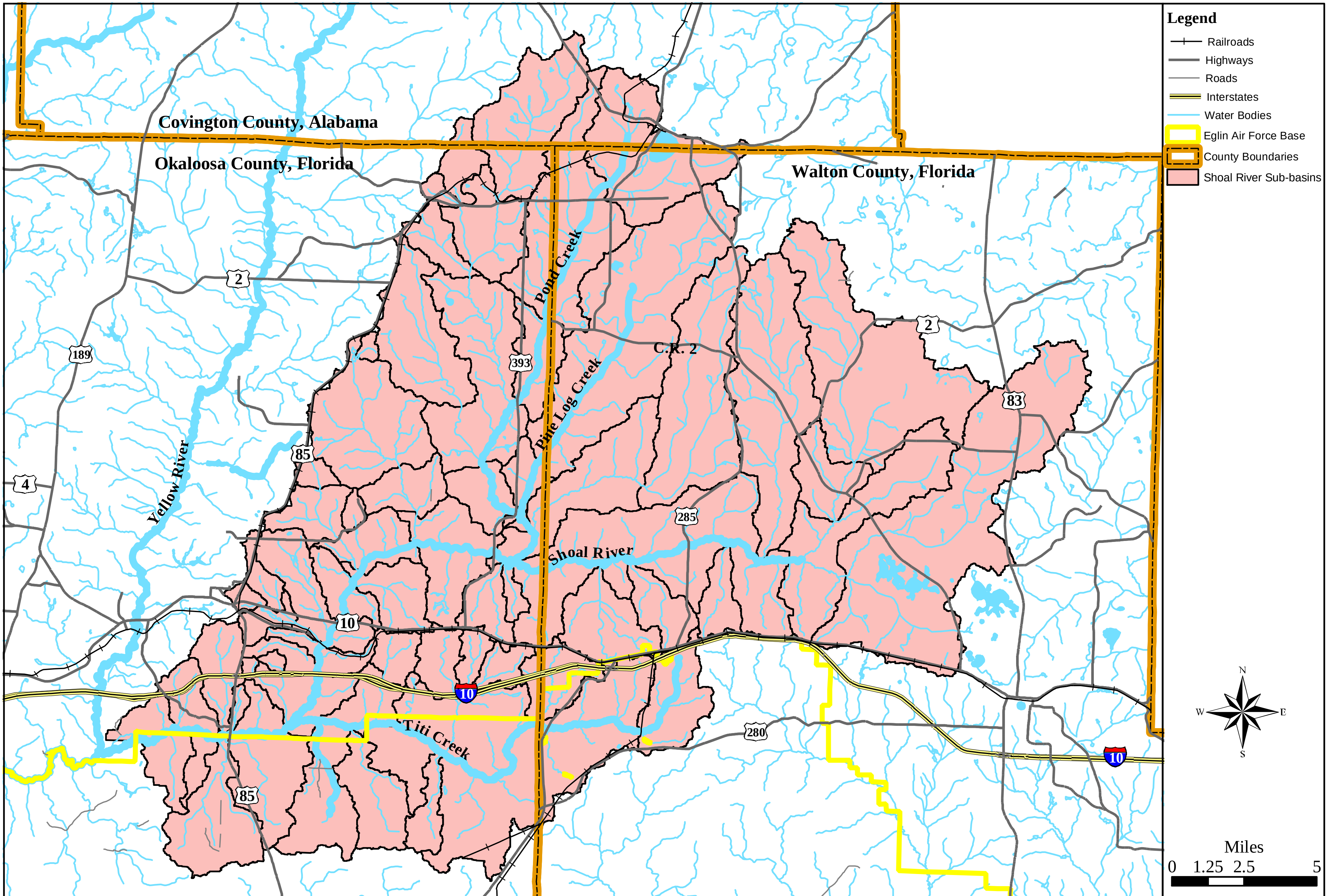
Table 5.2
Shoal River Basin
Existing and Future Land Use Summary
(Okaloosa County)

Land Use Group	Existing	Future
Agriculture	5	5
Barren	<1	<1
Brushland	1	1
Burned Areas	<1	<1
Commercial	<1	<1
Communications/Disturbed Land	<1	<1
Feeding Operations	<1	<1
Forests	80	80
Forest Regeneration	2	2
Golf Courses	<1	<1
Industrial	<1	<1
Institutional	<1	<1
Parks/Open Space	<1	<1
Residential, High Density	<1	<1
Residential, Low Density	<1	<1
Residential, Medium Density	<1	<1
Transportation	<1	<1
Tree Plantations	5	5
Water Bodies/Wetlands	4	4
Total	**100**	**100**

5.2 FLOOD HYDROLOGY

The HEC-HMS model was used to compute peak runoff rates for the 2-, 10-, 25-, 50-, 100-, and 500-year storm events. Detailed input and output data appears in the Hydrologic and Hydraulic Appendices. **Figure 5-5** depicts the sub-basin delineation used during modeling.

The HEC-HMS model was calibrated to the known flood event of March 8, 1998 with reference to measured values at USGS Gage 0269000. Initially, the hydrograph produced by HEC-HMS displayed a lower and earlier peak as compared to the peak measured at the gage. In addition, a base flow of 5.0 cfs per square mile was observed at the gage. To more closely match the



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Shoal River HEC-HMS Sub-basin Delineation

Figure 5-5

hydrograph shape, the transform method was changed from the SCS Unit Hydrograph method to the Clark's Method. To improve the timing, time of concentration values were increased by a factor of four, Manning's n was increased in the main channel to 0.055, and Manning's n was increased in the tributaries to 0.065. To improve agreement with the measured peak discharge, an antecedent moisture condition of AMC II plus 6 was used for determining curve numbers. Finally, a baseflow of 5.0 cfs per square mile was added.

Storm reconstitution efforts resulted in a computed peak discharge of approximately 18,390 cfs, versus a recorded peak discharge of 17,900 cfs. Time to peak from beginning of rainfall matched the recorded time to peak, with a 1.25-hour difference between computed and recorded values. Likewise, the total runoff volume at the gage was reconstituted with a difference of less than one percent between computed and observed values.

After storm reconstitution, the hydrologic model was then calibrated to peak discharges for various design storm events as computed by a log-Pearson Type III analysis of USGS Gage 02369000, which includes 61 years of record. Precipitation depths for the design storm events were taken from TP40 and Hydro-35. The frequency storm events applied a maximum storm duration of four days, a peak center of 75 percent, and a storm area of 400 square miles.

The peak discharge results from HEC-HMS for the various return period storm events were compared with the log-Pearson Type III analysis, which was completed using HEC-FFA. The HEC-HMS peak flows compared favorably to HEC-FFA results, with an average deviation from HEC-FFA of approximately 10%. In addition, all peak flows computed by HEC-HMS fell within HEC-FFA statistical confidence limits. The volume of flood runoff for each of the simulated frequency storms was then checked with the respective volume-duration frequencies of the gage data, with similar correlation

Table 5.3 provides a summary of existing peak runoff rates for selected storm events at key locations in the Shoal River Basin. Future development conditions were not considered, because the changes in curve number, due to small changes in land use, were slight and would not produce a significant difference in peak flows. A summary of the peak runoff rates for all sub-basin hydrographs can be found in the Hydrologic and Hydraulic Appendices.

Table 5.3
Shoal River Drainage Basin
Peak Runoff Summary for Existing Drainage System Conditions

Structure Id. No.[1]	HEC-HMS Node	Location	Drainage Area (sq.mi.)	Peak Runoff Rate (cfs)[2, 3]					
				2-Year	10-Year	25-Year	50-Year	100-Year	500-Year
54	J54	CR-393 Bridge	315.8	8010	18680	24130	28830	32610	47470
96	J24	US-90 Bridge	361.3	8310	19360	25100	30160	34190	50920
97	J25	CSX Railroad	372.7	8200	19060	24720	29770	33740	50630
98	J43	SR-85 Bridge	471.2	8970	21070	27620	33490	38070	58370
98 (overflow)	J43	SR-85 Overflow Br	471.2	8970	21070	27620	33490	38070	58370
102	J29	I-10 Bridges	375.1	8180	19010	24660	29710	33720	50650

1. See Figure 5-1 for location of structure identification number.
2. Peak runoff rates based on existing land use conditions.
3. Peak discharges reported are outflows from the specified nodes.

5.3 STREAM HYDRAULICS

HEC-RAS was utilized to determine the stream hydraulics of the channel and the bridges of the Shoal River. In the modeling and mapping of the stream hydraulics, it was observed that the digital elevation model provided by the County had, in some locations, insufficient overbanks to allow for accurate mapping. Where the digital elevation model was insufficient, cross-sections were extended based on general observations of overbank slope as determined from USGS quadrangle maps. The HEC-RAS model was calibrated with stage-discharge data for USGS Gage number 02369000 and NWFWMD Gage number 511. The initial HEC-RAS model compared well with the gage data, requiring only a minor modification to overbank Manning's 'n' values, which were set to 0.16, which is within the range of previous FEMA estimates.

Figure 5-6 shows the flood delineations for the 100- and 500- year return period storm events and **Figure 5-7** illustrates the flood profiles for the 2-, 10-, 25-, 50-, 100-, and 500-year storm events.

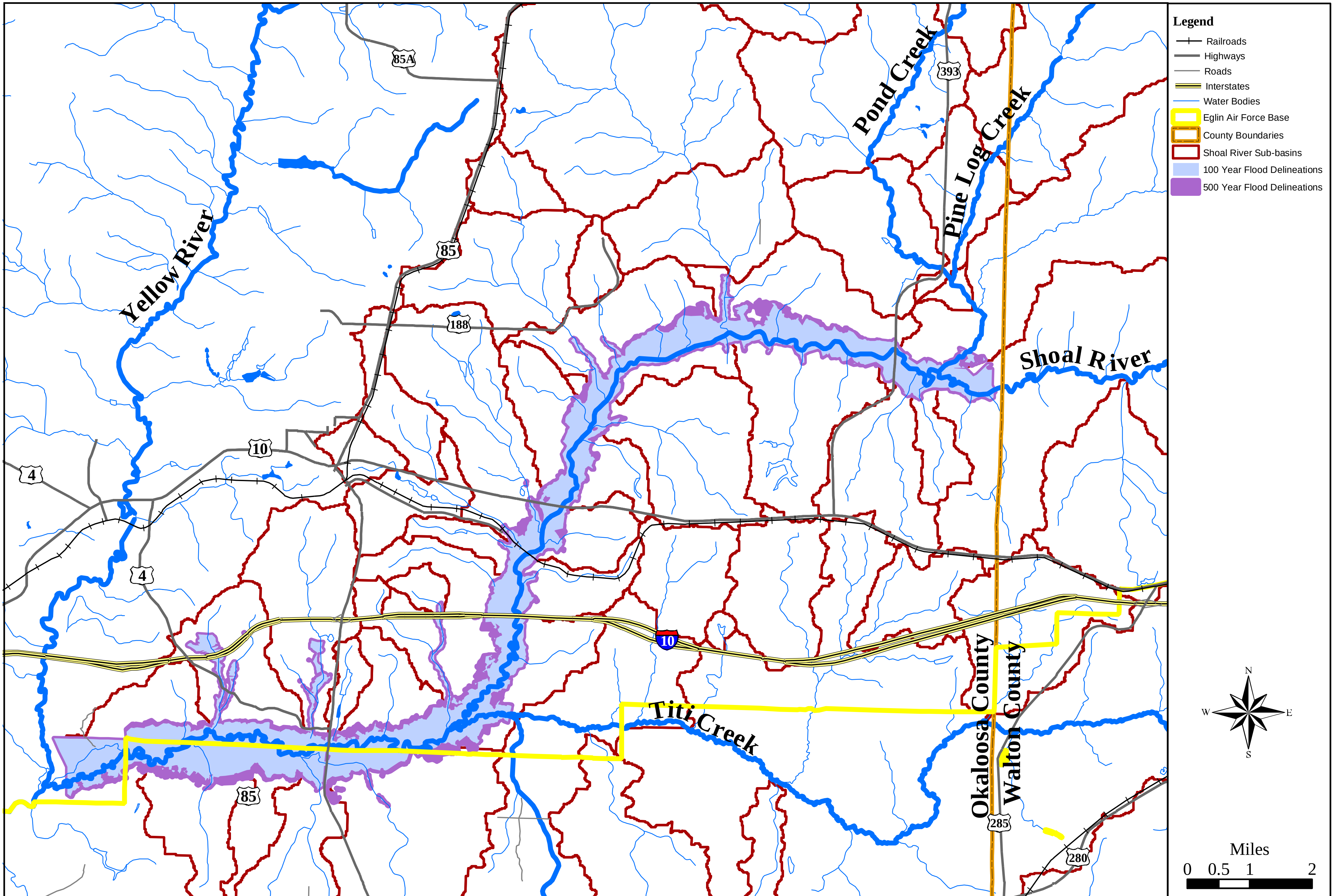
Six bridge crossings exist over the main stem of the Shoal River, all of which were analyzed within the model. A summary of the hydraulic capacity for each of the crossings studied is presented in **Table 5.4** for the 2-, 10-, 25-, 50-, 100-, and 500-year storm events.

Table 5.4
Shoal River Drainage Basin
Existing Hydraulic Capacity of Stream Crossings Summary

Structure Id. No.[1]	Location	Minimum Overtopping Elevation[2]	Depth of Overtopping (ft)[3]					
			2-Year	10-Year	25-Year	50-Year	100-Year	500-Year
54	CR-393 Bridge	103.2	-	3.1	4.5	5.5	6.1	8.2
96	US-90 Bridge	89.7	-	-	-	-	-	1.3
97	CSX Railroad	85.5	-	-	-	-	-	1.3
98	SR-85 Bridge	68.5	-	-	-	-	-	-
98 (overflow)	SR-85 Overflow Bridge	67.1	-	-	-	-	-	-
102	I-10 Bridges	83.5	-	-	-	-	-	-

1. See Figure 5-1 for location of structure identification number.
2. Minimum overtopping depth elevation based on topographic survey, unless otherwise noted.
3. Depth of overtopping based on HEC-RAS analysis.

The standards/criteria for passing the design flood event without roadway overtopping were used to evaluate each crossing. A summary of the hydraulic capacity and return period for each of the crossings studied is presented in **Table 5.5**.



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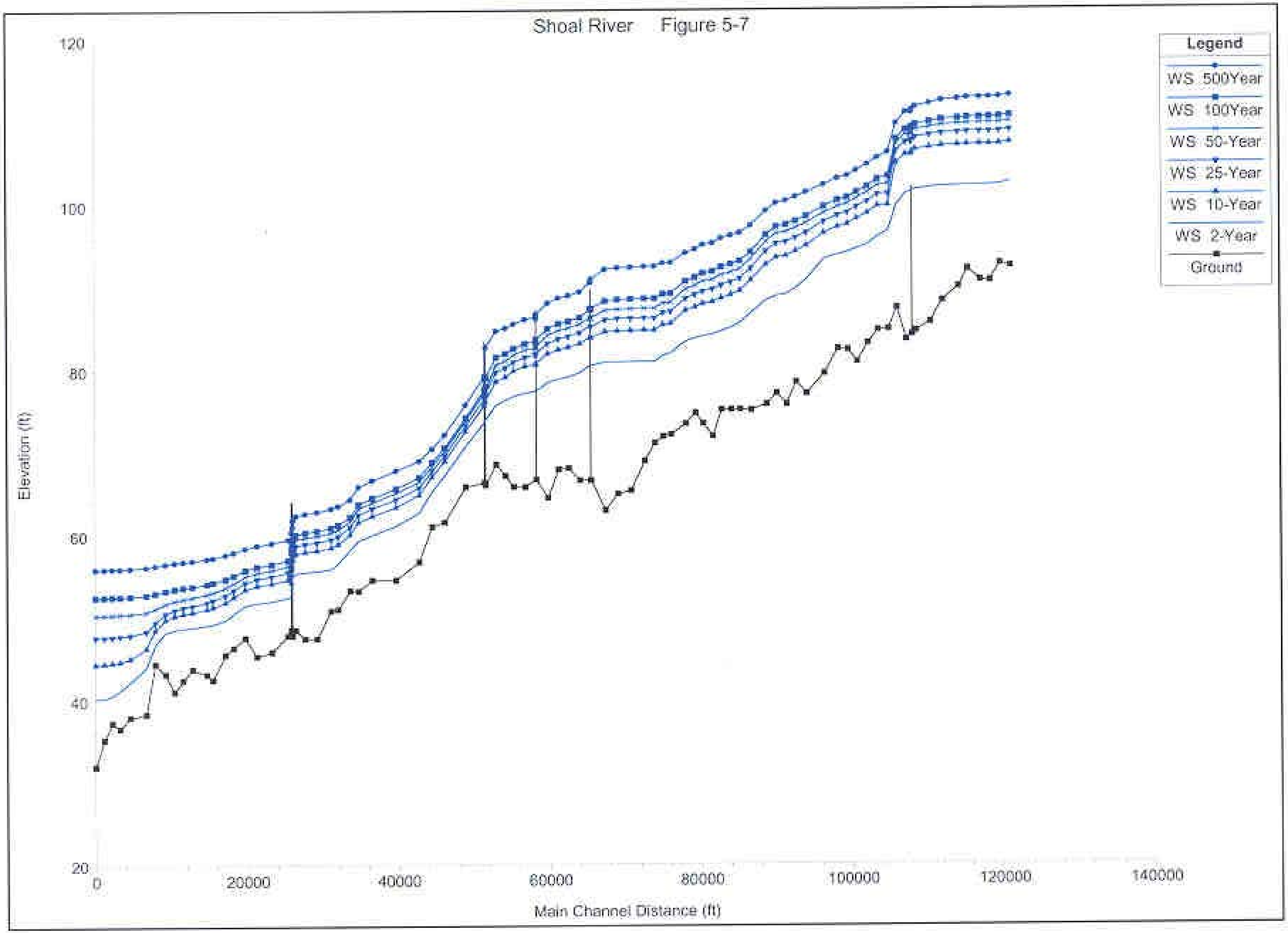
Shoal River 100-Year and 500-Year Flood Delineations

Figure 5-6



Shoal River Figure 5-7
Legend
WS 500Year
WS 100Year
WS 50-Year
WS 25-Year
WS 10-Year
WS 2-Year
Ground
Elevation (ft)
120
100
80
60
40
20
0
20000
40000
60000
80000
100000
120000
140000
Main Channel Distance (ft)

Table 5.5
Shoal River Drainage Basin
Existing Hydraulic Capacity and Return Period of Stream Crossings Summary

Structure Id. No.[1]	Location	Existing Structure Type	Roadway Classification	Hydraulic Capacity Return Period	
				Required	Actual
54	CR-393 Bridge	Bridge	Local	10-yr	2-yr
96	US-90 Bridge	Bridge	Arterial	50-yr	100-yr
98	SR-85 Bridge	Bridge	Arterial	50-yr	500-yr
98 (overflow)	SR-85 Overflow Bridge	Bridge	Arterial	50-yr	500-yr
102	I-10 Bridges	Bridge	Interstate	100-yr	500-yr

1. See Figure 5-1 for location of structure identification number.

5.4 LEVEL OF SERVICE ANALYSES

Table 5.6 summarizes the results of the culvert LOS analyses within the Shoal River Basin. Within Table 5.6 the size of the existing culvert, storm frequency required by the LDC, overtopping frequency, and a recommendation are shown. All of the culverts were analyzed with either the 25- or 50-year return period storm event. Based on the analysis it is recommended that structures 92, 93 and 94 be replaced to increase the capacity to that required by the LDC.

Table 5.6
Shoal River Basin Culvert LOS Analysis Summary

Structure Id. No.[1]	Location	Existing Culvert	Storm Frequency	Overtopping Frequency	Recommendation
49	Airport Road at South Mildred Heaton HS	3 – 10'x7'	25-year	80	NA
50	Airport Road at Billy Teel's Pond	7'x5'	25-year	>500	NA
72	SR 85 at Watson Bay Branch	17'x6'	50-year	50	NA
73	SR 85 at Horsehead Creek	3–12'x10'	50-year	77	NA
86	I-10, William Branch	14'x5'	50-year	>500	NA
91	Hwy. 90, Eden Lake	6'x6'	50-year	>500	NA
92	Hwy. 90, Toms Creek	12'x6'	50-year	22	2–8'x6' or add 6'x6' barrel
93	Hwy. 90, Mill Creek	4'x3' 36"	50-year	>5	9'x5'
94	Okaloosa Lane, Mill Creek	2–120"	25-year	17	2–11'x9'
95	Hwy. 90, Piney Woods Creek	3–10'x9'	50-year	>500	NA
99	I-10, Juniper Creek	2–16'x7'	50-year	>500	NA
100	I-10, King Branch West	9'x4'	50-year	>37[2]	Note But Accept
101	I-10, King Branch East	9'x4'	50-year	>500[2]	NA
103	I-10, Long Creek	2–9'x4'	50-year	63	NA
104	I-10	8'x5'	50-year	154[2]	NA
105	I-10, Gum Swamp	2–9'x6'	50-year	63[2]	NA
106	I-10	2–9'x5'	50-year	83[2]	NA

1. See Figure 5-1 for location of structure identification number.
2. Without desilting.

5.5 DETAILED STUDY AREAS

5.5.1 Antioch Road

Antioch Road is a rural collector roadway that connects P.J. Adams Parkway to SR 85, and provides access for adjoining neighborhoods. The County maintains a segment of the facility from west of P.J. Adams to east of Twain Lane (West Segment), and another segment from east of Ashley Drive to west of Juniper Creek (East Segment). The City of Crestview maintains the remainder of the roadway. This study focuses on the portion of the facility within County jurisdiction, and investigates pavement performance concerns and the adequacy of existing cross drains. A location map showing the limits of the East Segment, the limits of the West Segment, all significant County-maintained culvert crossings, and the drainage basins associated with each crossing appears as **Figure 5-8**.

5.5.1.1 Existing Conditions

The West Segment reportedly floods during extreme storm events. The segment features three significant culvert crossings, identified as Culverts A, B, and C in Figure 5-8. The NRCS Soil Survey Of Okaloosa County (Soil Survey) indicates poorly drained soils and seasonal high groundwater table (SHGWT) at or above existing ground in the vicinity of the culvert crossings.

The East Segment experiences frequent flooding, and has a reported history of poor pavement performance. In addition, as shown in **Figure 5-9**, runoff stands on the pavement following storm events. The segment features two significant culvert crossings identified as Culverts D and E in Figure 5-8, both of which are reported to overtop frequently. The Soil Survey indicates poorly drained soils and seasonal high groundwater at or above existing ground in the vicinity of all crossings.

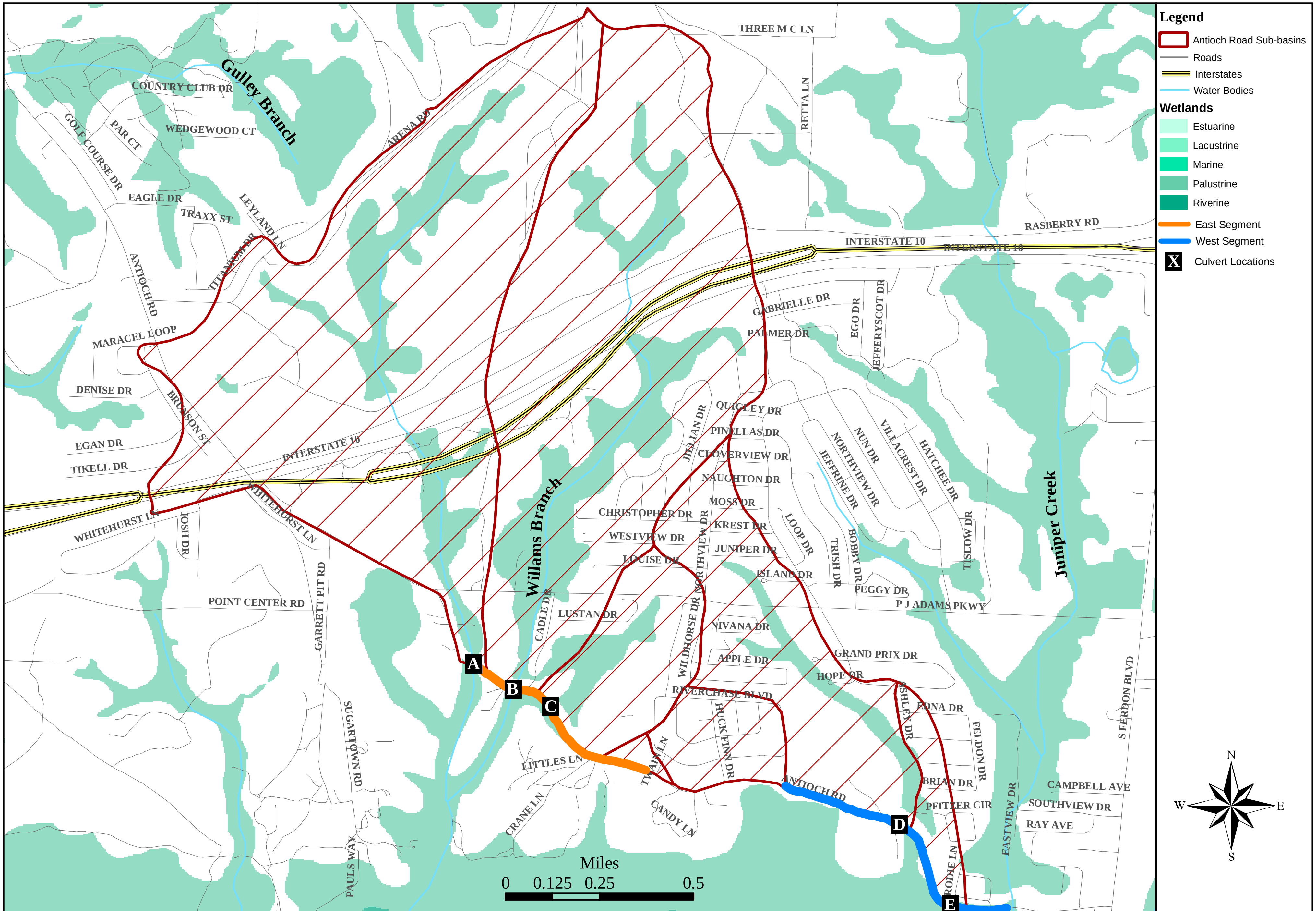
Figure 5-9
Antioch Road Photographs

Antioch Road Near Culvert A

Antioch Road Near Culvert E

5.5.1.2 Culvert Analysis

All culverts along the corridor were analyzed against a desired overtopping frequency of 25 years. The Rational Method was used to determine peak runoff, applying existing land use conditions as described in **Figure 5-10**. All analyses were performed considering both inlet and



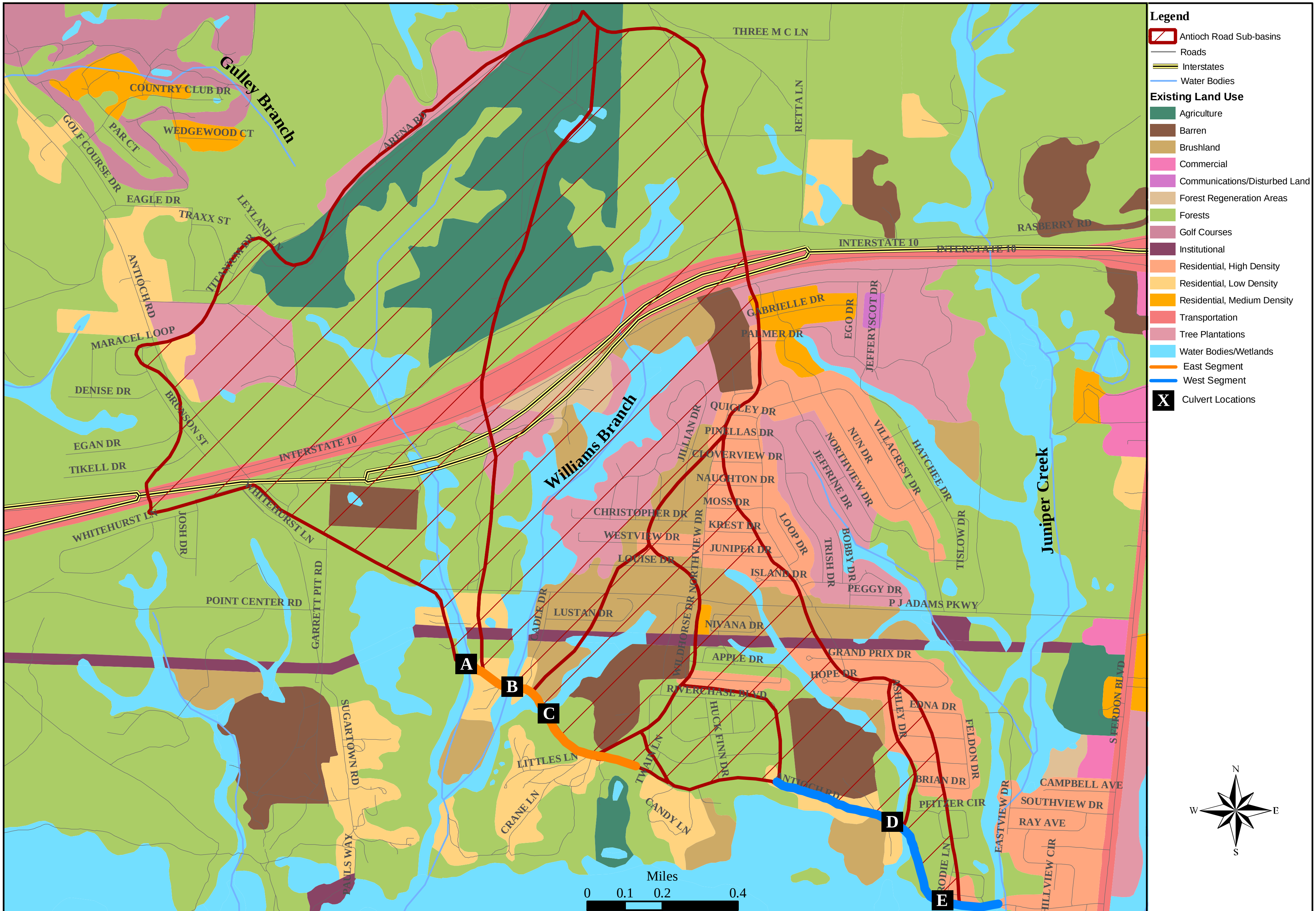
OKALOOSA COUNTY

Master Stormwater Management Plan

Antioch Road

Figure 5-8

HDR



Master Stormwater Management Plan

Antioch Road Existing Land Use

Figure 5-10

outlet control following HDS-5 procedures as applied by HY-8. As shown in **Table 5.7** these analyses indicate a need to upgrade all of the structures.

Table 5.7
Antioch Road Culvert Analysis Summary

Culvert	Basin Area (acres)	Existing Culvert Size	Proposed Culvert Size	Existing Overtopping Frequency (yr)	Proposed Overtopping Frequency (yr)
A	586	4 – 36" CMP	4 – 48" RCP	6	33
B	531	2 – 48" CMP	3 – 6'W x 4'H CBC	2	36
C	83	18" RCP	2 – 36" RCP	1	26
D	182	18" CMP	3 – 42" RCP	<1	53
E	33	18" CMP	36" RCP	<1	33

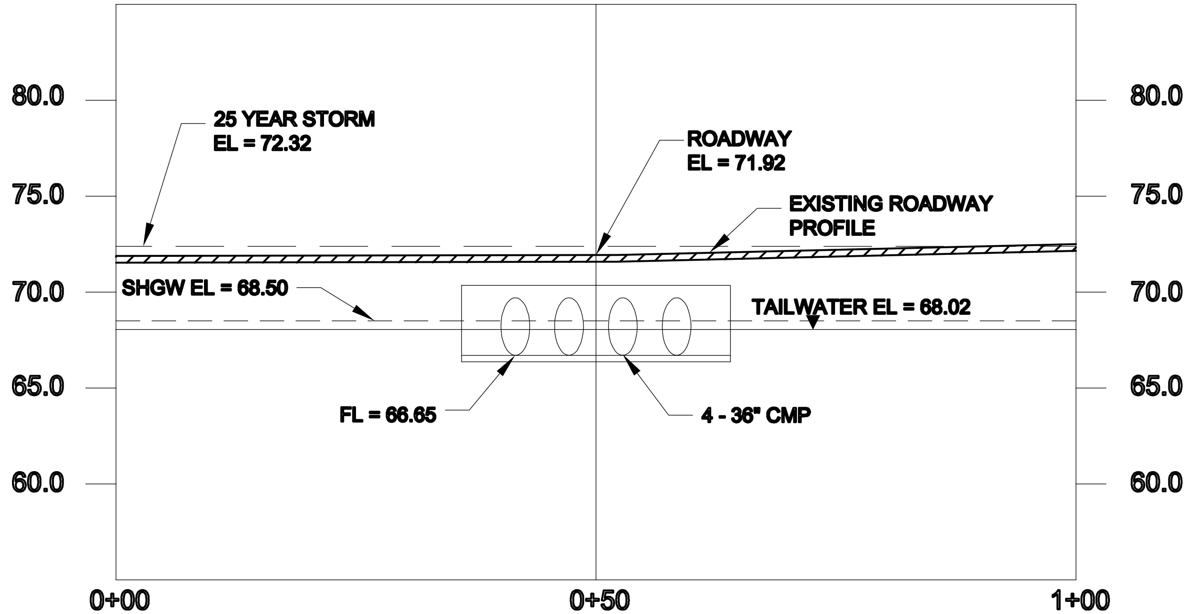
Culvert diagrams describing existing and proposed conditions at all culvert crossings appear in **Figures 5-11** through **5-15**. Detailed analytical results appear in the Hydrologic and Hydraulic Appendices (under separate cover).

5.5.1.3 Pavement Performance

In high groundwater areas, poor pavement performance can often be linked to water saturating the base. In this regard, published authority, including Section 2.6 of the Florida Department of Transportation (FDOT) Plans Preparation Manual (January 2003), recommends a base clearance of 1 to 2 feet over seasonal high groundwater with regard to two-lane rural facilities. Based on a comparison of roadway elevations to SHGWT elevations as reported in the Soil Survey, the existing profile along the West Segment provides a base clearance of at least 1 foot at all culvert crossings. However, the existing profile along the East Segment results in seasonal base saturation.

Common methodologies used to correct pavement problems associated with base saturation include raising the profile, using a non-absorbent base material, and installing roadside ditches with sufficient depth to drain the base. In this regard, this study recommends reconstructing the East Segment using non-absorbent base (e.g. FDOT Type B-12.5), raising the profile 2 to 3 feet throughout the East Segment, and constructing a roadside ditch along the north side of the East Segment. With regard to the West Segment, the lack of frequent saturation indicates that the proposed drainage improvements combined with resurfacing where needed should adequately protect the pavement.

Existing Conditions



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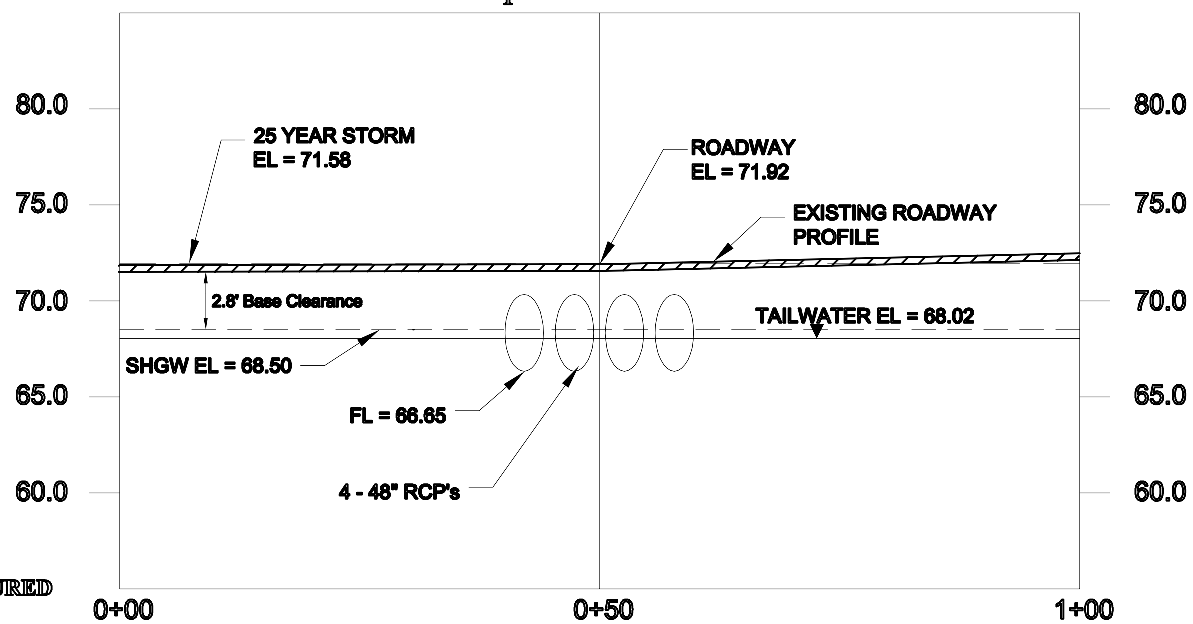
V: 1"=10'

Proposed Conditions



SCALE
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V: 1"=10'

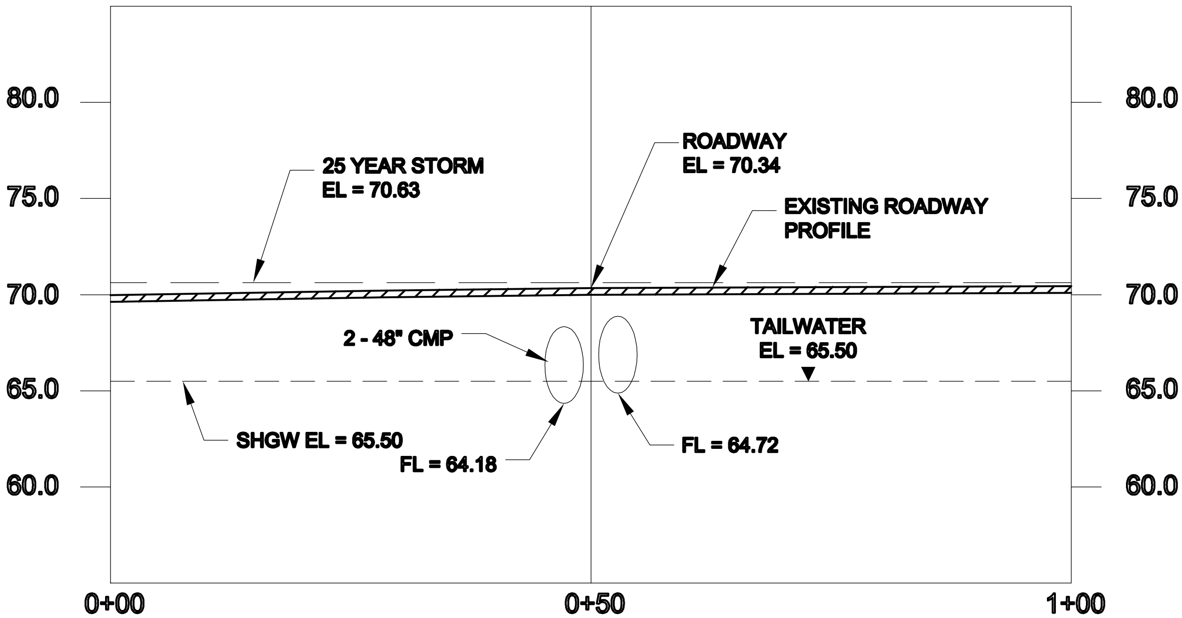
NOTE: BASE CLEARANCE MEASURED AT PROFILE LOW POINT

Okaloosa County Master Stormwater Management Plan

Antioch Road Proposed Improvement Culvert A

Figure 5-11 HDR

Existing Conditions



SCALE
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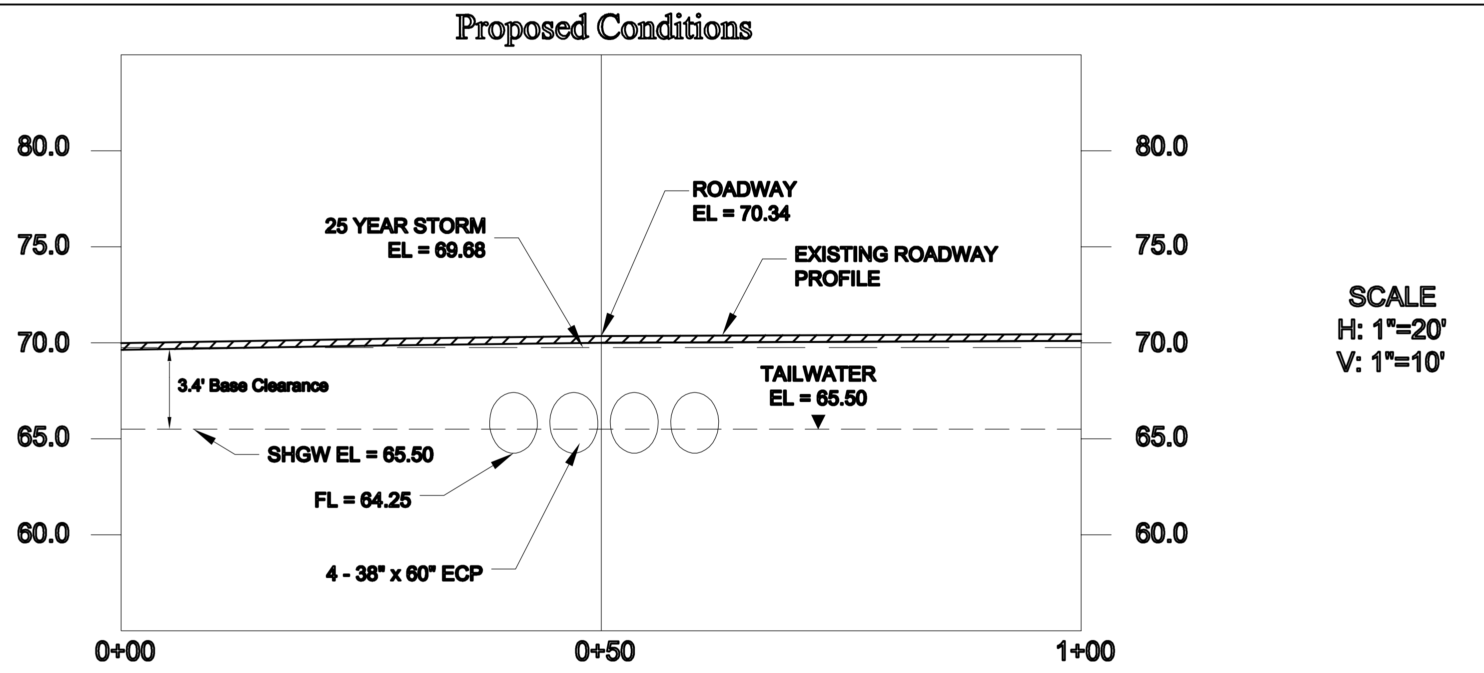
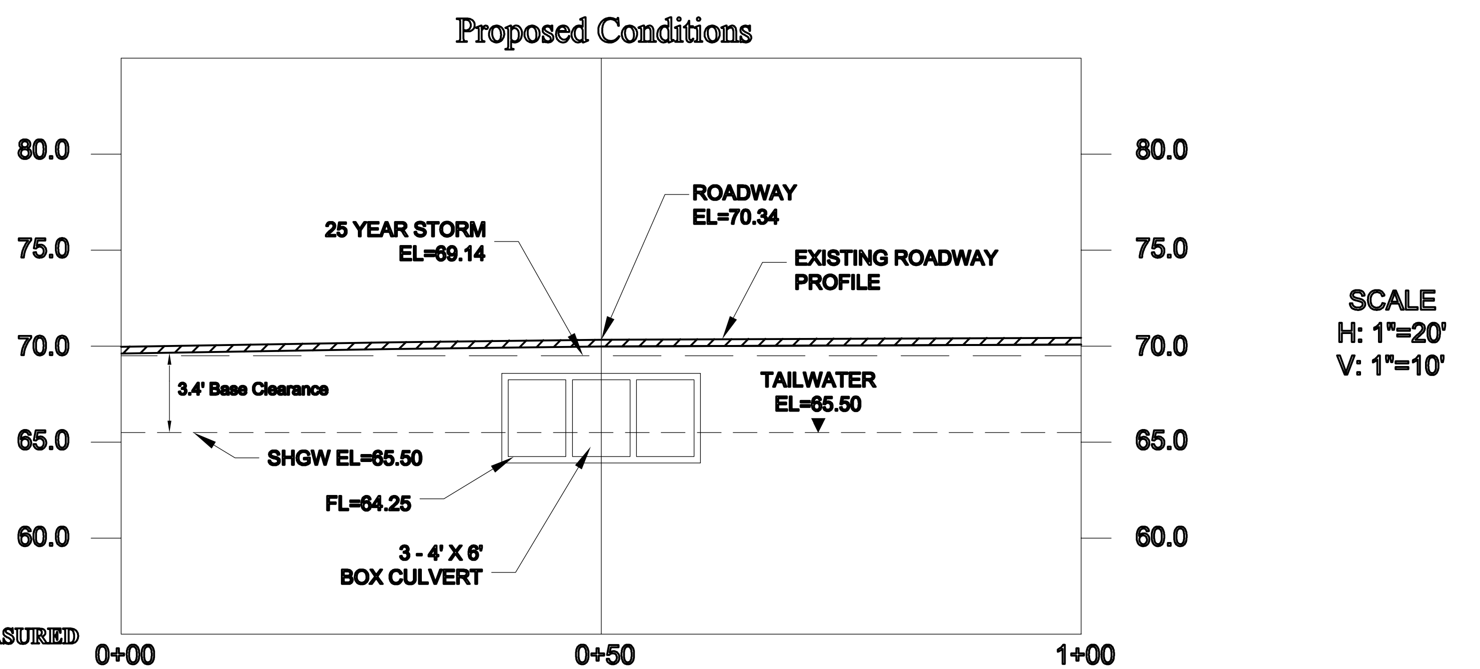
NOTE: BASE CLEARANCE MEASURED
AT PROFILE LOW POINT

Master Stormwater Management Plan	Antioch Road Proposed Improvement Culvert B	Figure 5-12a





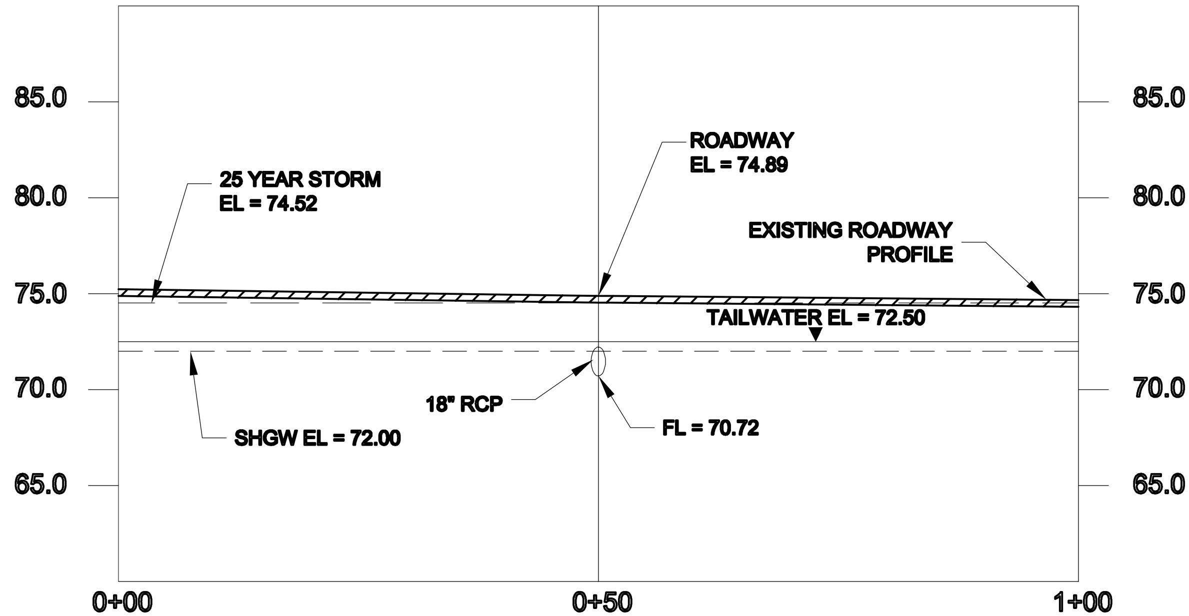
NOTE: BASE CLEARANCE MEASURED AT PROFILE LOW POINT

Master Stormwater Management Plan

Antioch Road Proposed Improvement Culvert B

Figure 5-12b

Existing Conditions



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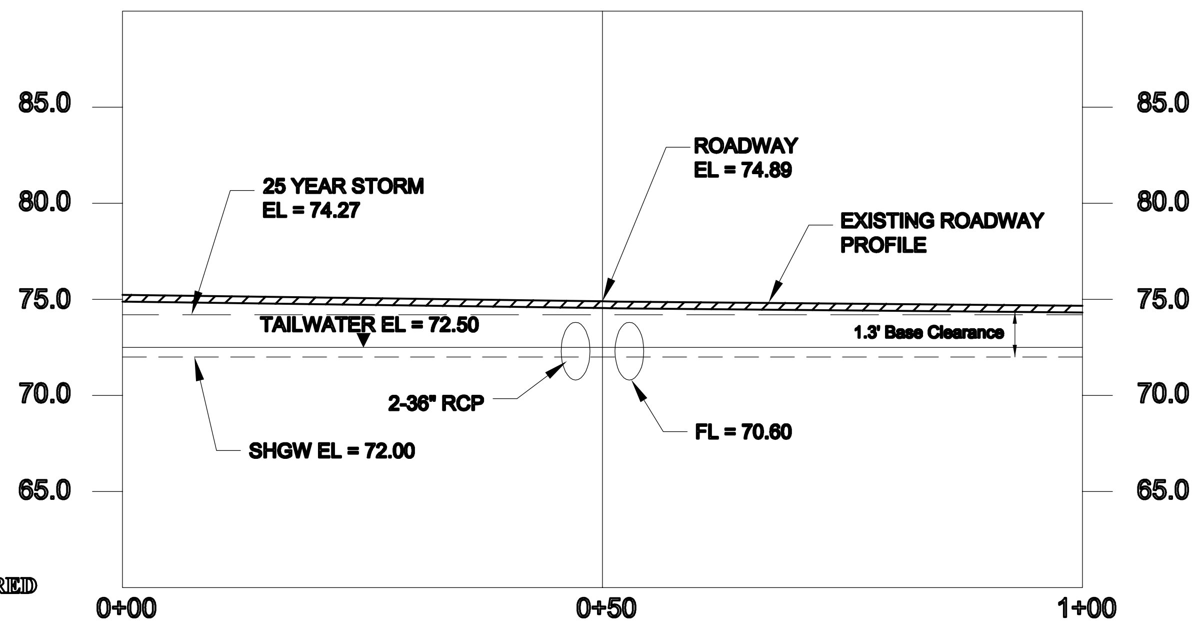
V: 1"=10'

Proposed Conditions



SCALE
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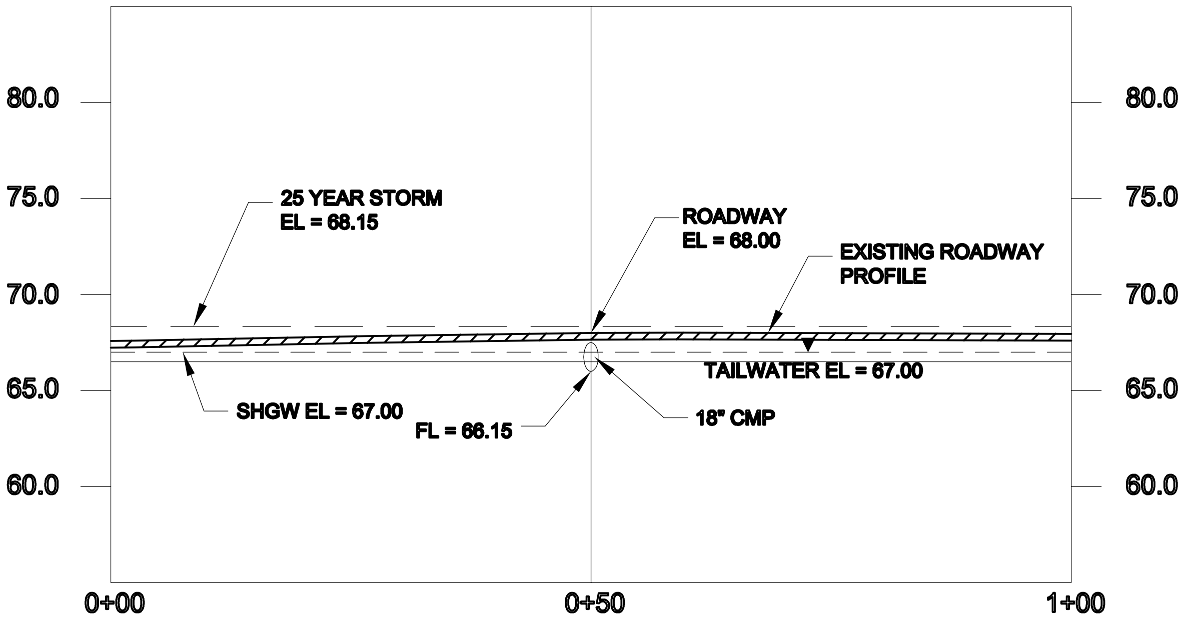
V: 1"=10'

NOTE: BASE CLEARANCE MEASURED
AT PROFILE LOW POINT

Master Stormwater Management Plan	Antioch Road Proposed Improvement Culvert C	Figure 5-13

Existing Conditions



SCALE
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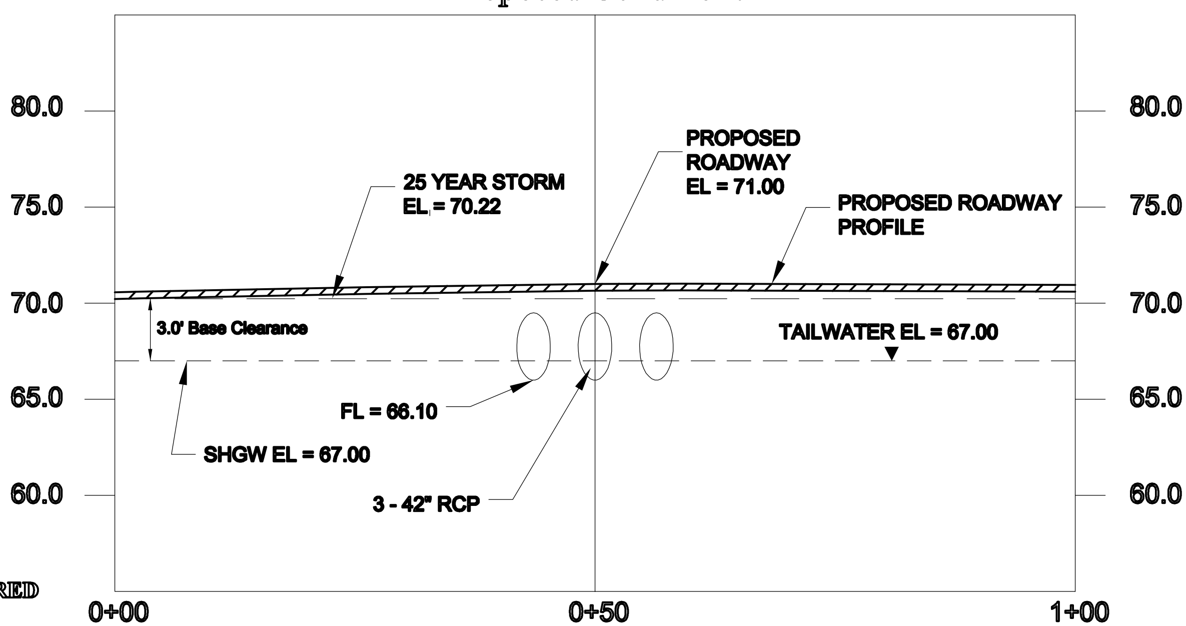
V: 1"=10'

Proposed Conditions



SCALE
H: 1"=20'
V: 1"=10'

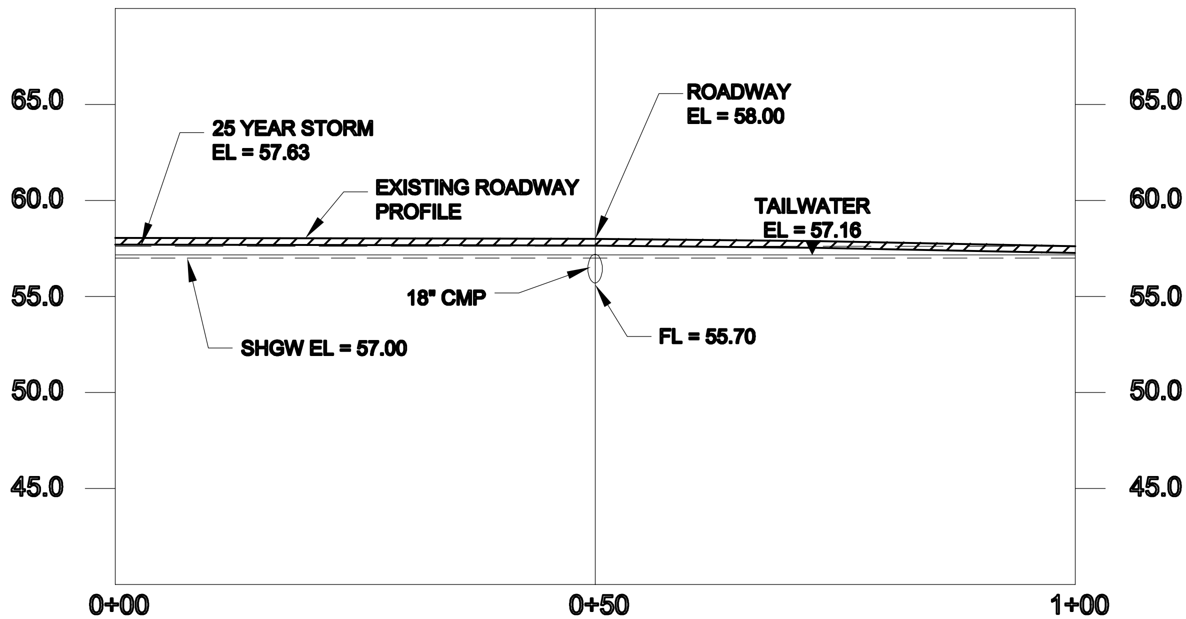
NOTE: BASE CLEARANCE MEASURED AT PROFILE LOW POINT

Master Stormwater Management Plan

Antioch Road Proposed Improvement Culvert D

Figure 5-14

Existing Conditions



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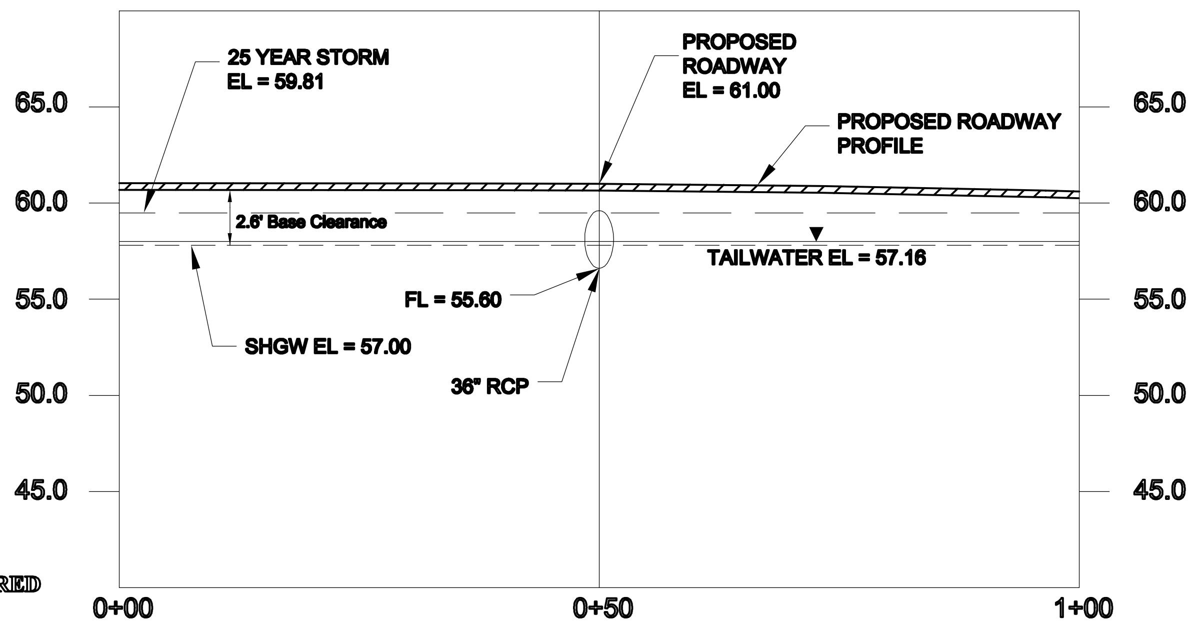
Proposed Conditions



SCALE
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V: 1"=10'

NOTE: BASE CLEARANCE MEASURED
AT PROFILE LOW POINT

Okaloosa County

Master Stormwater Management Plan

Antioch Road Proposed Improvement Culvert E

Figure 5-15

HDR

5.5.1.4 Summary

Table 5.8 presents a summary of the Antioch Road detailed study area recommendations.

Table 5.8
Antioch Road Recommendations

West Segment	East Segment
Resurface As Needed Upgrade Culverts	Reconstruct Roadway With Non-Absorbent Base Raise Profile 2 to 3 feet Upgrade Culverts Install Ditch Along North Side Of Corridor

6.0 COASTAL BASINS

6.1 GENERAL BASIN DESCRIPTION

There are two coastal basins in Okaloosa County, the East Bay Basin and the Choctawhatchee Bay Basin. These two basins are located in the southern portion of the County and are shown in **Figure 6-1**. The East Bay drainage basin measures approximately 114 square miles, of which 99 percent is within the County boundary. The Choctawhatchee Bay drainage basin measures approximately 255 square miles, of which 194 square miles are within the County boundary. Portions of the Choctawhatchee Bay Basin extend into Walton County. These basins are bounded by the Gulf of Mexico to the south and follow no landmarks to the north.

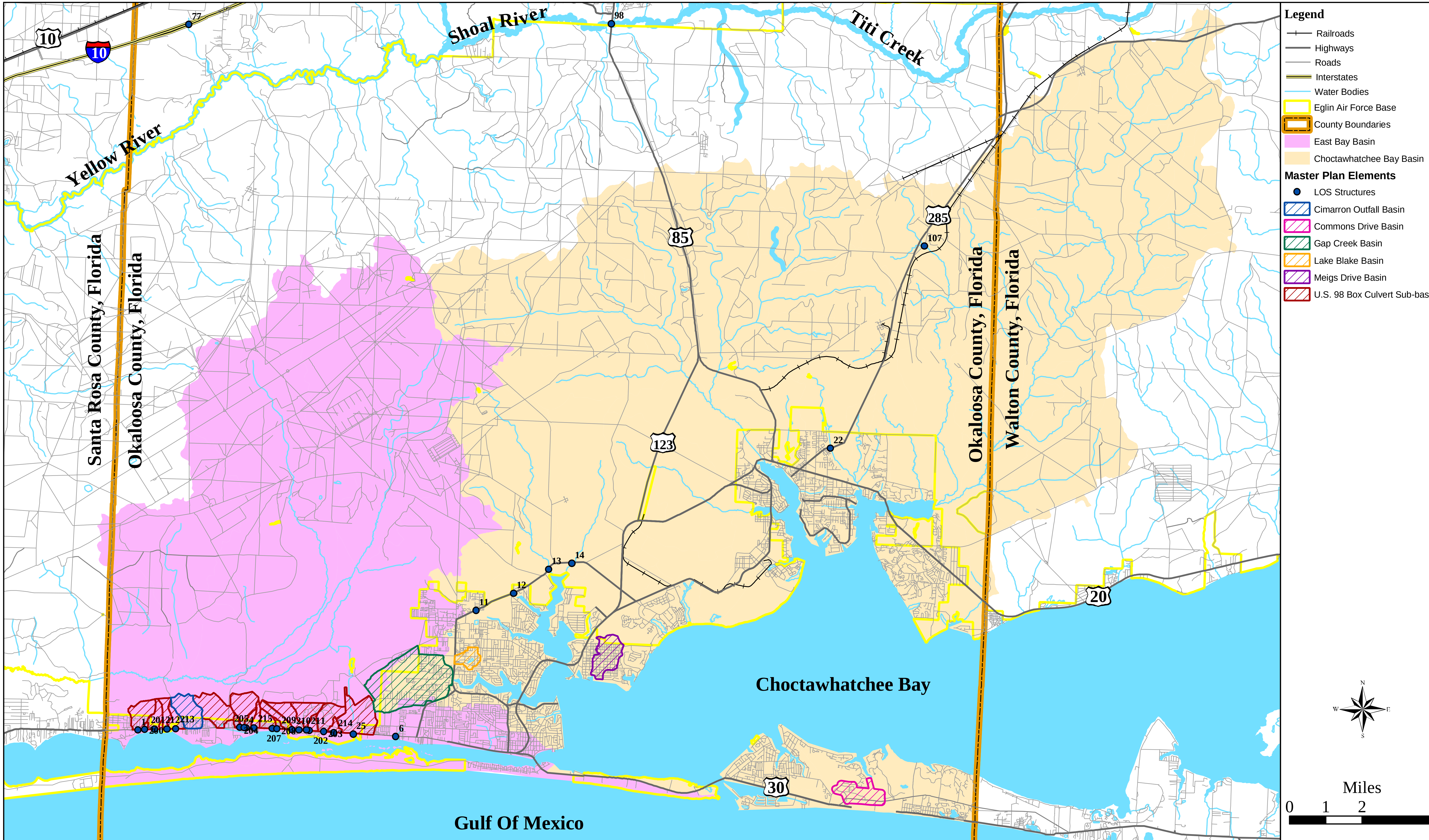
Elevations in the basins range from approximately 0 feet along the Bay to 295 feet in the northernmost area of the Choctawhatchee Bay basin in Walton County. Within Okaloosa County the elevations range from approximately 0 feet to 260 feet.

Table 6.1 shows the relative representation and general hydrologic characteristics for the soils found in the coastal basins within Okaloosa County. Within Okaloosa County the coastal basins contain 19 different soil types, of which the Lakeland series accounts for close to 100 percent of the total basin area. Although these basins consist mainly of the Lakeland series other noteworthy soil types are the Dorovan and Pickney series which surround the East Bay River and the Newhan and Koreb series which are the primary soils on Santa Rosa Island and the Destin Peninsula. For detailed study area analysis purposes, the different soil types were grouped by NRCS hydrologic soil type as Type A, C, and D. Both basins almost entirely consist of Type A soils as depicted in **Figure 6-2**.

Table 6.1
East Bay Basin and Choctawhatchee Bay Basin Soil Type Summary
(Okaloosa County)

Soil Series	General Hydrologic Characteristics	Texture	% Area
Lakeland	(0 to 30% slopes) Gently sloping excessively drained soil on nearly level to steep uplands. Rapidly permeable with slow runoff.	Sand	98.4
	Various soils, 13 soil types ranging from 0.01% to 0.96% area.		1.6
	Total Percent Area		100.0

Source: Soil Survey of Okaloosa County, Florida; NRCS June 1995.



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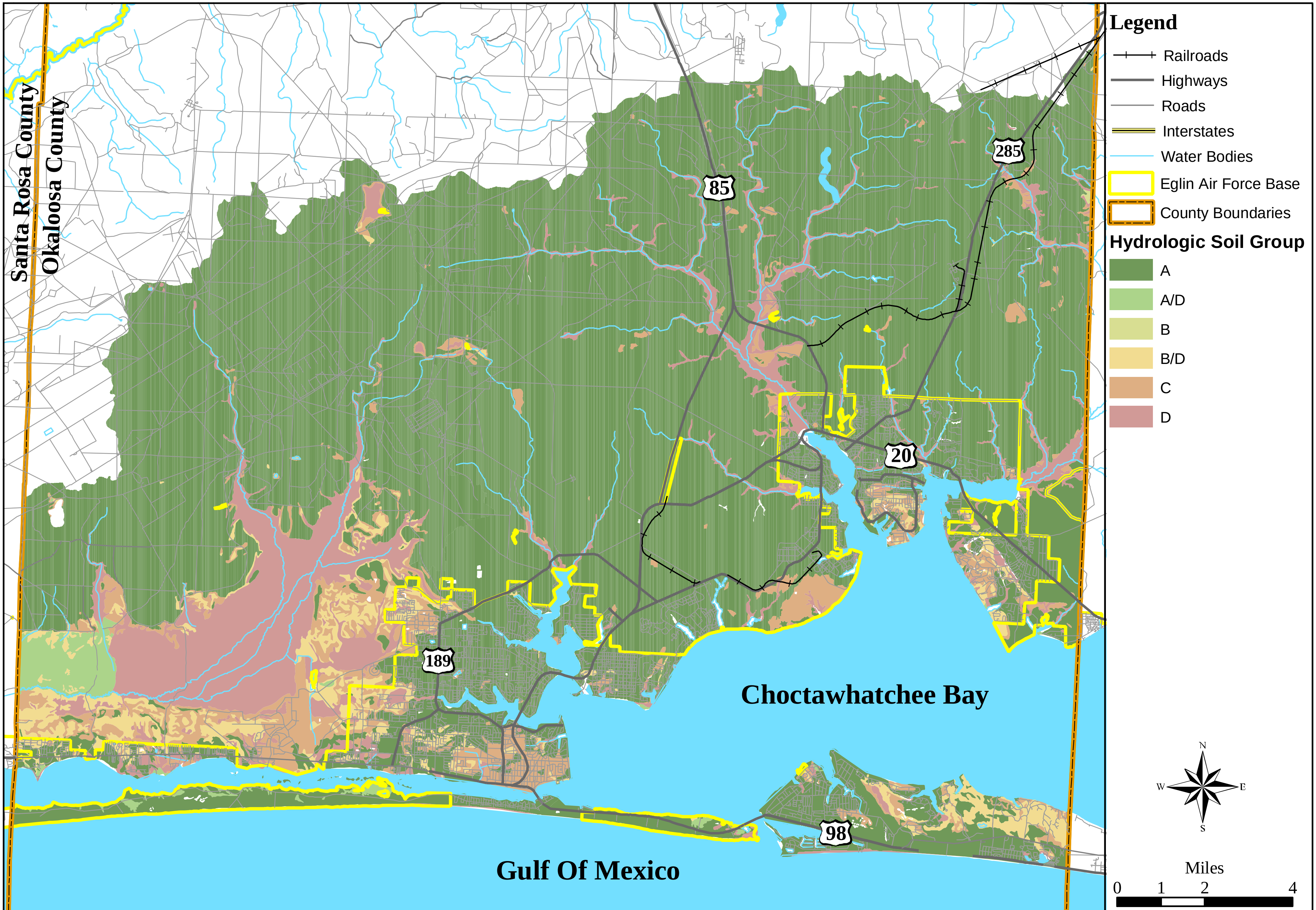
East Bay Basin and Choctawhatchee Bay Basin

Figure 6-1



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East Bay Basin And Choctawhatchee Bay Basin NRCS Soil Classification (within Okaloosa County)

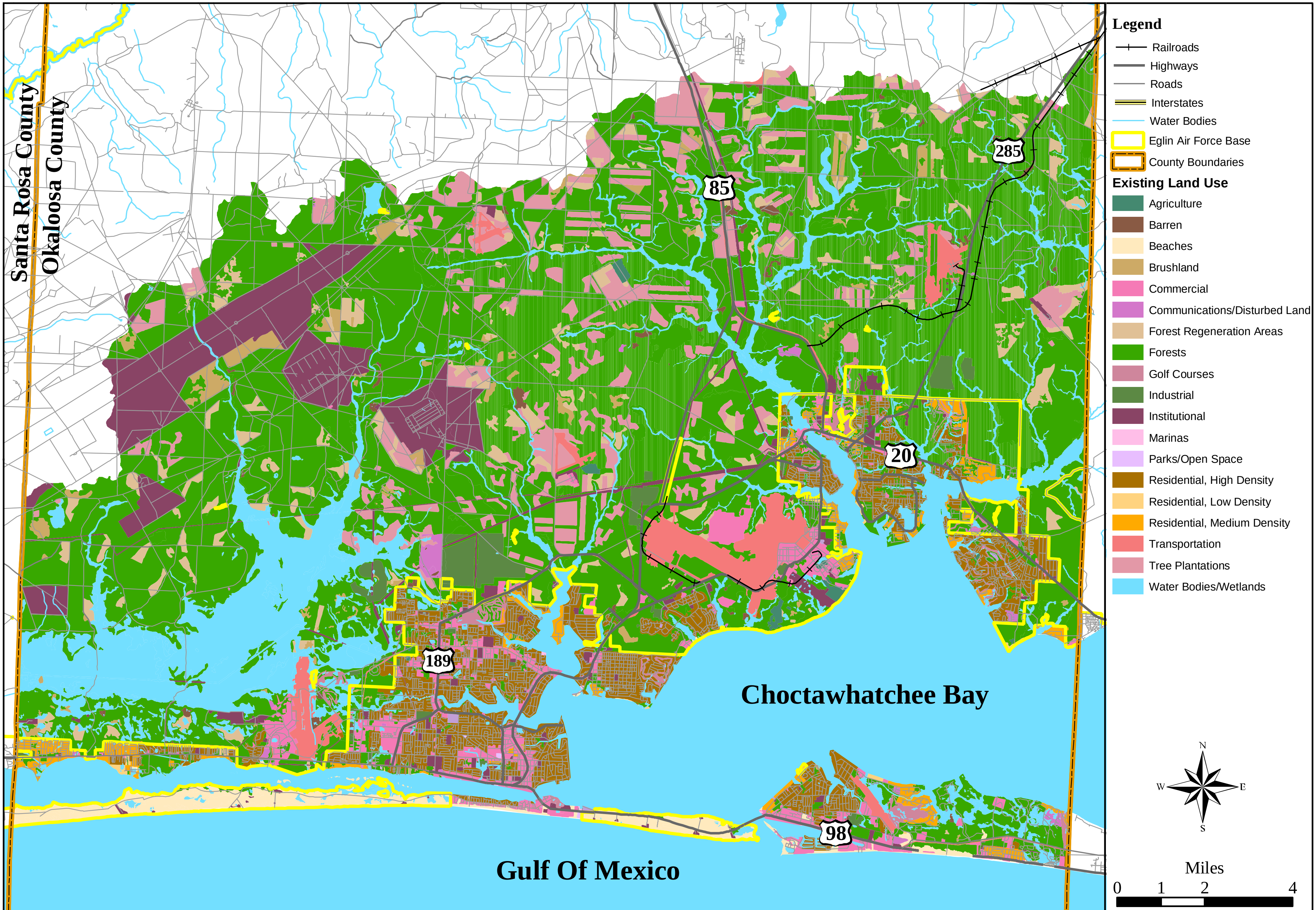
Figure 6-2

Land use classifications in the East Bay and Choctawhatchee Bay Basin range from forests to residential, with the majority of the basin classified as forest land. The breakdown of existing land use (grouped by classifications used for the H&H models) within the Coastal River Basin is shown in **Figure 6-3**.

Figure 6-4 shows the future land use (grouped by categories used for H&H models) within the Blackwater River Basin based on the County's future land use map, the municipalities' future land use maps, and existing land use data where necessary as discussed in Section 2.2.1.2. As shown the future land use is quite similar to the existing land use. **Table 6.2** shows a comparison of the percentage of each land use classifications for both existing and future conditions.

Table 6.2
East Bay Basin and Choctawhatchee Bay Basin
Existing and Future Land Use Summary
(Okaloosa County)

Land Use Group	Existing	Future
Agriculture	<1	<1
Barren	<1	<1
Beaches	<1	<1
Brushland	<1	<1
Commercial	<1	<1
Communications/Disturbed Land	<1	<1
Forests	83	83
Forest Regeneration	<1	<1
Golf Courses	<1	<1
Industrial	<1	<1
Institutional	2	3
Marinas	<1	<1
Parks/Open Space	<1	<1
Residential, High Density	2	2
Residential, Low Density	<1	<1
Residential, Medium Density	<1	<1
Transportation	1	1
Tree Plantations	1	1
Water Bodies/Wetlands	8	8
Total	**100**	**100**



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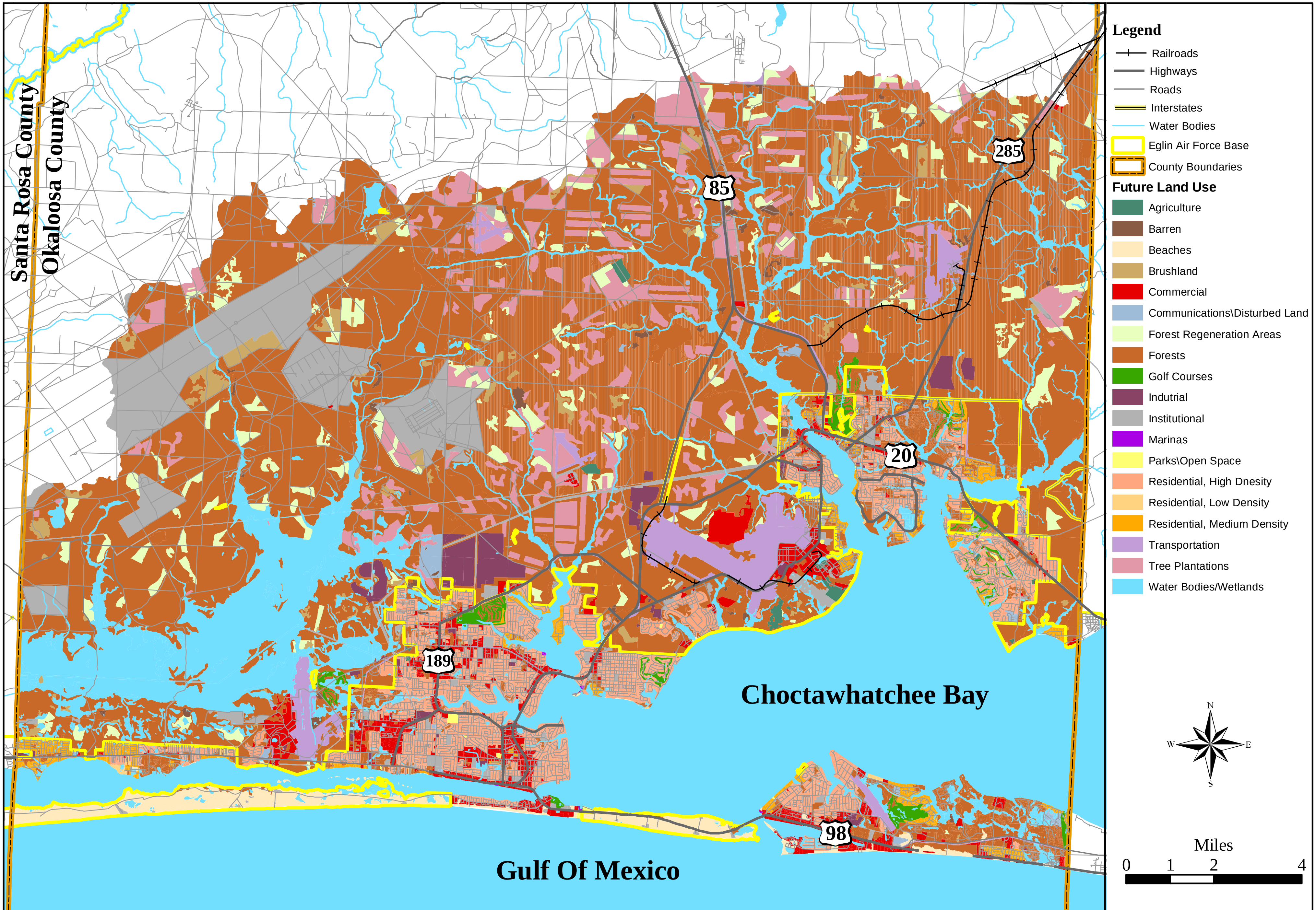
East Bay Basin And Choctawhatchee Bay Basin Existing Land Use (within Okaloosa County)

Figure 6-3



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East Bay Basin And Choctawhatchee Bay Basin Future Land Use (within Okaloosa County)

Figure 6-4

As previously noted, the future land use was based on the County's future land use map, the municipalities' future land use maps, and existing land use data where necessary. Figure 6-4 shows the future land use within the East Bay and Choctawhatchee Bay Basins.

6.2 LEVEL OF SERVICE ANALYSES

Table 6.3 summarizes the results of the culvert LOS analysis within the East Bay and Choctawhatchee Bay Basins. Within Table 3.3 the size of the existing culvert, storm frequency required by the LDC, overtopping frequency, and a recommendation are shown. All of the culverts were analyzed with the 25- or 50-year return period storm event. Based on the analysis it is recommended that structures 13, 14, 202, 203, 207, and 210-213 be replaced to increase the capacity to that required by the LDC.

Table 6.3
Coastal Basins Culvert LOS Analysis Summary

No.[1]	Location	Existing Culvert	Storm Frequency	Overtopping Frequency	Recommendation
1	US 98, west of Wynnhaven Beach Road	9.3’x4’	50-year	>500-year	NA
3	US 98 Near Timberlake Drive	8’x6’	50-year	>500-year	NA
4	US 98, 350 feet east of Skylark Road	30”	50-year	>500-year	NA
6	US 98, west of Doolittle	10’x4’	50-year	>500-year	NA
13	SR 189, over Lightwood Knot Creek	3-10’x6’	50-year	21-year[2] 7-year[3]	Add 13’x6’ Barrel
14	SR 189 over Garnier Creek	3-10’x4’	50-year	8-year	4-12’x7’
16	SR 85, over Tom’s Creek	3-10’-x9’	50-year	400-year[3]	NA
22	SR 285, over Swift Creek	2-10’x7’	50-year	200-year[3]	NA
25	US 98, east of Hurlburt runway	3-8’x3’’	50-year	455-year[3]	NA
201	US 98, 250 feet east of Magnolia Shores	30”	50-year	10-year	42”
202	US 98, 250 feet east of Hurlburt Ped overpass	48”	50-year	24-year	2-48”
203	US 98, 1500 feet east of Hurlburt Gate	5’x3’	50-year	1.2-year	3-9’x3’
205	US 98, 50 feet east of Neptune Drive	12’x4’	50-year	>500-year	NA
207	US 98, 125 feet west of Leisure Tyme RV	30”	50-year	4-year	48”
209	US 98, 1000 feet east of Tom Thumb	2-36”	50-year	>500-year	NA
210	US 98, southwest corner of Hurlburt Field Housing	42”	50-year	13-year	48”
211	US 98, 75 feet west of Ped Overpass	2-48”	50-year	28-year	2-54”
212	500 feet east of 98 West Liquor Store	30”	50-year	22-year	2-30”
213	US 98, at Betta Store IT	36”	50-year	8-year	48”
215	US 98 East of Florosa Baptist	30”	50-year	93-year	NA
216	US 98, 300 feet east of Timbre Lake Drive	30”	50-year	>500-year	NA
217	US 98, at Sunset Produce	24”	50-year	>500-year	NA
218	US 98, 100 feet west of The Happy Stores	36”	50-year	46-year	NA

1. See Figure 6-1 for location of structure identification number.
2. After desilting.
3. Without desilting.

6.3 DETAILED STUDY AREAS

6.3.1 Gap Creek

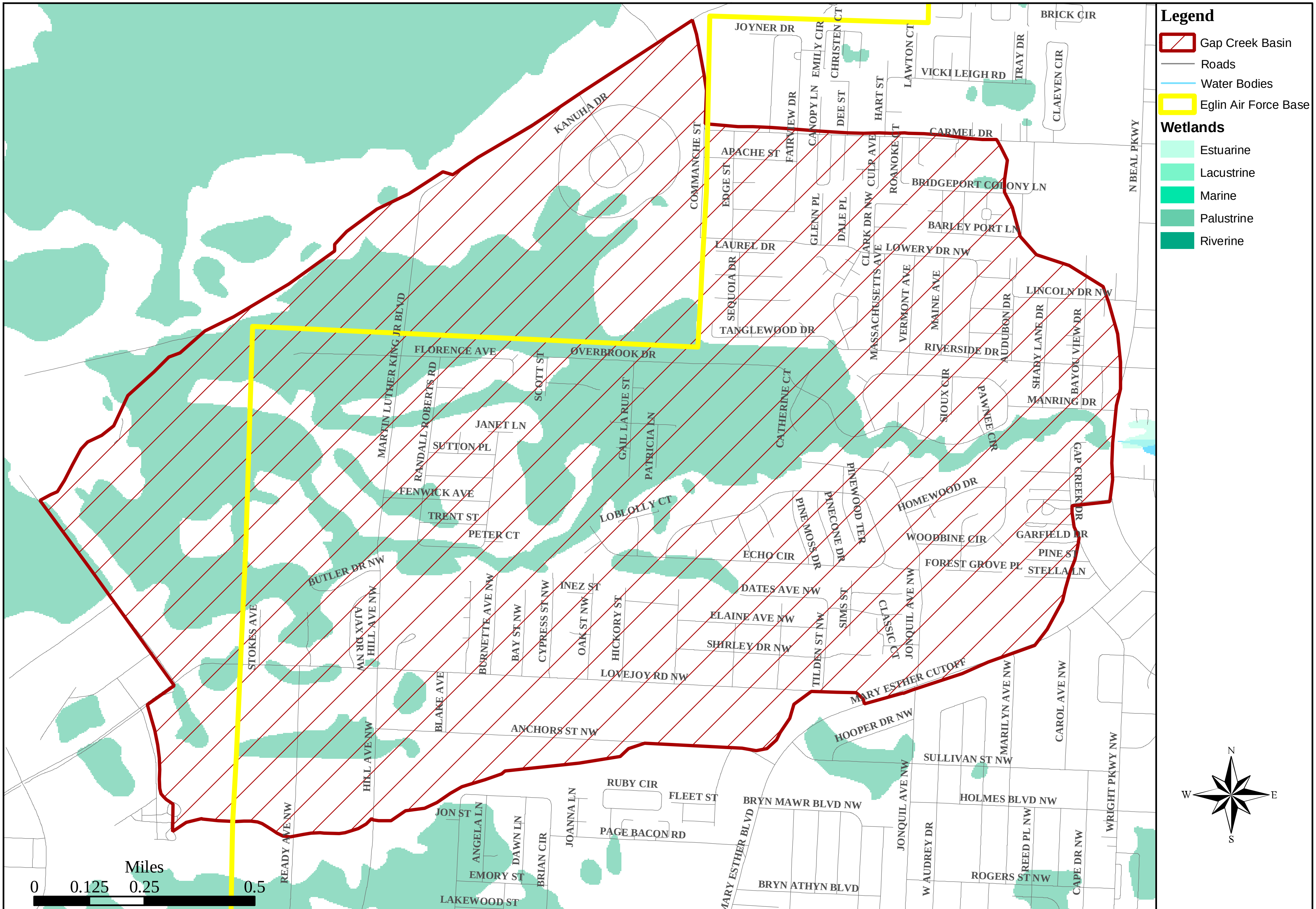
6.3.1.1 General Basin Description

Figure 6-6
Gap Creek Photograph

Gap Creek is located in the Fort Walton area as shown in **Figure 6-5**, and consists of a main channel with one significant tributary that joins the main channel near the Overbrook subdivision. Gap Creek has a drainage basin of approximately 2.9 square miles and discharges to Cinco Bayou. Beale Parkway to the east, a runway at Hurlburt Field to the west, Lovejoy Road and Hollywood Boulevard to the south, Mary Esther Cutoff to the southeast, and Carmel Road to the north roughly bound the basin. Martin Luther King Boulevard represents the only road that crosses Gap Creek, and features two separate culvert crossings that serve both the main channel and the tributary. **Figure 6-6** illustrates existing conditions.

The basin can be divided into two regions with distinctly different land uses. The upper basin (Upper Basin), located west of Martin Luther King Boulevard, is mostly undeveloped and consists of a mixture of privately held and government property. Much of the vacant land in the upper basin contains jurisdictional wetlands that provide quality and rate control benefits to Gap Creek. The lower basin (Lower Basin), located east of Martin Luther King Boulevard, has been developed to near saturation. The development consists mostly of single-family residential homes, with commercial uses along the basin's collectors and arterial roadways. All vacant land remaining in the Lower Basin appears to be jurisdictional. A map showing existing land use within the Gap Creek Basin is presented as **Figure 6-7.**

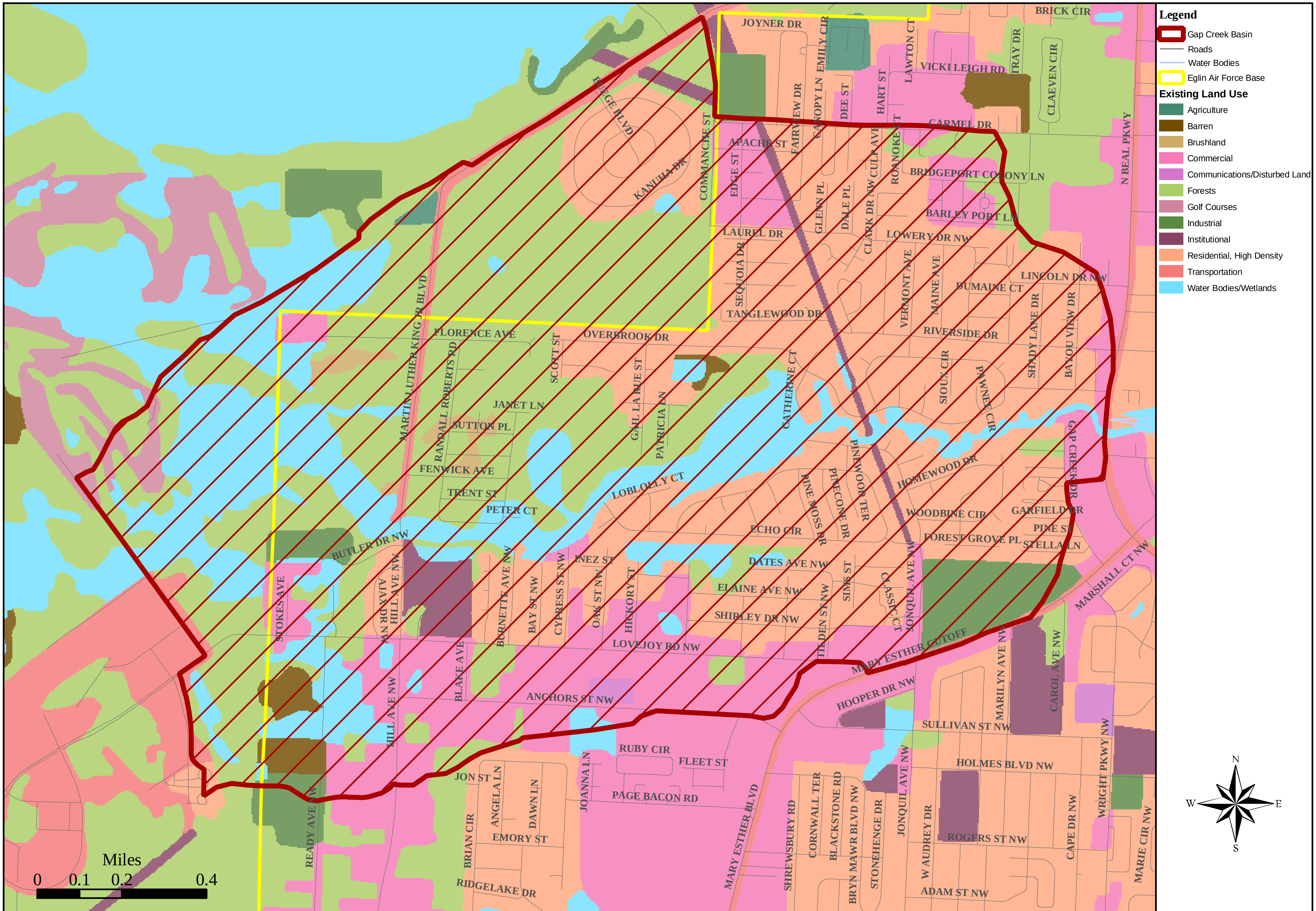
Due to the high density of development already present in the Lower Basin, all significant future development involving changes to the impervious area will likely occur in the Upper Basin. In this regard, based on future land use maps and conversations with County permitting authorities, it is anticipated that all non-jurisdictional land in the Upper Basin that is privately owned will be developed with a commercial land use. A map showing anticipated future land use within the Gap Creek Basin appears as **Figure 6-8**.



Master Stormwater Management Plan

Gap Creek

Figure 6-5



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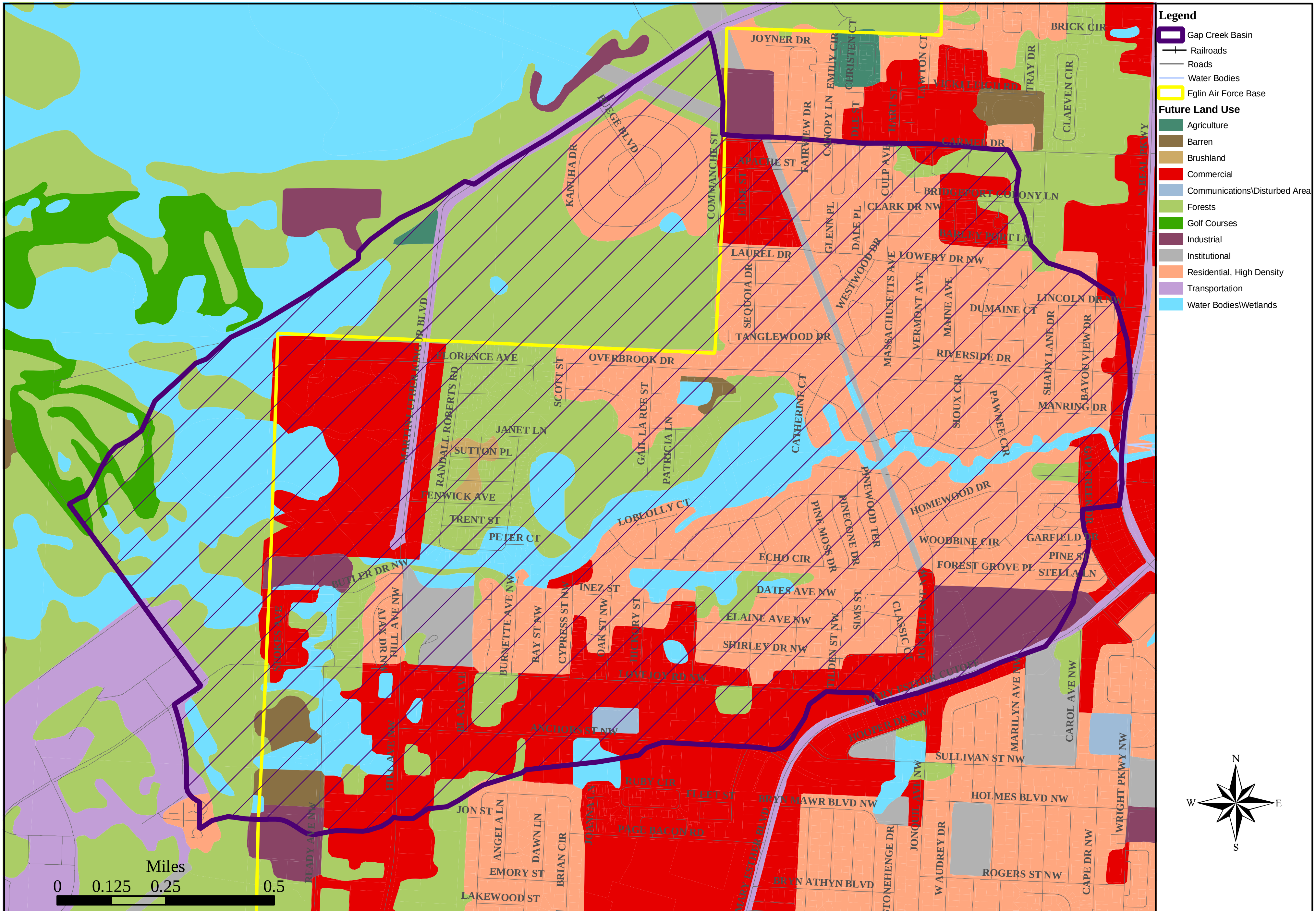
Gap Creek Existing Land Use

Figure 6-7



Master Stormwater Management Plan

Gap Creek
Future Land Use

Figure 6-8

6.3.1.2 Flood Hydrology

The Gap Creek basin was delineated into six sub-basins, including four on the main channel and two on the tributary. The upper basins of the main channel and the tributary are located west of Martin Luther King Boulevard. The middle basins of the main channel and tributary extend from Martin Luther King Boulevard to the Gap Creek and tributary confluence. The remaining basins are located downstream of the confluence.

For existing conditions, it was noted that the terrain in the upper sub-basins provides natural stormwater retention. Therefore, the stormwater runoff model was routed through several reservoirs identified by contours and aerial photography. Elevation/storage/flow curves were developed using the spatial terrain characteristic and HEC-RAS culvert analyses.

For future conditions, planned development in the basin west of Martin Luther King Boulevard will increase the amount of impervious area, and decrease available storage in natural ponding areas. Therefore, the future condition model did not include the extent of natural reservoir storage analyzed for existing conditions. This assumption is conservative, because according to development standards, a portion of project runoff must be contained on the site of future development using best management practices, such as retention ponds.

The SCS Transform method was used to generate runoff hydrographs and peak runoff rates for the 2-, 10-, 25-, 50-, 100-, and 500-year return period storm events, applying both existing and future development conditions. **Tables 6.4** and **6.5** provide a summary of existing and future peak runoff rates, respectively, for selected storm events at key locations in the Gap Creek Basin. A summary of the peak runoff rates for all sub-basin hydrographs can be found in the Hydrologic and Hydraulic Appendices.

Table 6.4
Gap Creek Drainage Basin
Peak Runoff Summary for Existing Drainage System Conditions

HEC-HMS Node No.	Location	Drainage Area (sq. mi)	Peak Runoff Rate (cfs) [1,2]					
			2-Year	10-Year	25-Year	50-Year	100-Year	500-Year
J1	MLK South	0.6	173	326	424	482	571	730
J3	MLK North	0.2	66	129	158	177	205	311
J4	At confluence	1.2	348	647	797	895	1071	1413
J2	Upstream of Lower sub-basin	2.0	812	1490	1802	2009	2321	2952
S1	At Beal Boulevard	2.9	1009	1862	2236	2485	2858	3608

1. Peak runoff rates based on existing land use condition and simulation of a 24-hour storm event.
2. Peak discharges reported are outflows from the specified nodes.

Table 6.5
Gap Creek Drainage Basin
Peak Runoff Summary for Future Drainage System Conditions

HEC-HMS Node No.	Location	Drainage Area (sq. mi)	Peak Runoff Rate (cfs) [1,2]					
			2-Year	10-Year	25-Year	50-Year	100-Year	500-Year
J1	MLK South	0.6	182	343	440	503	590	751
J3	MLK North	0.2	219	360	420	460	520	638
J4	At confluence	1.2	512	899	1065	1179	1350	1691
J2	Upstream of Lower sub-basin	2.0	940	1675	1994	2209	2528	3170
S1	At Beal Boulevard	2.9	1066	1940	2318	2570	2946	3701

1. Peak runoff rates based on future land use condition and simulation of a 24-hour storm event.
2. Peak discharges reported are outflows from the specified nodes.

6.3.1.3 Stream Hydraulics

HEC-RAS was used to determine the stream hydraulics of the channel and structures of Gap Creek and its tributary. The downstream boundary condition for the basin outfall was initially set to the mean high water of Cinco Bayou, 0.9 feet. However, the water surface elevation for all studied frequency storms was higher then the mean high tide elevation, and therefore the HEC-RAS analysis was re-evaluated with the normal depth boundary condition.

Figures 6-9a and **6-9b** show flood delineations for the 2-, 25-, and 100- year return period storm events, applying existing and future conditions, respectively. **Figures 6-10a** and **6-10b** show the flood profiles for the 2-, 10-, 25-, 50-, 100-, and 500-year return period storm events, applying existing and future conditions, respectively.

A total of two road crossings, both consisting of culverts under Martin Luther King Boulevard, were analyzed in the Gap Creek Basin. A summary of the existing and future hydraulic capacity for each of the crossings studied is presented in **Tables 6.6** and **6.7** for the 2-, 10-, 25-, 50-, and 100-year storm events. The bridge over Gap Creek at Beale Parkway was not modeled, but estimated modeling output does not indicate any potential overtopping problems at this location.

Table 6.6
Gap Creek Drainage Basin
Existing Hydraulic Capacity of Stream Crossings Summary

Location	Minimum Overtopping Elevation[1]	Depth of Overtopping (ft)[2]				
		2-Year	10-Year	25-Year	50-Year	100-Year
Martin Luther King Boulevard North	32.3	-	-	-	-	-
Martin Luther King Boulevard South	29.4	-	0.3	0.5	0.6	0.7

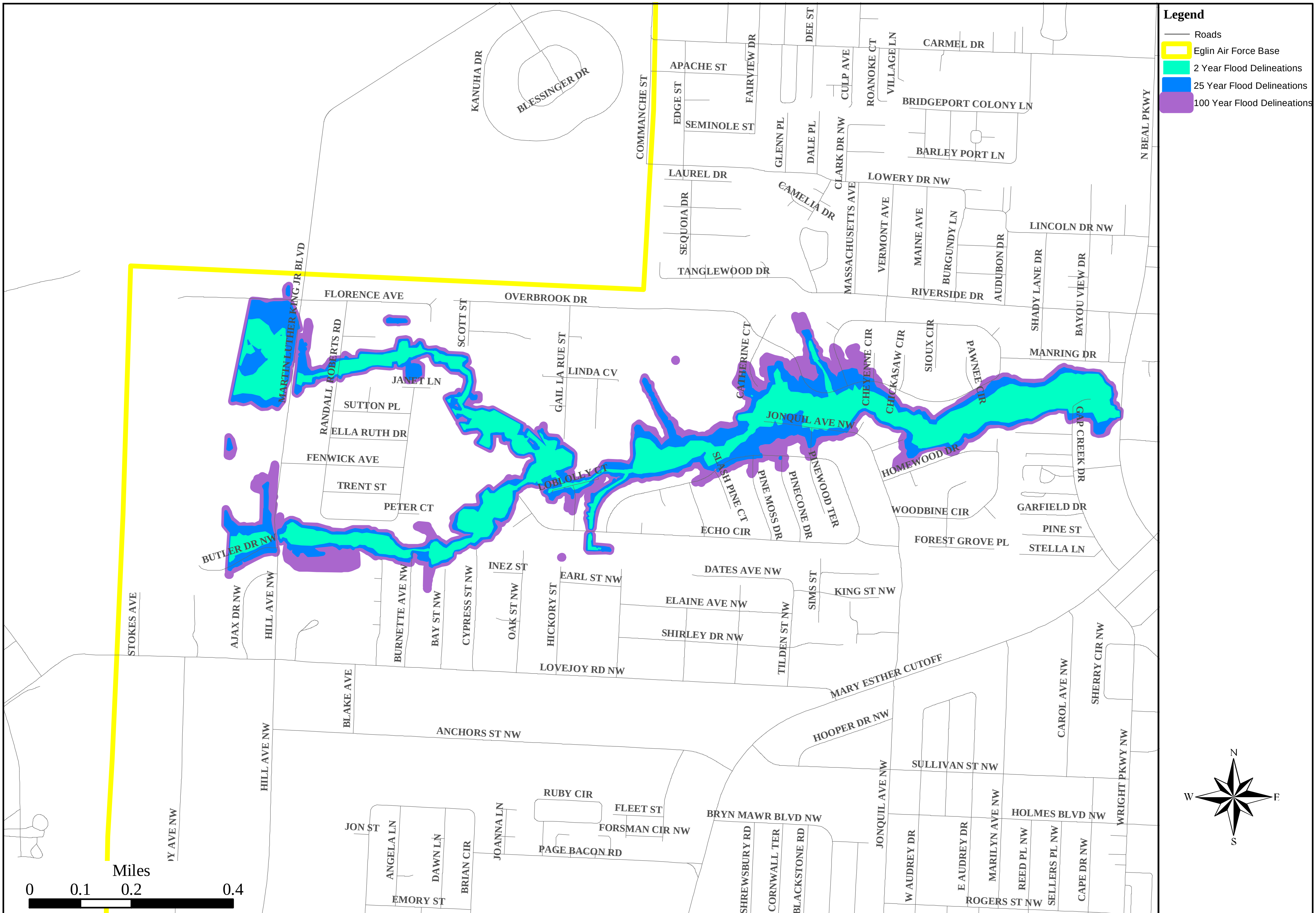
1. Minimum overtopping depth elevation based on topographic survey, unless otherwise noted.
2. Depth of overtopping obtained from HEC-RAS analysis, unless otherwise noted.

Table 6.7
Gap Creek Drainage Basin
Future Hydraulic Capacity of Stream Crossings Summary

Location	Minimum Overtopping Elevation[1]	Depth of Overtopping (ft)[2]				
		2-Year	10-Year	25-Year	50-Year	100-Year
Martin Luther King Boulevard North	32.3	-	-	-	-	-
Martin Luther King Boulevard South	29.4	-	0.3	0.5	0.6	0.6

1. Minimum overtopping depth elevation based on topographic survey, unless otherwise noted.
2. Depth of overtopping obtained from HEC-RAS analysis, unless otherwise noted.

County standards for passing the design flood event without roadway overtopping were used to evaluate each crossing. A summary of the existing and future hydraulic capacity and return period for each of the crossings studied is presented in **Tables 6.8** and **6.9**, respectively.



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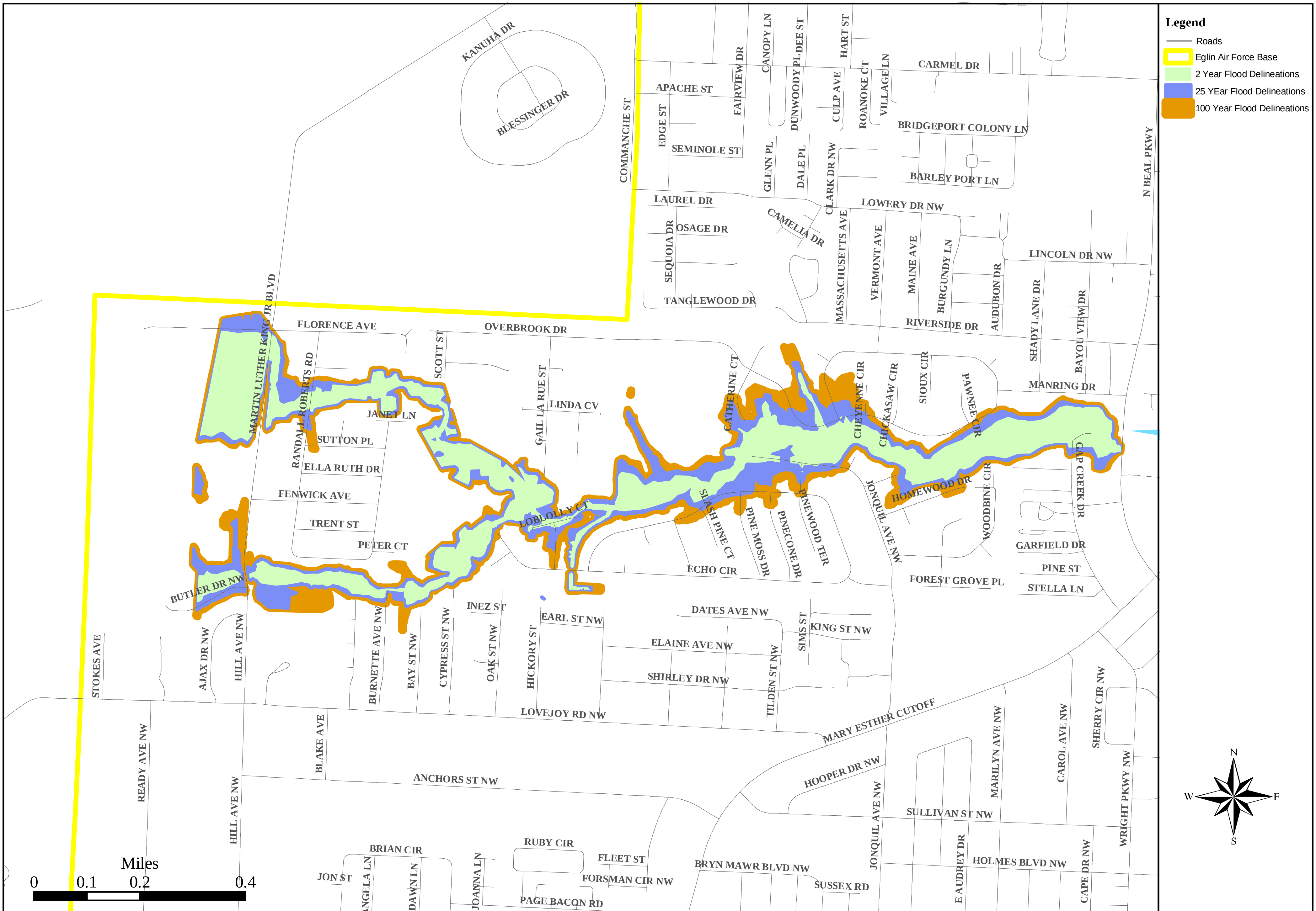
Gap Creek Existing Conditions
2-Year, 25-Year, and 100-Year Flood Delineations

Figure 6-9a



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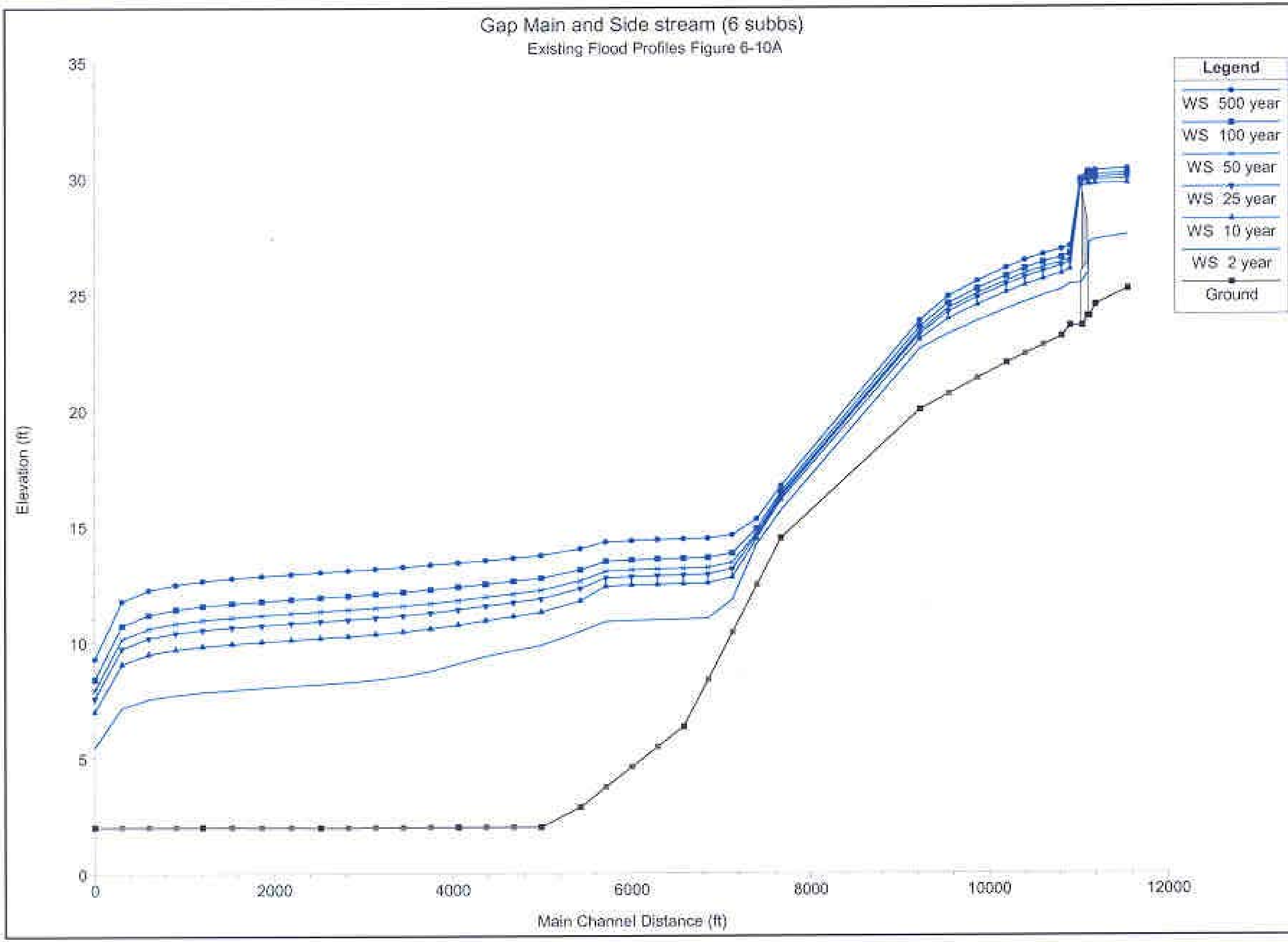
Gap Creek Future Conditions
2-Year, 25-Year, and 100-Year Flood Delineations

Figure 6-9b



Gap Main and Side stream (6 subbs)
Existing Flood Profiles Figure 6-10A
Legend
WS 500 year
WS 100 year
WS 50 year
WS 25 year
WS 10 year
WS 2 year
Ground
Elevation (ft)
Main Channel Distance (ft)
0
5
10
15
20
25
30
35
0
2000
4000
6000
8000
10000
12000

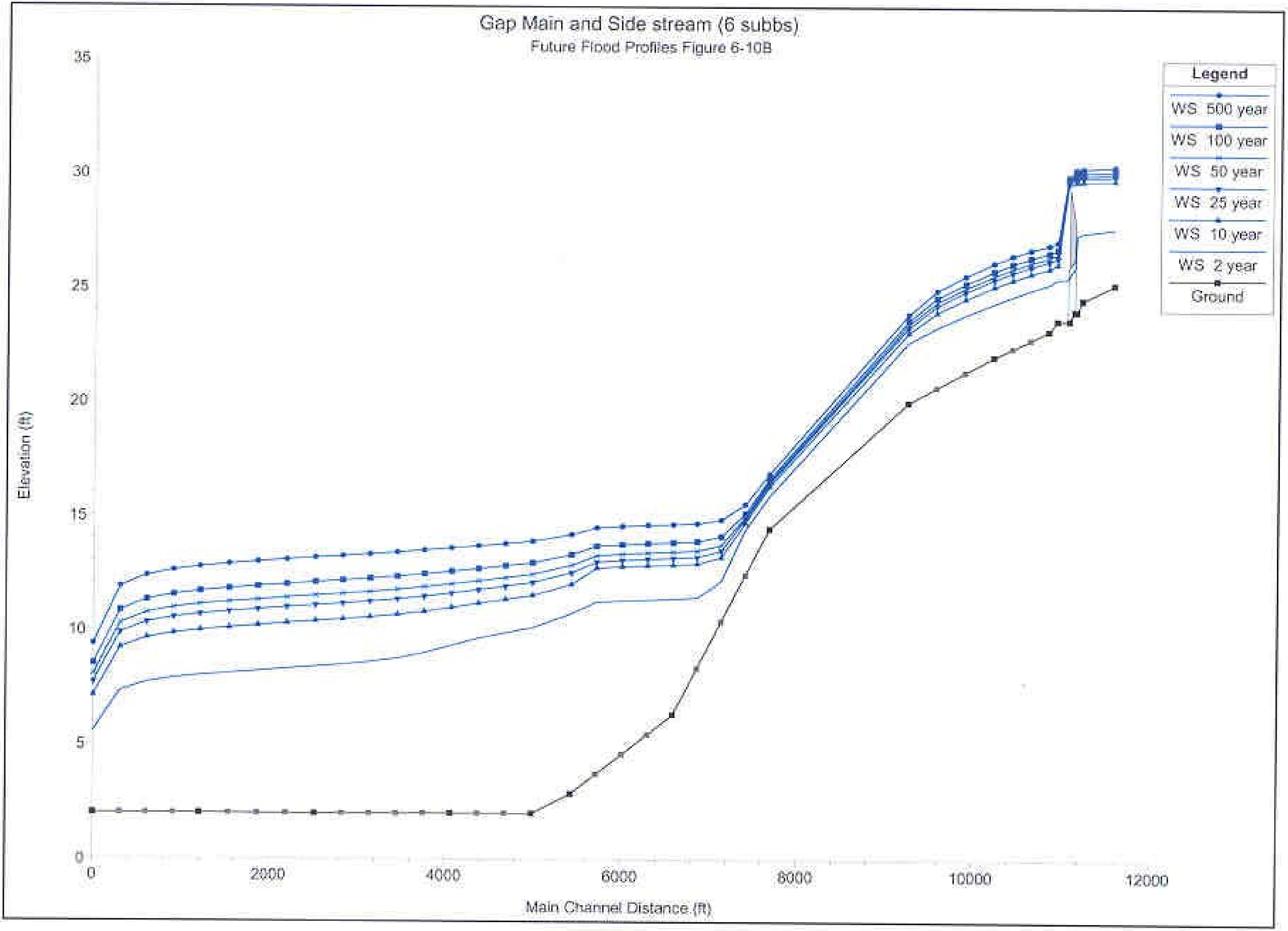

Gap Main and Side stream (6 subbs)
Future Flood Profiles Figure 6-10B
Legend
WS 500 year
WS 100 year
WS 50 year
WS 25 year
WS 10 year
WS 2 year
Ground
Elevation (ft)
Main Channel Distance (ft)
0
5
10
15
20
25
30
35
2000
4000
6000
8000
10000
12000

Table 6.8 Gap Creek Drainage Basin Existing Hydraulic Capacity and Return Period of Stream Crossings Summary				
Location	Existing Structure Type[1]	Roadway Classification	Hydraulic Capacity Return Period	
			Required	Actual
Martin Luther King Boulevard North	CEP	Collector	25-year	100-year
Martin Luther King Boulevard South	CEP	Collector	25-year	<10-year
1. CEP-concrete elliptical pipe				

Table 6.9 Gap Creek Drainage Basin Future Hydraulic Capacity and Return Period of Stream Crossings Summary				
Location	Existing Structure Type[1]	Roadway Classification	Hydraulic Capacity Return Period	
			Required	Actual
Martin Luther King Boulevard North	CEP	Collector	25-year	100-year
Martin Luther King Boulevard South	CEP	Collector	25-year	<10-year
1. CEP-concrete elliptical pipe				

6.3.1.4 Recommendations

Gap Creek is currently surrounded by development in the Lower Basin, limiting the available flood plain. Accordingly, any increase in runoff has the potential to cause increased flooding in the area, and corresponding increased risk to property.

Although the future conditions stormwater model predicts increased runoff to Gap Creek, the model assumes no additional stormwater management facilities associated with new development. In practice, new development will carry a regulatory mandate to limit post-development runoff to the pre-development rate for the regulated storm event. Normally this would protect the creek from future increases in stage resulting from development.

Without careful management, however, two concerns exist relating to future development. First, County standards currently limit the required stormwater analysis to the 25-year 24-hour storm event. This means that Gap Creek could be subjected to more runoff from storms exceeding the 25-year frequency, and from storms of less than 24-hour duration. Second, the development planned for the Upper Basin to date involves introducing two to three feet of fill. As described above, the Upper Basin currently has three areas that operate as natural reservoirs, attenuating runoff by storing it in natural ponding areas. These reservoirs are located in a central wetland, and immediately upstream of each culvert. To the extent development encroaches upon areas currently storing runoff, any fill activities will reduce available storage and increase runoff.

To protect Gap Creek from increases in stage, the following activities are recommended:

- For future development, limit post-development runoff to the pre-development rate for all storms through the 100-year frequency. This will involve amendments to the land development code.

- When reviewing site plans for the Upper Basin, ensure that pre-development discharge computations used to benchmark allowable discharge rates account for attenuation due to natural storage on the site. In other words, account for potential increases to discharge resulting from the filling of natural storage.

- Coordinate with Hurlburt Field regarding capacity restrictions on Gap Creek, and request adequate rate controls in connection with any future Air Force development discharging to the basin.

- Investigate the possibility of a regional or joint-use stormwater management facility west of Martin Luther King Boulevard to reduce the rate of discharge, compensate for lost storage, and improve water quality in the basin.

- Hand clear and maintain the stream channel. Following initial efforts, recruit local residents to keep the channel clear of obstructions.

- Avoid upsizing the culverts under Martin Luther King Boulevard. Although the south culvert is undersized according to the analysis, this culvert serves to stage runoff, and provides attenuation.

- Install a recording gage on the Beal Parkway bridge over Gap Creek that monitors rainfall, stage, and streamflow. This will allow better monitoring of the effects of development in the watershed, and better analysis of existing and future conditions.

6.3.2 Cimarron Outfall

The Cimarron Outfall consists of a well defined ditch system that conveys runoff from Eglin AFB south to Santa Rosa Sound near the Cimarron Subdivision, as shown in **Figure 6-11**. The ditch crosses Quail Hollow Drive, Bob White Drive, US 98, Brookwood Boulevard and Parish Point Road before discharging to a tidal wetland that connects directly to the Sound.

6.3.2.1 Existing Conditions

The Cimarron Outfall features a 377-acre drainage basin. Approximately 150 acres of the basin are residential, and the remaining 277 acres contain wetlands and forested areas on Eglin AFB. A map showing existing land use appears as **Figure 6-12**. The Soil Survey indicates poorly drained soils and a seasonal high groundwater table at or above existing ground across the entire basin.

Flooding of existing development has been reported immediately north of US 98 and in the vicinity of Brookwood Boulevard. In addition, Parish Point Road reportedly overtops frequently, and saturates adjacent residences.

6.3.2.2 Flood Hydrology

The HEC-HMS model was used to compute peak runoff rates for the 2-year, 10-year, 25-year, 50-year and 100-year storm events. Detailed input and output data appears in the Hydrologic and Hydraulic Appendices. Streamflow data was not available for calibration.

Table 6.10 contains a summary of existing peak runoff rates for selected storm events at each structure. Because the area is fully developed, future development conditions were not considered.

Legend

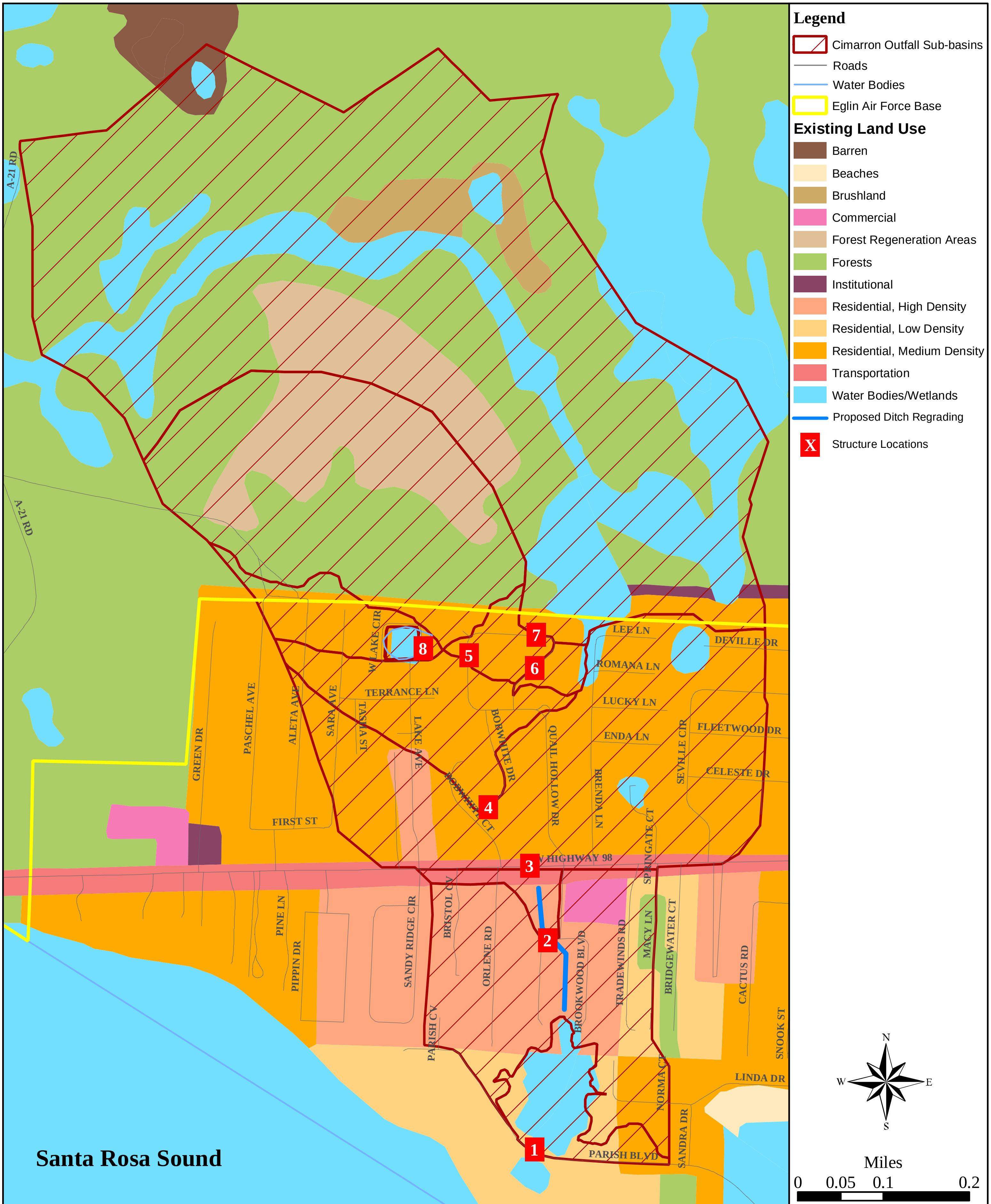
- Cimarron Outfall Sub-basins
- Highways
- Roads
- Eglin Air Force Base
- Proposed Ditch Regrading
- X Structure Locations

N
W E
S

Master Stormwater Management Plan

Cimarron Outfall

Figure 6-11



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Cimarron Outfall Existing Land Use

Figure 6-12

Table 6.10 Cimarron Outfall Peak Runoff Summary for Existing Drainage System Conditions								
Structure Id. No. [1]	HEC-HMS Node No.	Location	Drainage Area (sq. mi)	Peak Runoff Rate (cfs) [2,3]				
				2-Year	10-Year	25-Year	50-Year	100-Year
1	J1	Parish Point Road	0.587	218.0	418.0	523.8	612.4	733.7
2	J2	Brookwood Boulevard	0.513	172.5	347.1	426.6	479.4	559.3
3	J3	US 98	0.507	169.6	342.3	420.7	472.8	555.7
4	J4	Bob White Drive	0.405	99.6	204.0	252.3	284.7	334.0
5	J5	Quail Hollow Drive	0.274	60.6	137.1	170.8	193.3	226.9
6	J6	Quail Hollow Drive	0.003	6.5	11.3	13.3	14.7	16.7
7	J7	Quail Hollow Drive	0.264	59.8	135.6	169.0	191.4	224.8
8	R2	Lake Perry	0.014	12.6	22.3	26.5	29.4	33.7

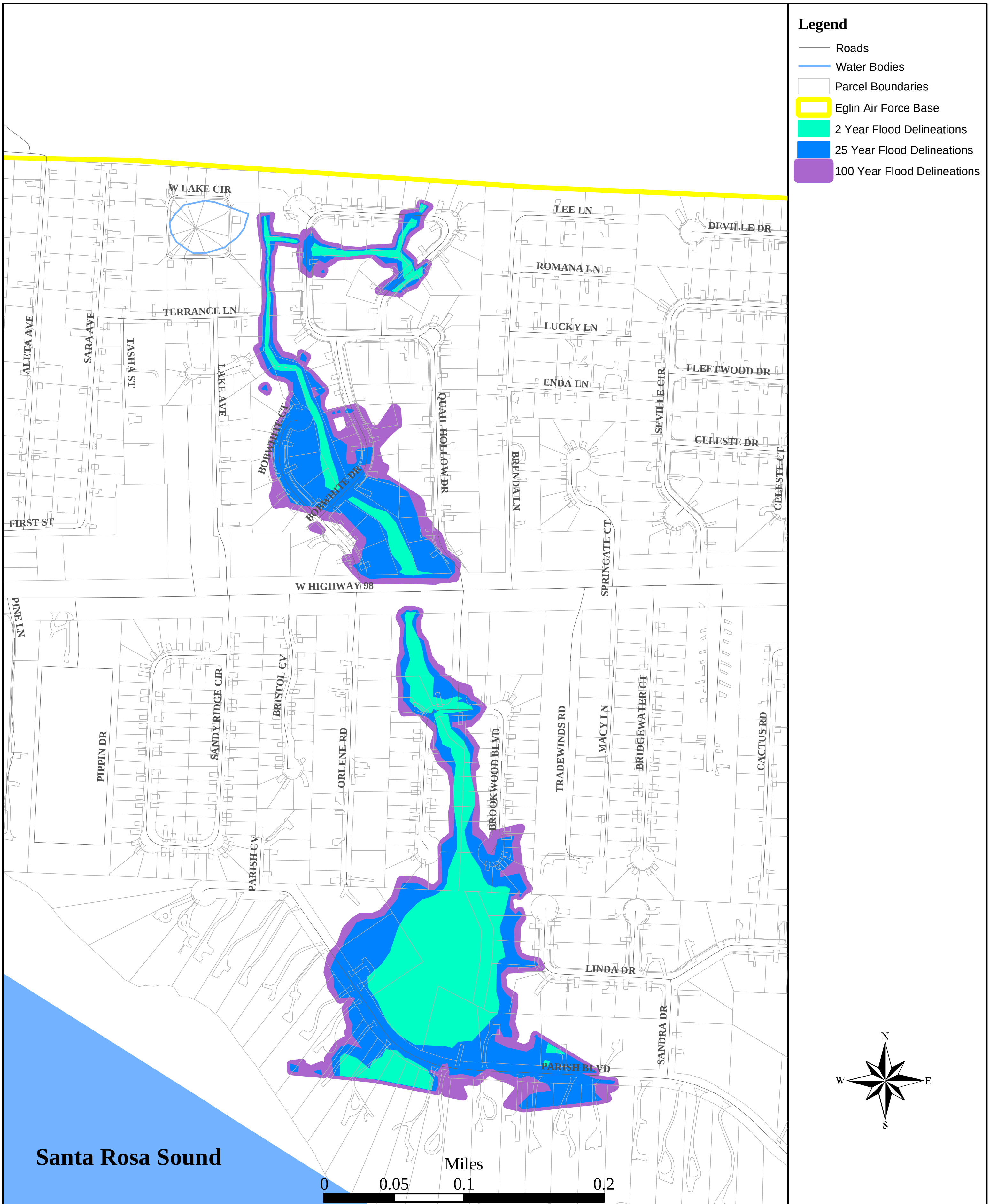
1. See Figure 6-11 for location of structure identification number.
2. Peak runoff rates based on existing land use conditions.
3. Peak discharges reported are outflows from the specified nodes.

6.3.2.3 Flood Hydraulics

6.3.2.3.1 Hydraulic Analysis Of Existing System

HEC-RAS was utilized to determine the current performance of the ditch and associated drainage structures. **Figure 6-13a** shows flood delineations for the 2-year, 25-year and 100-year storm events, and **Figure 6-14a** shows flood profiles for the 2-year, 10-year, 25-year, 50-year and 100-year storm events. As shown in these figures, significant encroachment of the 25-year flood boundary onto surrounding development is expected along Bob White Court, Quail Hollow Drive, Brookwood Boulevard, and Parish Point Road.

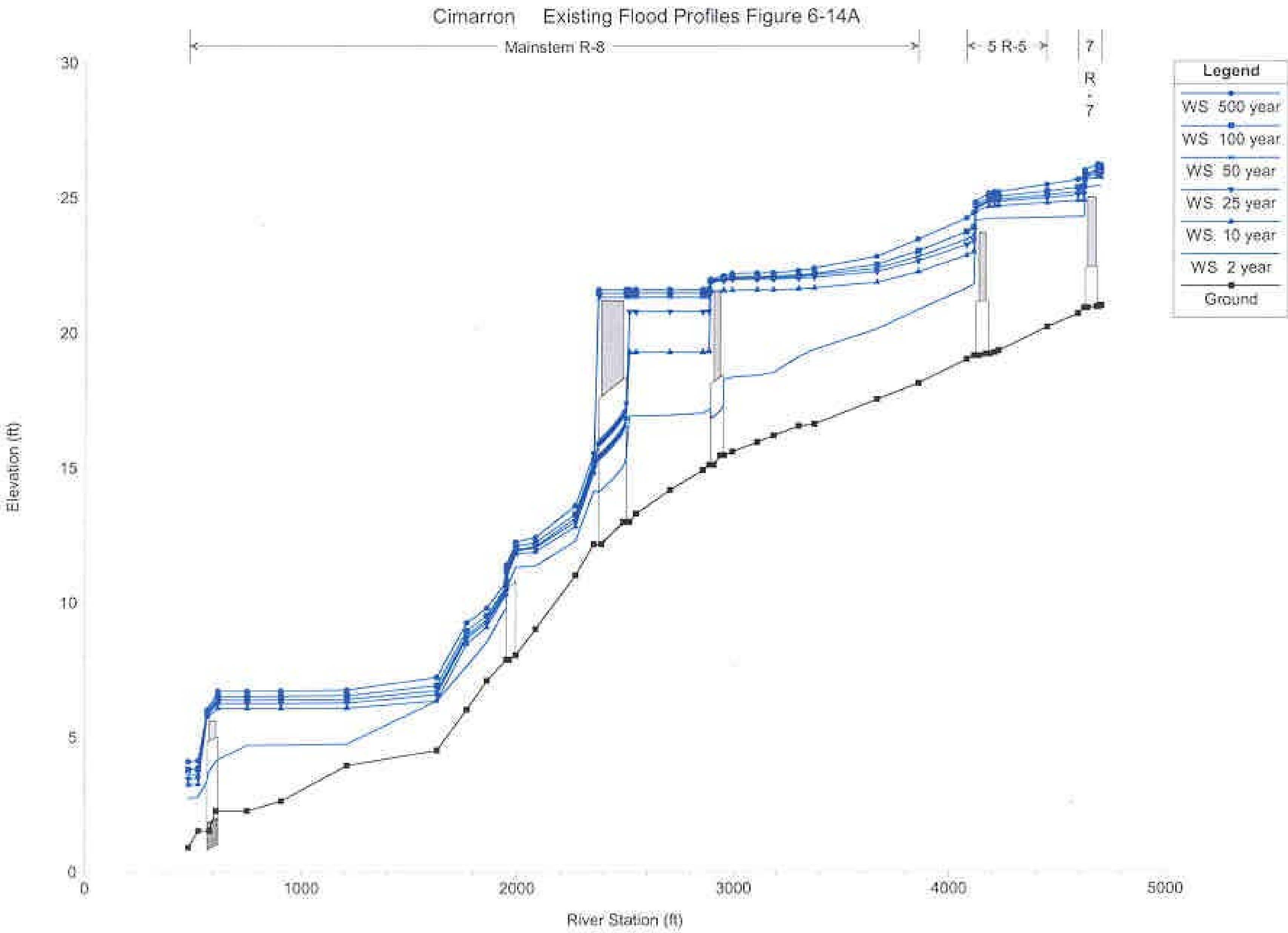
A summary of the existing hydraulic capacity of each of the culvert crossings studied, including the frequency and depth of overtopping, is presented in **Table 6.11** for the 2-year, 10-year, 25-year, 50-year and 100-year storm events. Applying County performance standards, only Lake Perry meets the minimum established criteria for overtopping.



Master Stormwater Management Plan

Cimarron Outfall Existing Conditions
2-Year, 25-Year, and 100-Year Flood Delineations

Figure 6-13a



Cimarron Existing Flood Profiles Figure 6-14A
Mainstem R-8
5 R-5
7
R
-
7
Legend
WS 500 year
WS 100 year
WS 50 year
WS 25 year
WS 10 year
WS 2 year
Ground
Elevation (ft)
River Station (ft)
0
5
10
15
20
25
30
0
1000
2000
3000
4000
5000

Table 6.11 Cimarron Outfall Existing Hydraulic Capacity of Stream Crossings Summary							
Structure Id. No.[1]	**Location**	**Minimum Overtopping Elevation[2]**	**Depth of Overtopping (ft)[3]**				
			2-Year	**10-Year**	**25-Year**	**50-Year**	**100-Year**
1	Parish Point Road	5.61	-	0.4	0.6	0.8	0.9
2	Brookwood Boulevard	10.86	0.4	0.9	1.1	1.1	1.2
3	US 98	21.22	-	-	-	0.1	0.2
4	Bob White Drive	21.56	-	-	0.4	0.4	0.5
5	Quail Hollow Drive	23.70	0.5	1.0	1.2	1.2	1.3
6	Quail Hollow Drive	23.88	0.4	1.0	1.2	1.3	1.4
7	Quail Hollow Drive	25.02	0.4	0.7	0.8	0.9	1.0
8	Lake Perry	25.00	-	-		-	-

1. See Figure 6-11 for location of structure identification number.
2. Minimum overtopping depth elevation based on topographic survey, unless otherwise noted.
3. Depth of overtopping based on HEC-RAS analysis.

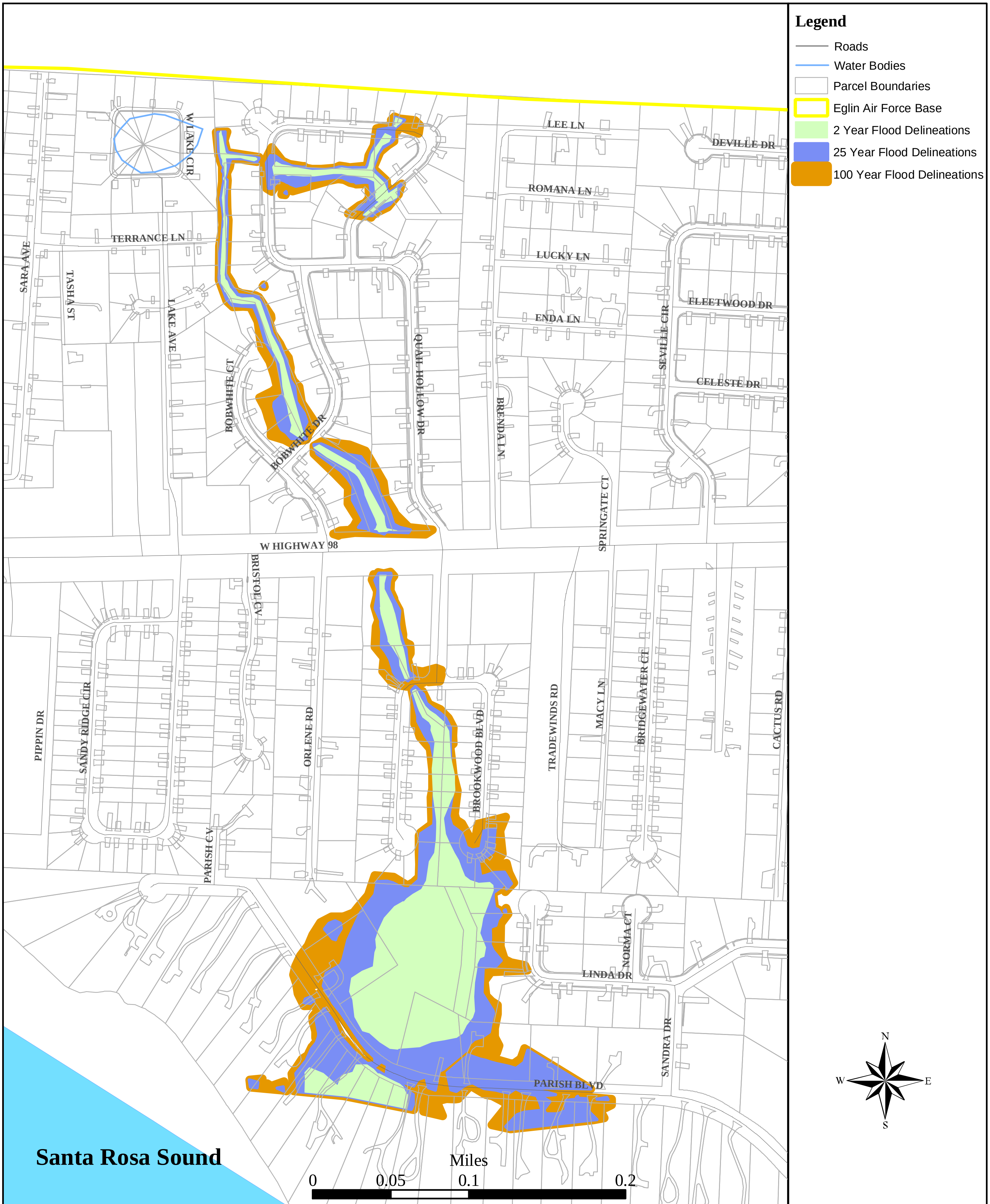
6.3.2.3.2 Hydraulic Analysis Of Proposed Improvements

HEC-RAS was utilized to evaluate potential improvements to the Cimarron Outfall, including the following:

- Replace the 5 - 48” CMP at Parish Point Road with 5 - 60” x 38” RCP.
- Replace the 3 - 48” x 33” CMP at Brookwood Boulevard with 2 – 8’W x 4’H CBC, and lower the flow line of the culvert by 2.2 ft.
- Add an additional 5.5’ x 5.5’ barrel to the existing 8’W x 5.5’H box culvert at US 98.
- Replace the 3 – 36” RCP at Bob White Drive with 2 – 6’W x 4’H CBC.
- Regrade 640 LF of ditch between US 98 and a point 341 feet downstream of the culvert at Brookwood Boulevard, lowering the ditch an average of 1.5 ft.

Applying these improvements, **Figure 6-13b** shows flood delineations for the 2-year, 25-year, and 100-year storm events, and **Figure 6-14b** shows flood profiles for the 2-year, 10-year, 25-year, 50-year and 100-year storm events. As shown in these figures the proposed improvements are expected to reduce flood elevations in the area.

A summary of the hydraulic capacity of each of the proposed structures studied appears in **Table 6.12** for the 2-year, 10-year, 25-year, 50-year and 100-year storm events. As indicated in the



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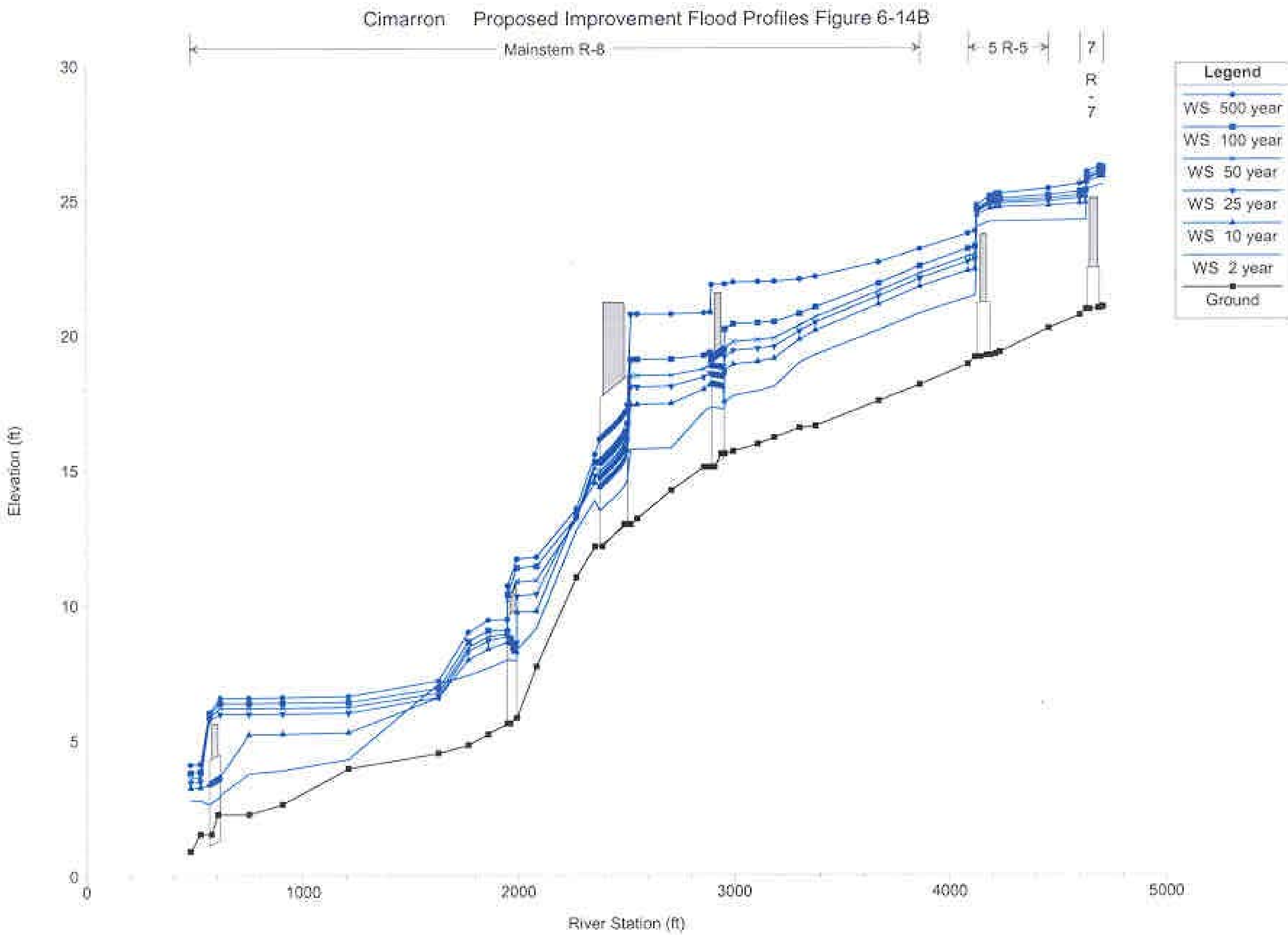
Cimarron Outfall Future Conditions
2-Year, 25-Year, and 100-Year Flood Delineations

Figure 6-13b



Cimarron Proposed Improvement Flood Profiles Figure 6-14B
Mainstem R-8
5 R-5
7
R
-
7
Legend
WS 500 year
WS 100 year
WS 50 year
WS 25 year
WS 10 year
WS 2 year
Ground
Elevation (ft)
River Station (ft)
0
5
10
15
20
25
30
1000
2000
3000
4000
5000

table, all of the replaced structures are anticipated to meet County criteria for roadway overtopping.

Table 6.12
Cimarron Outfall
Proposed Hydraulic Capacity of Stream Crossings Summary

Structure Id. No.[1]	Location	Minimum Overtopping Elevation[2]	Depth of Overtopping (ft)[3]				
			2-Year	10-Year	25-Year	50-Year	100-Year
1	Parish Point Road	5.61	-	-	0.4	0.6	0.8
2	Brookwood Boulevard	10.86	-	-	-		0.5
3	US 98	21.22	-	-	-	-	-
4	Bob White Drive	21.56	-	-	-	-	-
5	Quail Hollow Drive	23.70	0.5	1.0	1.1	1.2	1.2
6	Quail Hollow Drive	23.88	0.3	0.90	1.1	1.2	1.4
7	Quail Hollow Drive	25.02	0.5	0.8	0.9	0.9	1.0
8	Lake Perry	25.00	-	-		-	-

1. See Figure 6-11 for location of structure identification number.
2. Minimum overtopping depth elevation based on topographic survey, unless otherwise noted.
3. Depth of overtopping based on HEC-RAS analysis.

At the Parish Drive crossing, excessive trash was observed partially blocking flow through the culverts. Trash collectors exist at the culverts, but have been constructed across the pipe entrances. To improve the performance of the collectors, a redesign is recommended to move the collectors away from the inlet a distance of 5 – 10 feet. This will present a larger area of collector to the flow, and introduce a collection surface parallel to the flow, which will be less likely to clog.

Two additional proposed solutions were analyzed, but judged to provide insufficient benefit to justify the cost. The additional considered solutions follow:

- Provide Additional Storage On Eglin AFB – The construction of a stormwater management facility was considered on Eglin AFB property to attenuate peak flows leaving the reservation. However, the facility considered would not attenuate flows sufficiently to provide meaningful improvements to the basin, and would benefit only properties near the base boundary.
- Upgrade All Culverts – Upgrading all structures, including the three structures under Quail Hollow Drive, was considered. However, any additional benefits would require simultaneous ditch improvements and have potential right-of-way impacts. In this

regard, upgrading all structures is not recommended at this time, because flooding in the upper basin appears contained on lawns and the public has not reported excessive stages.

Note that the ditch elevations used for hydraulic routing were initially taken from the County TINs. However, ditch bottom elevations indicated by the TINs did not agree with flow line data determined from County survey efforts. All conflicts were resolved in favor of ground survey, and ditch bottom elevations determined from the County TINs were interpolated and adjusted were necessary.

6.3.2.4 Summary

A summary of all proposed improvements appears in **Table 6.13** below.

Table 6.13
Cimarron Outfall
Proposed Improvement Summary

Structure Id. No.[1]	Location	Existing Structure[2]	Proposed Structure
1	Parish Point Road	5-48" CMP	5-60"x 38" RCP
2	Brookwood Boulevard	3-48"x 33" CMP	2-8'x 4' CBC
3	US 98	1-8'x 5.5' CBC	Existing structure to remain & add 1-5.5'x 5.5' CBC
4	Bob White Drive	3-36"-RCP	2-4'x 6' CBC
5	Quail Hollow Drive	1-24"-RCP	Existing structure to remain
6	Quail Hollow Drive	1-18"-RCP	Existing structure to remain
7	Quail Hollow Drive	1-18"-RCP	Existing structure to remain
8	Lake Perry	30" -Riser	Existing structure to remain
NA	South of US 98	Existing Ditch	Regrade Ditch

1. See Figure 6-11 for location of structure identification number.
2. CMP – corrugated metal pipe, CBC – concrete box culvert, RCP – reinforced concrete pipe.

6.3.3 Commons Drive

The Commons Drive Ditch is located in the southeast portion of Okaloosa County, as shown in **Figure 6-15**. The ditch serves to convey runoff from surrounding development to an FDOT drainage easement that discharges to Choctawhatchee.

6.3.3.1 Existing Conditions

The Commons Drive ditch features a drainage basin of approximately 0.4 square miles. Existing land use is a balance of commercial development, residential development, and undeveloped forest, with commercial development concentrated along US 98 and Commons Drive. A map showing existing land use appears as **Figure 6-16**. In the future, it is anticipated that most of the existing vacant land, all of which has frontage on US 98 or Commons Drive, will be replaced with commercial development. A map showing future land use appears as **Figure 6-17.** Note that the future land use shown in Figure 6-17 represents a modified version of the future land use provided by the County. Modifications were made to add commercial land use in areas that are known by County personnel to be targeted for development.

6.3.3.2 Flood Hydrology

The Commons Drive basin was divided into six sub-basins for analysis. The upper two sub-basins discharge to the ditch headwaters and drain residential and mixed land uses, respectively. Both of the upper two basins contain stromwater ponds. The middle three sub-basins discharge directly to the ditch and contain commercial development including Emerald Coast Shopping Center and Wal-Mart. The middle sub-basins feature two stormwater ponds, located at Wal-Mart and the property immediately west of Wal-Mart. The lower sub-basin, located between Henderson Beach Road, Commons Drive, and Tropic Trail, is partially developed and also discharges directly to the ditch. The lower sub-basin contains a single in-line stormwater pond.

For both existing and future land use conditions, stormwater runoff was routed through the six identified stormwater ponds, which were modeled as reservoirs in HEC-HMS. The required elevation/storage/flow relationship was estimated using plans provided by the County where available, or using provided spatial data.

Note that additional stormwater ponds were not assumed for future land use conditions in order to show the effects of uncontrolled development. In practice, land development regulations will limit the post-development rate of runoff to the pre-development rate, resulting in flood characteristics similar to that predicted for existing conditions. For this reason, the existing conditions model was used to evaluate proposed improvements, applying a rate-limiting assumption.

Master Stormwater Management Plan

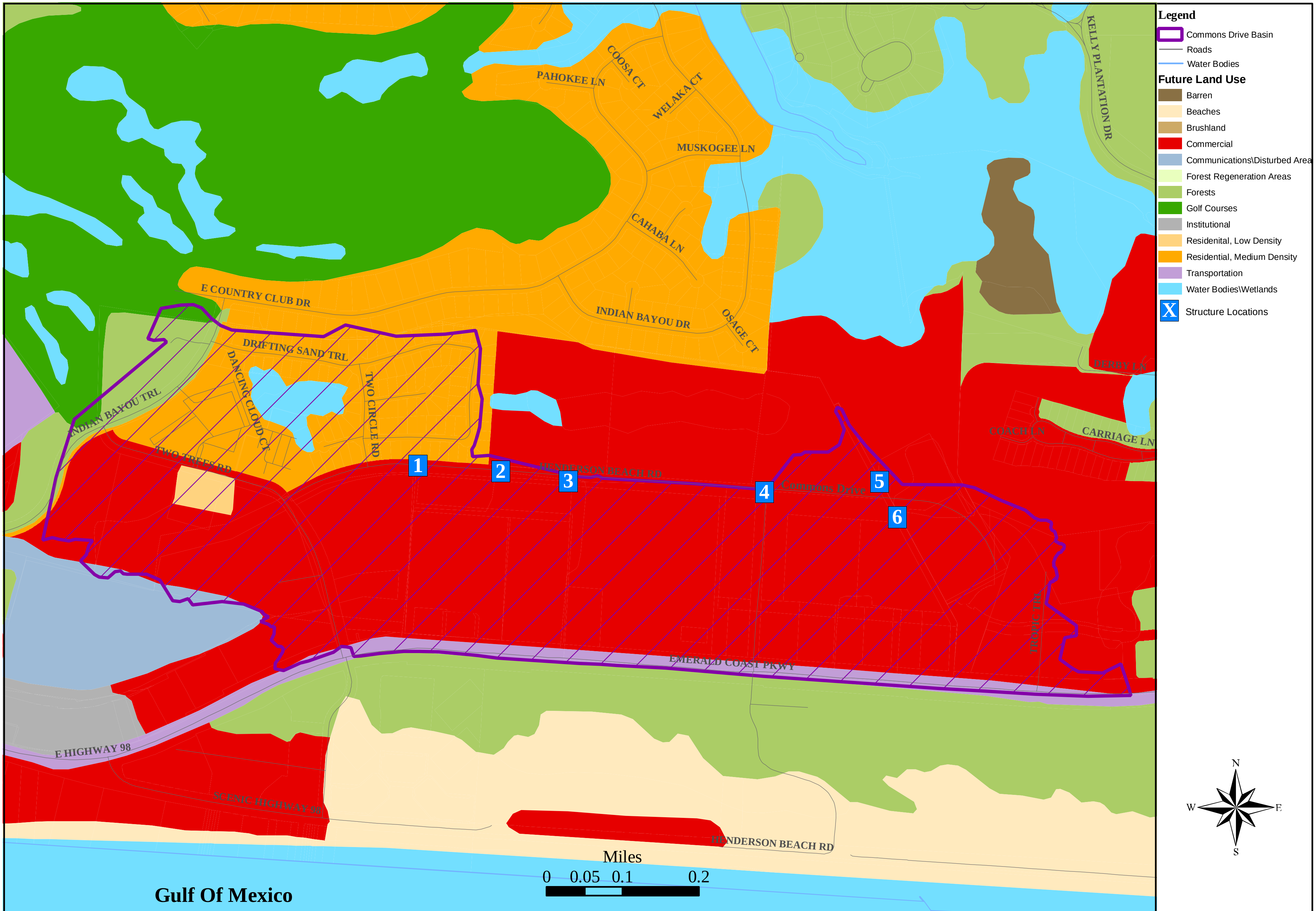
Commons Drive

Figure 6-15

Master Stormwater Management Plan

Commons Drive
Existing Land Use

Figure 6-16



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Commons Drive Future Land Use

Figure 6-17

HDR

The SCS Transform Method was used to generate the hydrograph and peak runoff rates for the 2-, 10-, 25-, 50-, and 100-year return period storm events. Detailed input and output data appear in the Hydrologic and Hydraulic Appendix. Streamflow data was not available for calibration.

Tables 6.14 and **6.15** contain a summary of existing and future peak runoff rates for selected storm events at critical locations.

Table 6.14
Commons Drive Ditch
Peak Runoff Summary for Existing Drainage System Conditions

HEC-HMS Node No.	Location	Drainage Area (sq. mi)	Peak Runoff Rate (cfs) [1,2]				
			2-Year	10-Year	25-Year	50-Year	100-Year
J1	Beginning of the Ditch	0.16	36	96	120	137	164
J4	D/S of Emerald Coast Shopping Center	0.23	49	144	187	215	259
J2	D/S of Wal-Mart outfall, U/S of Commons Drive 3x24” culvert	0.29	71	207	253	283	334
J3	U/S of 2x36” culvert	0.40	70	187	233	264	310

1. Peak runoff rates based on existing land use condition and simulation of a 24-hour storm event.
2. Peak discharges reported are outflows from the specified nodes.

Table 6.15
Commons Drive Ditch
Peak Runoff Summary for Future Drainage System Conditions

HEC-HMS Node No.	Location	Drainage Area (sq. mi)	Peak Runoff Rate (cfs) [1,2]				
			2-Year	10-Year	25-Year	50-Year	100-Year
J1	Beginning of the Ditch	0.16	64	129	181	238	315
J4	D/S of Emerald Coast Shopping Center	0.23	116	244	293	354	455
J2	D/S of Wal-Mart outfall, U/S of Commons Drive 3x24” culvert	0.29	159	300	356	413	509
J3	U/S of 2x36” culvert	0.40	190	349	438	500	593

1. Peak runoff rates based on future land use condition and simulation of a 24-hour storm event.
2. Peak discharges reported are outflows from the specified nodes.

6.3.3.3 Flood Hydraulics

6.3.3.3.1 Hydraulic Analysis of Existing System

HEC-RAS was utilized to determine the performance of the ditch and associated drainage structures. **Figures 6-18a** and **6-18b** show flood delineations for the 2-year, 25-year and 100-year storm events, for existing conditions and proposed improvements, respectively. **Figures 6-19a** and **6-19b** show flood profiles for the 2-, 10-, 25-, 50-, and 100-year storm events, for existing conditions and proposed improvements, respectively.

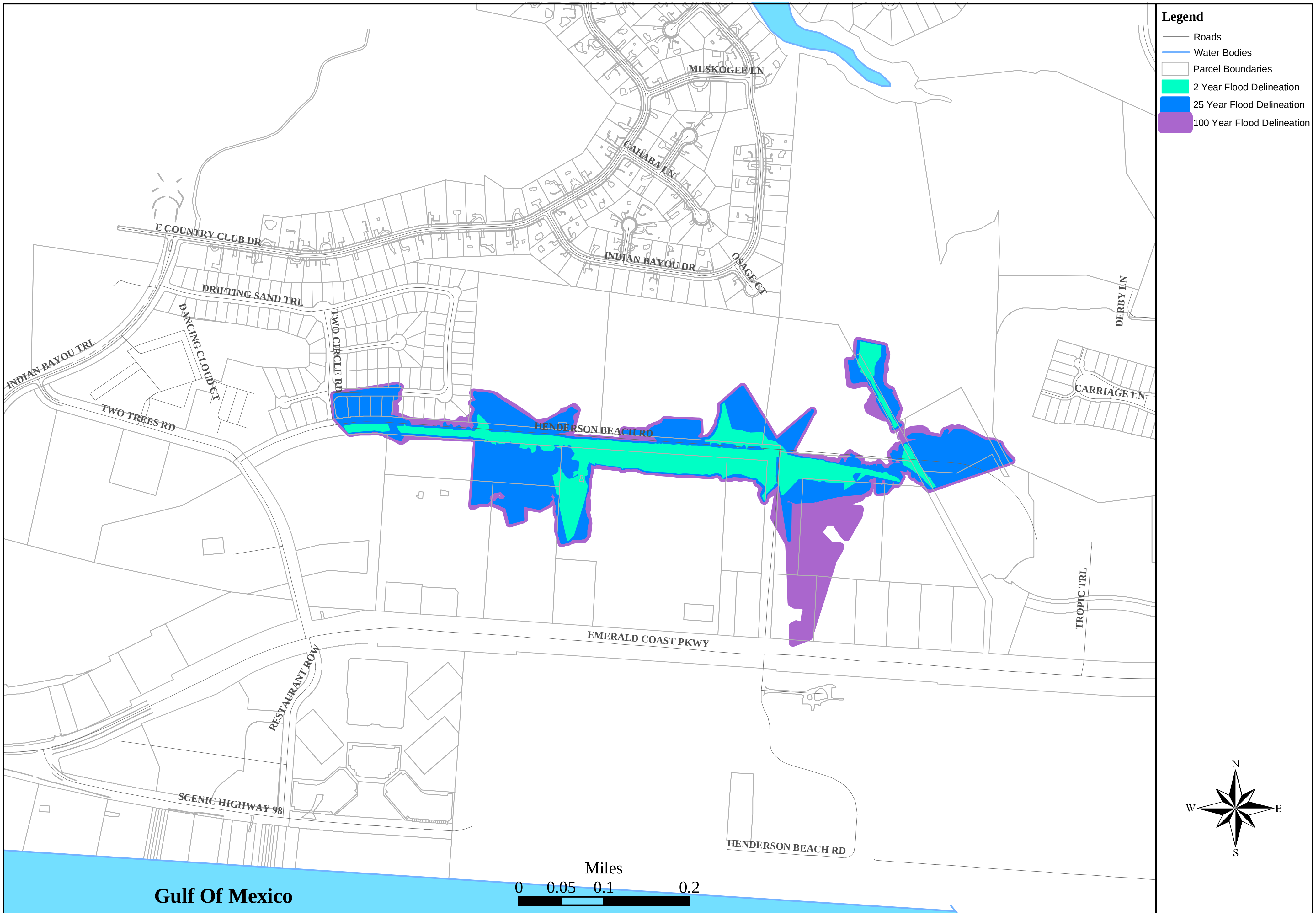
A summary of the existing hydraulic capacity of each of the culvert crossings studied is presented in **Table 6.16** for the 2-year, 10-year, 25-year, 50-year and 100-year storm events.

Table 6.16
Commons Drive Ditch
Existing Hydraulic Capacity of Culvert Crossings Summary

Structure Id. No.[1]	Location	Minimum Overtopping Elevation[2]	Depth of Overtopping (ft)[3]				
			2-Year	10-Year	25-Year	50-Year	100-Year
1	Driveway into shopping center	18.0	-	0.5	0.5	0.5	0.6
2	Driveway into shopping center	16.6	0.3	0.8	0.9	1.0	1.1
3	Driveway into shopping center	16.8	-	0.2	0.3	0.3	0.3
4	Commons Drive West	14.6	0.4	0.5	0.7	0.9	1.0
5	Commons Drive East	14.0	0.4	0.6	1.1	1.2	1.3
6	Fine Arts Council Road	14.2	-	0.2	0.2	0.2	0.3

1. See Figure 6-15 for location of structure identification number.
2. Minimum overtopping depth elevation based on topographic survey, unless otherwise noted.
3. Depth of overtopping based on HEC-RAS analysis.

County standards for passing the design flood event without roadway overtopping were used to evaluate each crossing. Applying existing land use and existing conditions, a summary of the hydraulic capacity and return period for each of the crossings studied is presented in **Table 6.17**.



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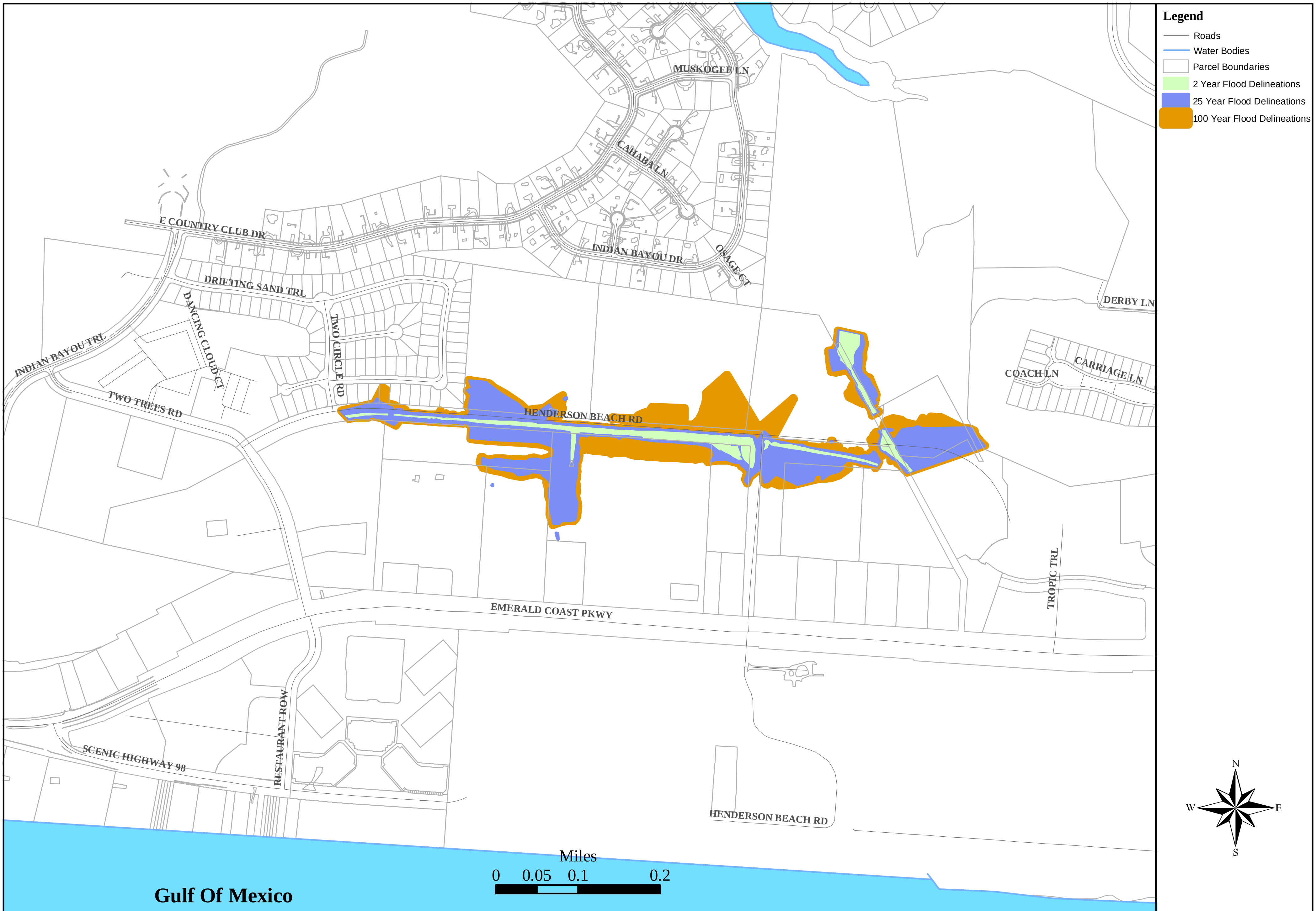
Commons Drive Existing Conditions
2-Year, 25-Year, and 100-Year Flood Delineations

Figure 6-18a



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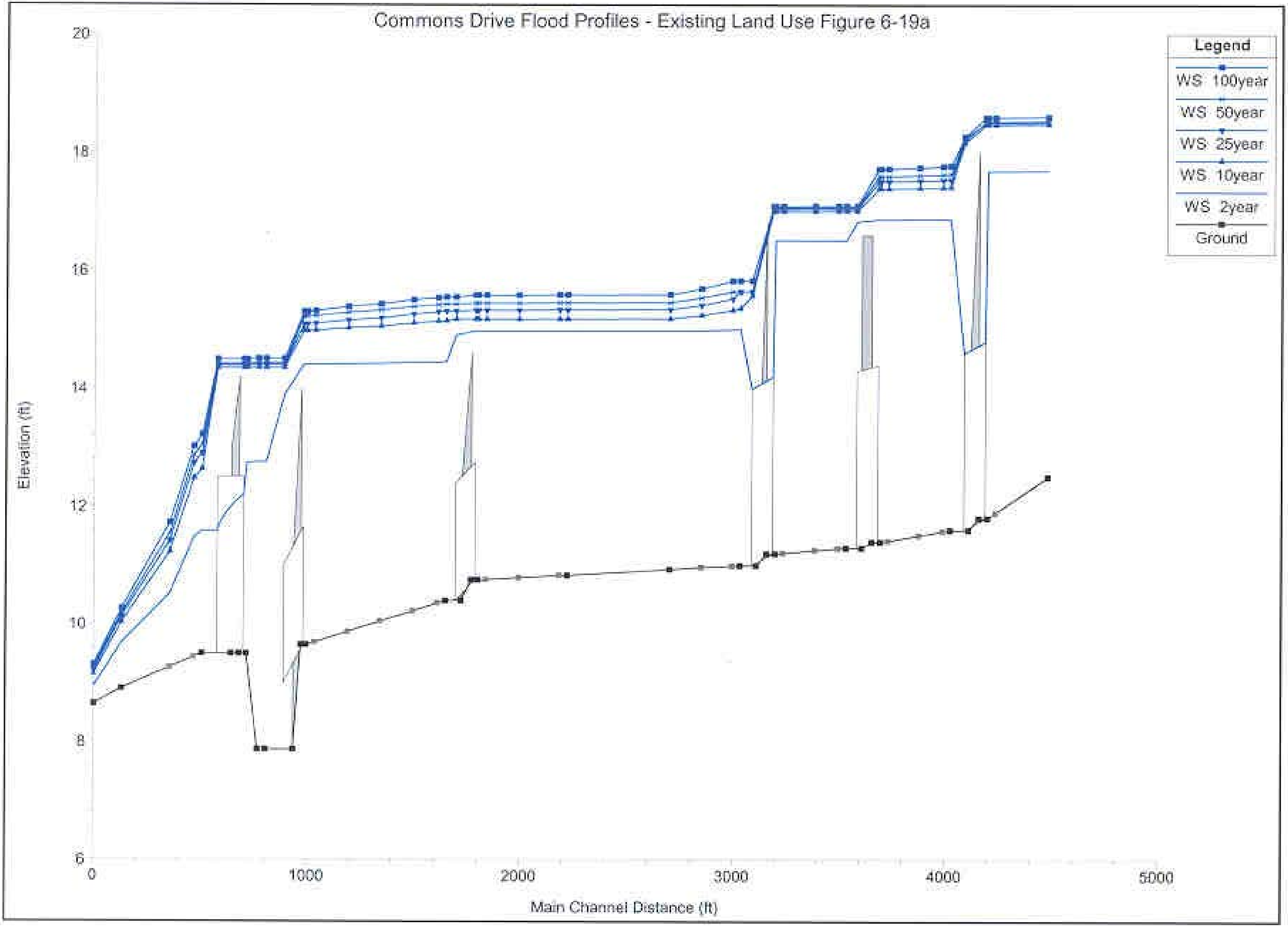
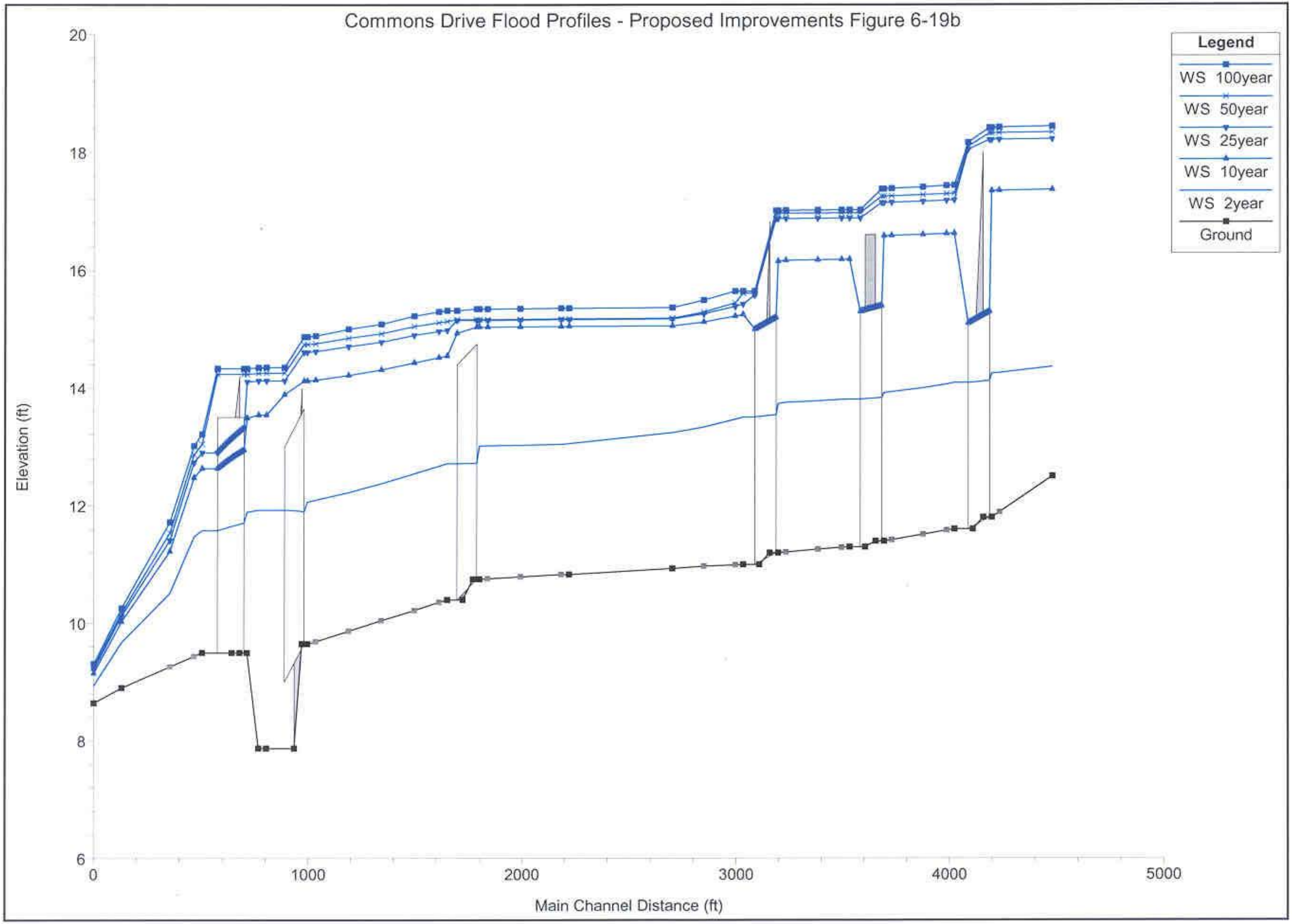
Commons Drive Proposed Improvements
2-Year, 25-Year, and 100-Year Flood Delineations

Figure 6-18b



Commons Drive Flood Profiles - Existing Land Use Figure 6-19a
Legend
WS 100year
WS 50year
WS 25year
WS 10year
WS 2year
Ground
Elevation (ft)
20
18
16
14
12
10
8
6
0
1000
2000
3000
4000
5000
Main Channel Distance (ft)



Commons Drive Flood Profiles - Proposed Improvements Figure 6-19b
Legend
WS 100year
WS 50year
WS 25year
WS 10year
WS 2year
Ground
Elevation (ft)
20
18
16
14
12
10
8
6
0
1000
2000
3000
4000
5000
Main Channel Distance (ft)

Table 6.17 Commons Drive Ditch Existing Hydraulic Capacity and Return Period of Culvert Crossings Summary					
Structure Id. No. [1]	**Location**	**Existing Structure Type**[2]	**Roadway Classification**	**Hydraulic Capacity Return Period**	
				Required	**Actual**
1	Driveway into shopping center	36" RCP	Driveway	10-year	2-year
2	Driveway into shopping center	36" RCP	Driveway	10-year	<2-year
3	Driveway into shopping center	36" RCP	Driveway	10-year	2-year
4	Commons Drive West	3-24"RCP	Local	10-year	<2-year
5	Commons Drive East	2-24"RCP	Local	10-year	<2-year
6	Fine Arts Council Road	2-36" RCP	Local	10-year	2-year

1. See Figure 6-15 for location of structure identification number.
2. RCP – reinforced concrete pipe

6.3.3.3.2 Hydraulic Analysis of Proposed Improvements

HEC-RAS was utilized to evaluate potential improvements to the Commons Drive ditch, including the following:

- Replace structure number 1 (36-inch pipe) with two 42-inch pipes
- Replace structure number 2 (36-inch pipe) with two 48-inch pipes
- Replace structure number 3 (36-inch pipe) with two 48-inch pipes
- Replace structure number 4 (three 24-inch pipes) with two 48-inch pipes
- Replace structure number 5 (two 24-inch pipes) with two 48-inch pipes
- Replace structure number 6 (two 36-inch pipes) with two 48-inch pipes

A summary of the hydraulic capacity and return period of each of the proposed structures studied appears in **Tables 6.18a and 6.18b** for the 2-, 10-, 25-, 50-, and 100-year storm events.

Table 6.18a
Commons Drive Ditch
Proposed Improvements Hydraulic Capacity of Culvert Crossings Summary

Structure Id. No.[1]	Location	Minimum Overtopping Elevation[2]	Depth of Overtopping (ft)[3]				
			2-Year	10-Year	25-Year	50-Year	100-Year
1	Driveway into shopping center	18.0	-	-	0.2	0.3	0.4
2	Driveway into shopping center	16.6	-	-	0.5	0.8	1.0
3	Driveway into shopping center	16.8	-	-	0.1	0.2	0.2
4	Commons Drive West	14.6	-	0.4	0.6	0.6	0.8
5	Commons Drive East	14.0	-	0.1	0.6	0.7	0.9
6	Fine Arts Council Road	14.2	-	-	-	0.1	0.1

1. See Figure 6-15 for location of structure identification number.
2. Minimum overtopping depth elevation based on topographic survey, unless otherwise noted.
3. Depth of overtopping based on HEC-RAS analysis.

Table 6.18b
Commons Drive Ditch
Proposed Improvements Hydraulic Capacity and Return Period of Culvert Crossings Summary

Structure Id. No.[1]	Location	Proposed Structure Type[2]	Roadway Classification	Hydraulic Capacity Return Period	
				Required	Actual
1	Driveway into shopping center	2-42” RCP	Driveway	10-year	10-year
2	Driveway into shopping center	2-48” RCP	Driveway	10-year	10-year
3	Driveway into shopping center	2-48” RCP	Driveway	10-year	10-year
4	Commons Drive West	3-48” RCP	Local	10-year	2-year
5	Commons Drive East	3-48” RCP	Local	10-year	2-year
6	Fine Arts Council Road	3-48” RCP	Local	10-year	25-year

1. See Figure 6-15 for location of structure identification number.
2. RCP – reinforced concrete pipe.

6.3.3.4 Recommendations

Future development, if uncontrolled, could lead to capacity issues associated with the Commons Drive ditch. To minimize capacity issues, following measures are recommended.

- Replace structure number 1 (36-inch pipe) with two 42-inch pipes
- Replace structure number 2 & 3 (36-inch pipe) with two 48-inch pipes
- Replace structure number 4 (three 24-inch pipes) with two 48-inch pipes
- Replace structure number 5 (two 24-inch pipes) with two 48-inch pipes
- Replace structure number 6 (two 36-inch pipes) with two 48-inch pipes
- Increase the ditch capacity, if possible, by providing a wider bottom and steeper slope
- Ensure future development limits post-development runoff to pre-development rates
- Minimize runoff from newly created impervious areas by rain gardens or buffer strips

6.3.4 Lake Blake

Figure 6-21
Lake Blake Photograph

Lake Blake is located east of SR 189 (Beal Parkway) off of Lewis Street/Mayflower Avenue, as shown in **Figure 6-20**. According to the Soil Survey, Lake Blake lies within Chipley and Hurricane soil units, and the basin draining to the lake contains Foxwood and Lakeland sand. The Chipley and Hurricane soil units are characterized as having somewhat poorly drained soils in areas bordering drainageways. Foxworth sand is a moderately well drained soil, and Lakeland sand is considered an excessively drained soil. While Lake Blake currently functions as a drainageway, the physical components of the soils, the unnatural shape of the lake, and the natural topography of the area suggest that the lake is not a natural occurrence. **Figure 6-21** contains a photograph of Lake Blake.

6.3.4.1 Environmental Considerations

A site visit conducted in March 2003 showed a highly disturbed area that appears to have been historically dredged and excavated either for aesthetic reasons or as a borrow pit. The lake features a center upland span that appears to consist of side cast material from excavating

Master Stormwater Management Plan

Lake Blake

Figure 6-20

activities or a staging area where heavy equipment accessed the site. Residential and commercial development surrounds the lake, as shown on the existing land use map that appears as **Figure 6-22**. The lake receives runoff from surrounding development, and appears to serve as a stormwater retention area. The lake is connected to surface waters of the state through a storm drain system, which provides a basis for FDEP and USACE jurisdiction. Vegetation in and around the lake consists of wax myrtle, titi, red maple, myrtle leaf holly, yaupon holly, slash pine, needlerush, red cedar, and invasive exotics like torpedo grass and Chinese tallow.

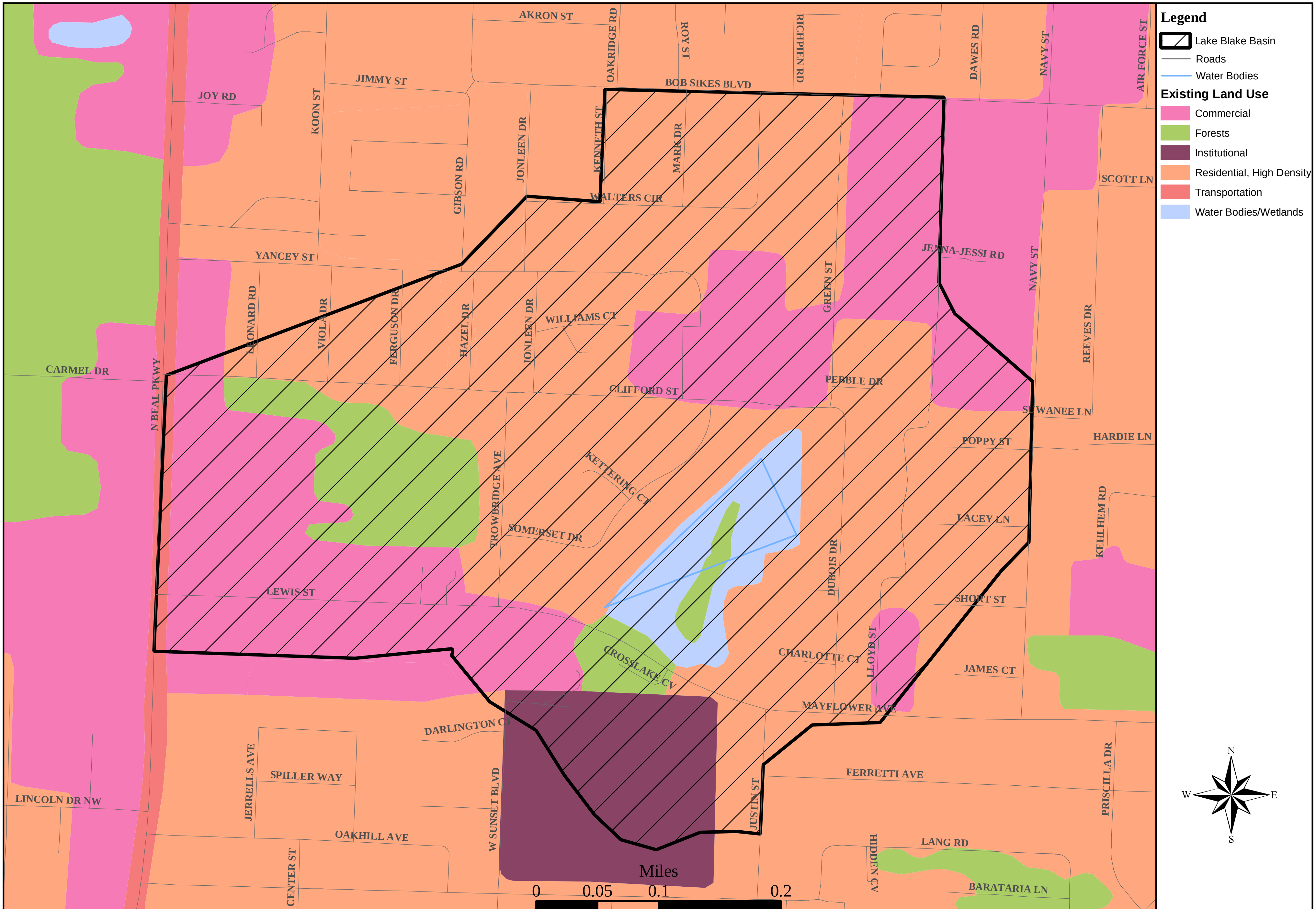
Even though the site appears to have been historically altered, the lake shows signs of stability. Vegetation surrounding the lake appears to be healthy and supporting wildlife and habitat. Waterfowl such as ducks and grebes were seen on site in March as well as red-bellied woodpeckers, and common songbirds. Water quality in the lake appeared poor, most likely due to urban runoff from recent precipitation.

6.3.4.2 Hydrologic and Hydraulic Analysis

Lake Blake discharges to Cinco Bayou through an existing storm drain system. The storm drain system conveys runoff east along Mayflower Avenue, south along Priscilla Drive, west along Lang Road, then south to an outfall located in Cinco Bayou. The system features 30-inch pipes between Lake Blake and Priscilla Drive, and 48-inch pipes from Priscilla Drive to Cinco Bayou.

Residences surrounding the lake would be threatened by excessive stages in the lake. Using encroachment on structures as a benchmark, an allowable stage of 16 feet was established by overlaying contours generated from the County TIN's on aerial photography, and assuming first floor elevations at least 1foot above existing ground. Although this stage should protect surrounding development based on available data, a survey of first floor elevations should be performed to verify this assumption.

Hydrologic and hydraulic analyses were performed on Lake Blake using ICPR2 for the 2-year through 100-year storm frequencies and a 24-hour duration. The results of these analyses appear in **Table 6.19** below.



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Lake Blake Existing Land Use

Figure 6-22

HDR

Table 6.19 Lake Blake Existing Conditions			
Storm Frequency (yr)	**Storm Duration (hr)**	**Allowable Stage (ft)**	**Computed Stage (ft)**
2	24	16	14.04
5	24	16	15.27
10	24	16	16.06
25	24	16	16.83
50	24	16	17.32
100	24	16	18.07

The size of the outfall pipe conveying discharge east along Mayflower Avenue controls the rate of flow out of Lake Blake, and therefore influences the peak stage. Additional hydrologic analyses were performed to determine the extent a larger outfall pipe would improve the lake's LOS, considering 10-year, 25-year, 50-year and 100-year design storms, as presented in **Table 6.20** below.

Table 6.20 Lake Blake With Outfall Improvements				
Outfall Pipe Size (in)	**Storm Frequency (yr)**	**Storm Duration (hr)**	**Allowable Stage (ft)**	**Computed Stage (ft)**
36	10	24	16	15.67
	25	24	16	16.38
	50	24	16	16.83
	100	24	16	17.51
42	10	24	16	15.22
	25	24	16	15.90
	50	24	16	16.30
	100	24	16	16.90

Note that the lake dimensions used for hydraulic routing were initially taken from the County TINs. However, an inaccurate triangular network was observed in the vicinity of the pond, due to interpolation issues between an island in the center of the lake and the shore. Accordingly, a base contour of 11.8 feet was established as the water surface appearing in aerial photography, benchmarked to field observation and survey data. The 16-foot contour, which did not exhibit interpolation issues, was used from the County TIN's. All other elevations were derived from these bounding assumptions.

6.3.4.3 Conclusions

Although Lake Blake is not likely a naturally occurring water body, it has stabilized into a functional ecosystem. In addition, the facility provides both water quality and flood control benefits to the area. Water quality benefits result from the storage of runoff in the lake, which facilitates the settling of suspended solids introduced from urban runoff, and the biological uptake of nutrients. Water quantity benefits result from the attenuation of peak discharge rates as stormwater is conveyed from surrounding development to Cinco Bayou. Accordingly, preservation of the lake's existing function is recommended as a benefit to the community. In addition, better flood control can be achieved by upgrading the storm drain pipe between the lake and Pricilla Drive from a 30-inch pipe to a 42-inch pipe.

6.3.5 Meigs Drive

Meigs Drive is a local road that serves residential development east of Shalimar. The facility features a culvert crossing that connects a wetland to Lake Vivian as shown in **Figure 6-23**. Lake Vivian is a tidally influenced salt water lake with direct access to Choctawhatchee Bay. Meigs Drive periodically floods at the culvert crossing during extreme storm events, due to both freshwater flow and storm surge.

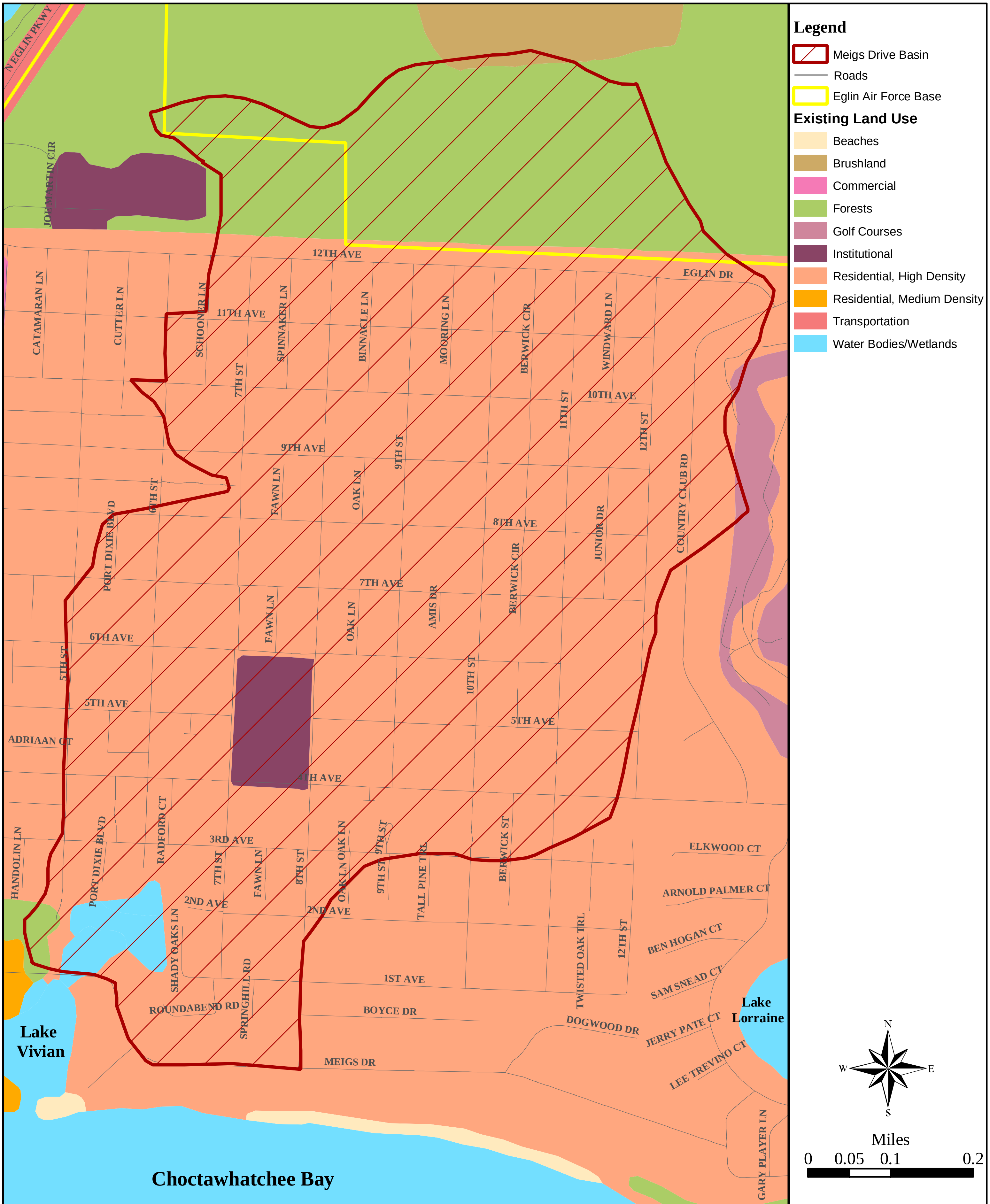
Figure 6-23
Meigs Drive Photograph

Meigs Drive Looking South Toward Bay

6.3.5.1 Existing Conditions

The drainage basin contributing to the culvert at Meigs Drive totals 444 acres. Land use within this basin is largely residential, with some forested areas and recreational use. A map detailing land use appears as **Figure 6-24**.

According to the most recent FEMA Flood Insurance Study (December 2002), the 10-year storm surge reaches 4.0 feet and the 50-year storm surge reaches 6.8 feet. **Figure 6-25** shows the landward extent of the 10-year and the 50-year storm surges.



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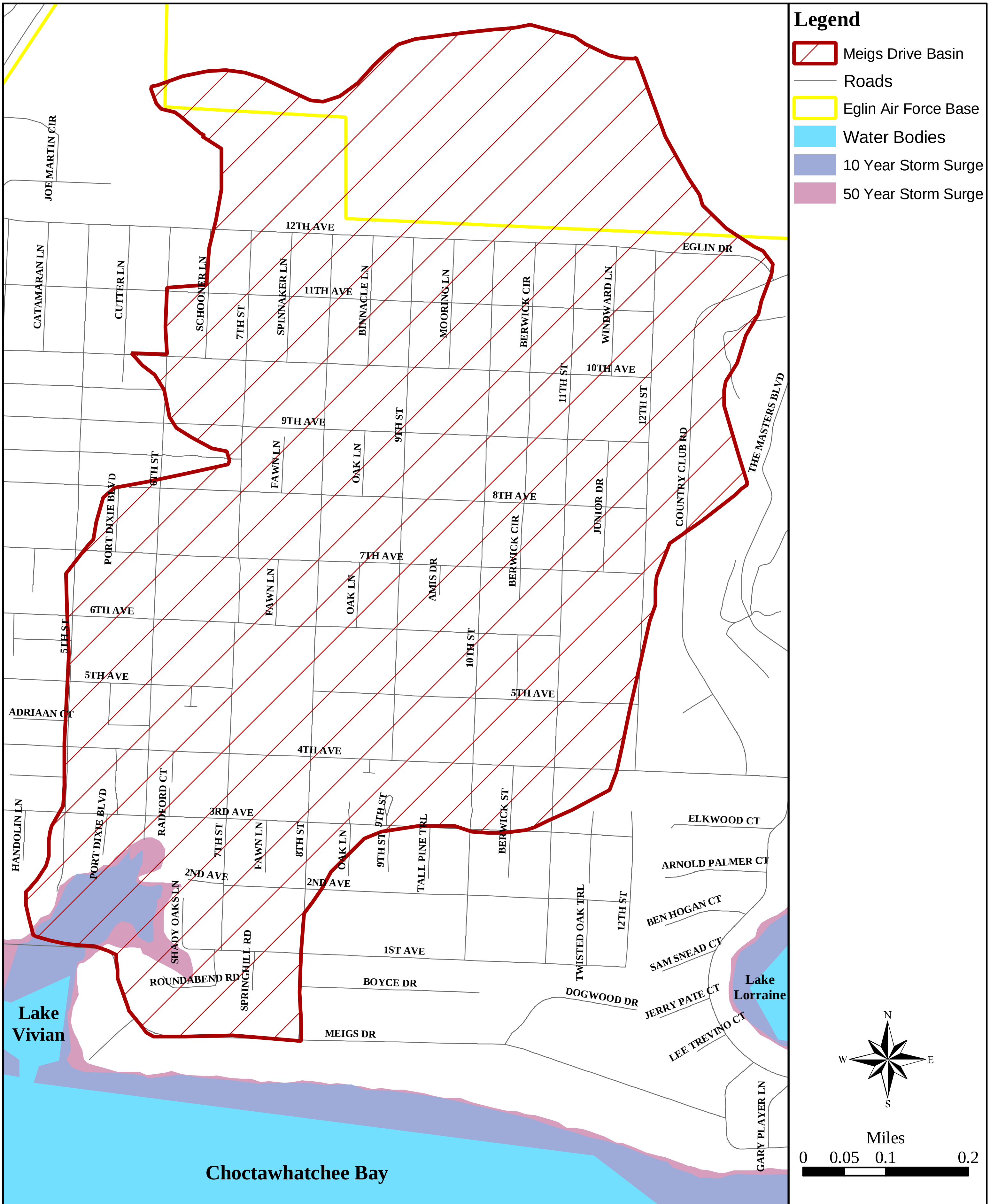
Meigs Drive Existing Land Use

Figure 6-24



Meigs Drive Storm Surge

Figure 6-25

The current roadway has an elevation of 3.1 feet at the culvert, and overtops at an elevation of 2.4 feet approximately 50 feet west of the crossing. This condition results in overtopping from freshwater flows with a 2-year return frequency, and overtopping due to storm surge at a return frequency less than 10-years.

6.3.5.2 Culvert Analysis

All culverts along the corridor were analyzed against a desired overtopping frequency of 10 years for freshwater flow. The Rational Method was used to determine peak runoff, applying existing land use conditions as described in Figure 6-24. All analyses were performed considering both inlet and outlet control following HDS-5 procedures as applied by HY-8, and assuming a tailwater equal to the mean high tide.

Analysis results indicate a need to raise the roadway overtopping elevation to 4.0 feet over the structure. This will allow the culvert to operate during the 10-year storm surge. In addition, this study recommends a larger culvert to increase the LOS during extreme freshwater floods, and to mitigate the increase in headwater associated with raising the roadway profile. A summary of results appears in **Table 6.21** below.

Table 6.21
Meigs Drive Culvert Analysis Summary

Current Overtopping Elevation (ft)	Proposed Overtopping Elevation (ft)	Existing Culvert Size	Proposed Culvert Size[1]	Existing Freshwater Overtopping Frequency	Proposed Freshwater Overtopping Frequency
2.4	4.0	2 – 48”	2 – 6’W x 4’H CBC	2-year	15-year

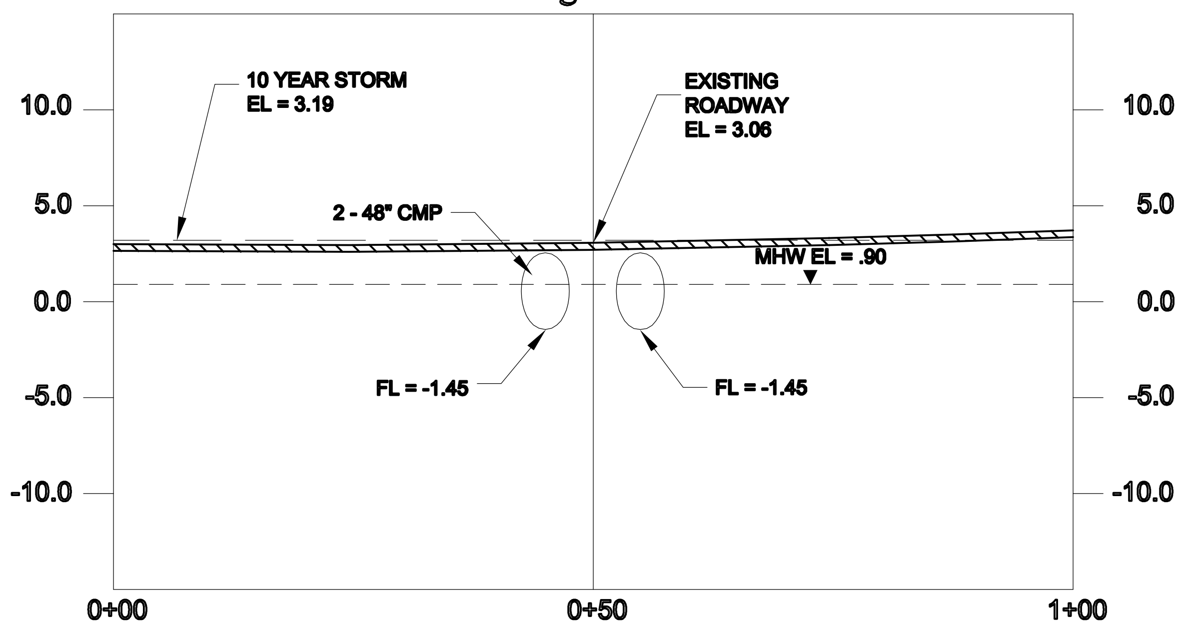
1. CBC – concrete box culvert.

A culvert diagram describing existing and proposed conditions appears in **Figure 6-26**. Detailed analytical results appear in the Hydrologic and Hydraulic Appendices (under separate cover).

Note that it is anticipated that the proposed improvement will increase the headwater at the culvert by 0.88 feet during the 25-year storm event, and more during the 100-year storm event. While it is not anticipated that this will produce property damage based on field review, this evaluation should be confirmed by survey during design.

A sensitivity analysis was performed to analyze the response of the proposed culvert with tailwater conditions exceeding mean high tide. This analysis concluded that the proposed culvert would provide 10-year protection for all tailwaters up to and including 3.0 feet.

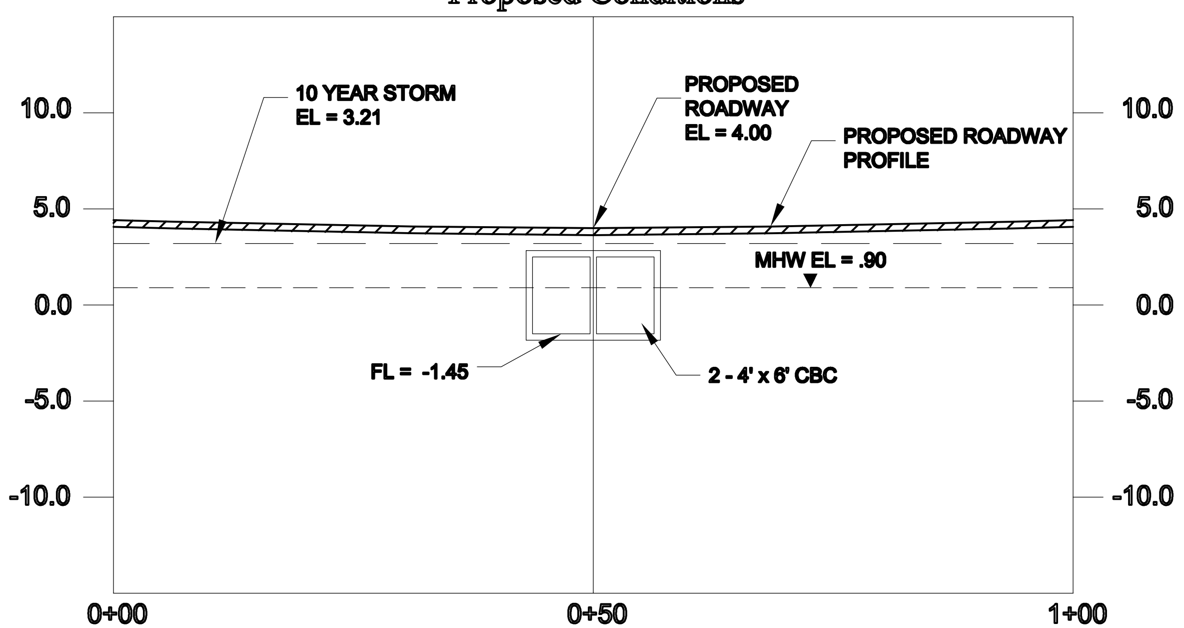
Existing Conditions



SCALE
H: 1"=20'
V: 1"=10'

NOTE: EXISTING PROFILE LOW POINT IS 2.41'

Proposed Conditions



SCALE
H: 1"=20'
V: 1"=10'

Master Stormwater Management Plan

Meigs Drive Culvert Diagram

Figure 6-26

A coincident frequency analysis has not been performed. In other words, the improvements provide 10-year protection against storm surge, and 15-year protection against freshwater flows. However, combinations of more frequent storm surges coincident with more frequent precipitation events could cause overtopping at a higher frequency.

6.3.6 US 98 Box Culverts

6.3.6.1 Existing Conditions

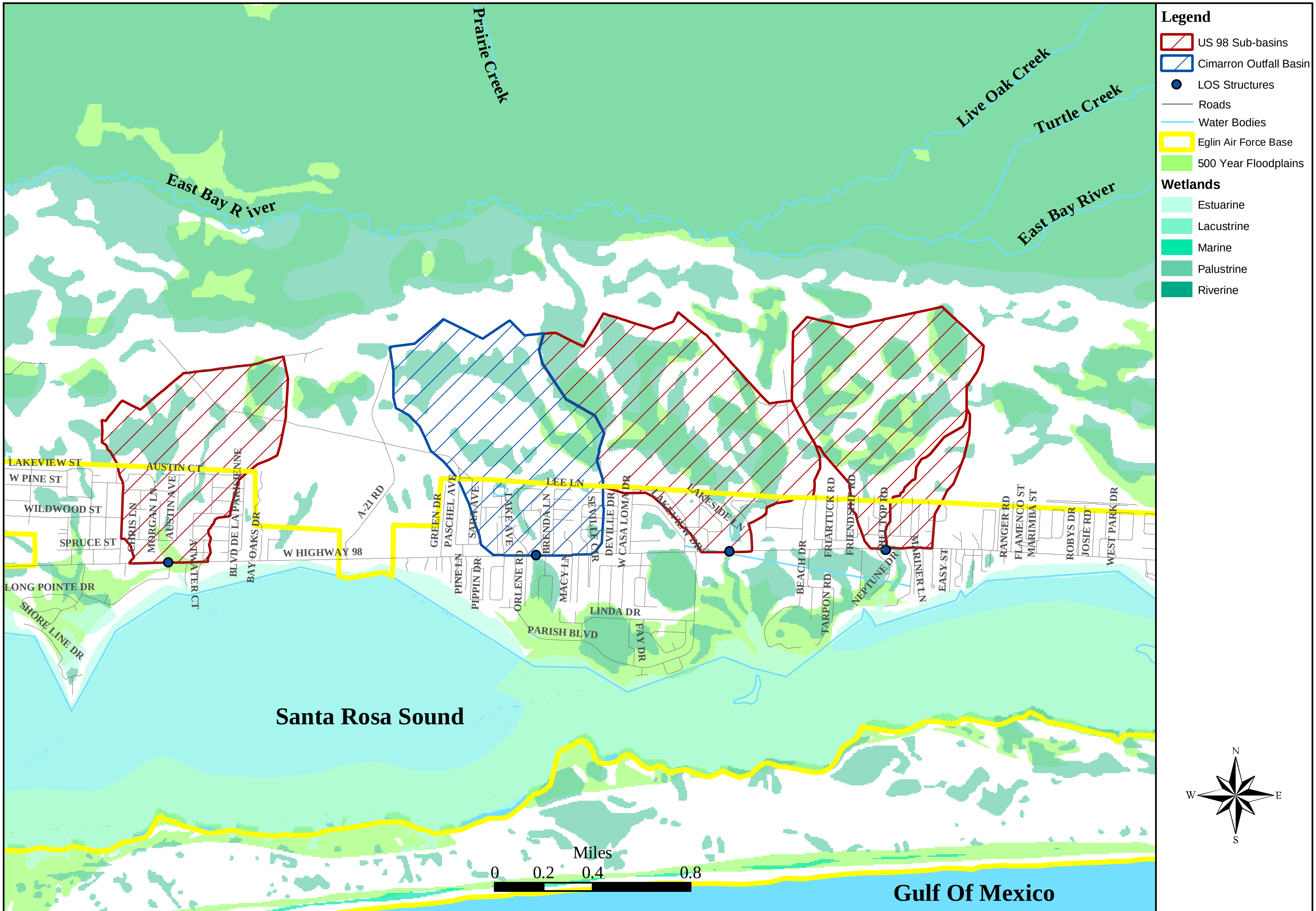
Four box culverts convey runoff under US 98 west of Hurlburt Field (the “US 98 Box Culverts”). The locations of these culverts and their contributing drainage basins appear as **Figure 6-27.** County personnel report that the US 98 Box Culverts have a history of excess sedimentation, and collectively operate with insufficient capacity.

6.3.6.2 Analysis

LOS analyses have been performed on three of the culverts, designated Structures 1, 3 and 205, the results of which appear in Table 6.3. The fourth culvert has been analyzed as part of the Cimarron Outfall Detailed Study Area, the results of which appear in Section 6.3.2 above. These analyses indicate that with regard to capacity, only the culvert at Cimarron requires improvement at this time.

It should be noted that the East Bay River floodplain, located less than a mile north of US 98, may overtop toward US 98 during extreme events, creating capacity issues. This overtopping would occur as elevations in the floodplain exceed 30 feet. Although considered possible by local officials, historical evidence of this overtopping is not available. To better evaluate the threat of overtopping, a gaging station has been recommended to monitor stages in the East Bay River floodplain near Hurlburt Field by the Data Collections Sites Report delivered under separate cover.

With regard to sedimentation, only moderate blockage was observed in the identified box culverts. Generally, the deposition of sediment occurs when velocity in the carrying stream slows, allowing material to settle out of suspension. In natural streams, this can occur due to changes in channel grade, changes in channel roughness, or obstructions such as fallen trees, excessive vegetation, or beaver dams. These flow impediments result in lower velocities, higher stages, and settlement. To minimize the accumulation of undesirable material in the future, regular maintenance is recommended downstream of the culverts.



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US 98 Box Culverts West of Hurlburt Field

Figure 6-27

6.3.6.3 Summary

A summary of the recommended improvements to the US 98 box culverts west of Hurlburt Field follows:

- Upsize Box Culvert at Cimarron to add an additional 5.5'W x 5.5'H CBC to the existing 8'W x 5.5'H barrel

- Install a gage in the East River floodplain near Hurlburt Field to document potential overtopping

- Maintain channels downstream of the culverts free of obstructions

7.0 POLLUTANT LOADING MODEL

This evaluation provides a review of Okaloosa County's watershed's current pollutant loadings and estimates pollutant loadings based on existing and future land use conditions. Information on existing water quality conditions was obtained from the EPA 305(b) and 303(d) reports.

7.1 BASIN GEOGRAPHY

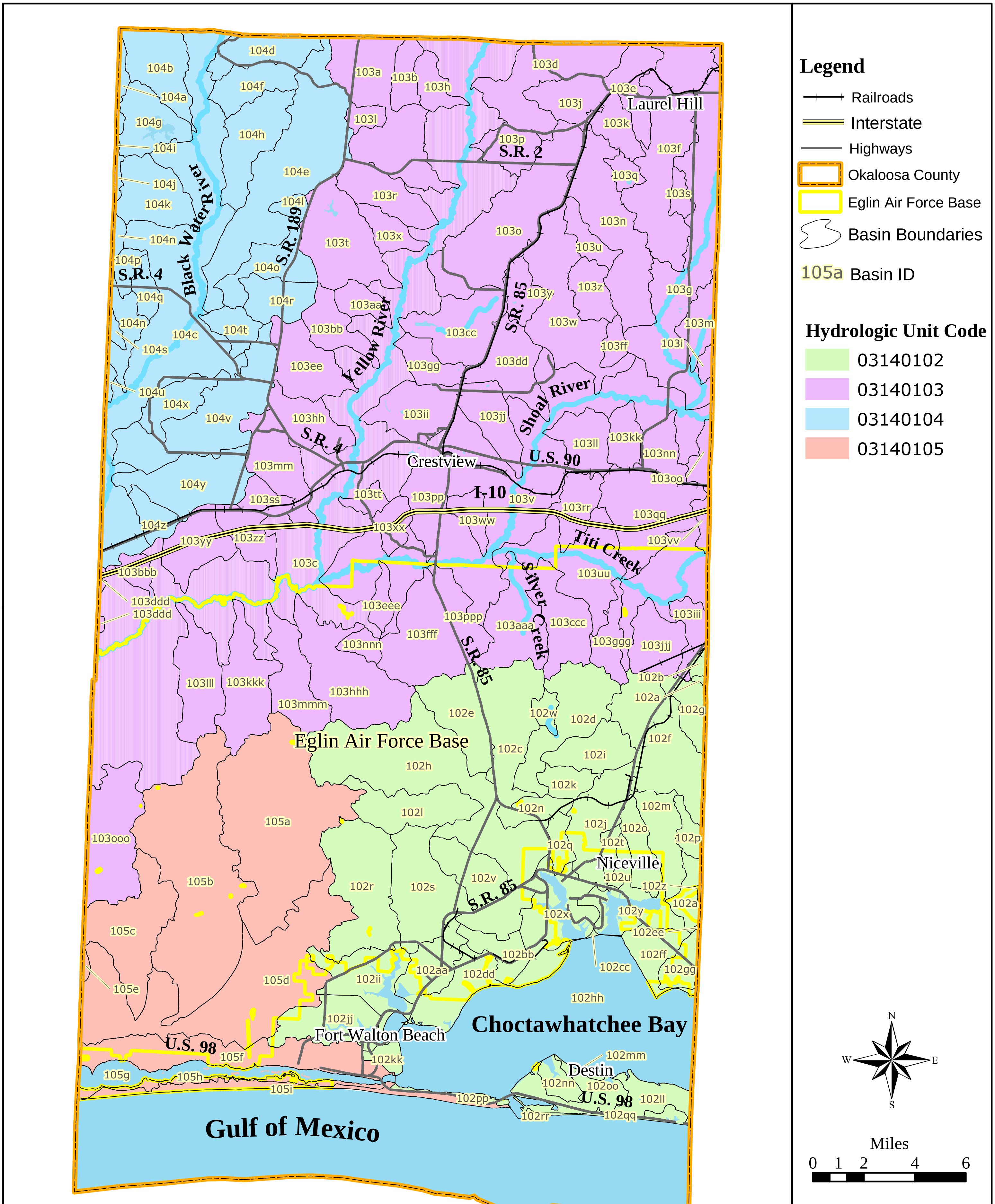
Figure 7-1 illustrates and indexes the sub-basin geography used in the pollutant loading evaluation. This sub-basin data originally prepared by the DEP was obtained from the Florida Geographic Data Library (FGDL). Major hydrologic units included the watershed of the Yellow River (Hydrologic Use Code 03140103), the Black Water River (03140104), the Choctawhatchee Bay drainage area (03140102), and the area draining to Santa Rosa Sound (03140105).

The FGDL identifies a total of 147 sub-basins in Okaloosa County. Figure 7-1 shows each sub-basin by the last three digits that uniquely identify each sub-basin, plus a letter. For example, the Adams Mill Creek sub-basin is designated 104z. The letter designations are arbitrary, and do not denote the sub-basin's place in the watershed with respect to other sub-basins. The sub-basin identifiers are necessary to uniquely label sub-basins, since many sub-basins share the same name and are not hydrologicly connected. For example, four separate streams are named "Long Creek" in the County. For clarity, these were renamed with numeric designators in the GIS tabular database (e.g., "Long Creek 1," "Long Creek 2,"). Note that the sub-basins used for the pollutant loading model do not correspond to those used for the H&H models as a different purposes is served.

7.2 EXISTING WATER QUALITY CONDITIONS

The Federal Clean Water Act requires all states to assess the quality of its' navigable waters and report the results to EPA. The results are compiled to form the 305(b) report, which provides an overview of the water quality for each state. The report provides information on pollution control, aquatic life problems, causes and sources of pollution, and public health problems. It also summarizes water quality statewide by waterbody type and any restoration efforts. The information compiled in the 305(b) report has been used to select Surface Water Improvement and Management (SWIM) priority waters, prepare Florida's Total Maximum Daily Load (TMDL) list, and develop ecosystem management area plans.

Florida's 303(d) list is made up of waterbodies listed as fair and poor in the 305(b) report. The 303(d) list identifies those water quality-limited segments requiring TMDL's which are then



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Basin Index Map

Figure 7-1

ranked for TMDL development. EPA requires submittal of these lists for review and approval in April of even years. The status of the four (for purposes of the water quality analysis the Shoal River Basin is encompassed by the Yellow River Basin) principal basins located in Okaloosa County based on the 303(d) lists is discussed in the following sections.

7.2.1 Blackwater River Basin

The Blackwater River Basin encompasses 253 miles of rivers, streams, and creeks; and 5 square miles of bays and estuaries (EPA). According to the 1998 305(b) list, the Blackwater River Basin contains ten waterbodies that did not meet water quality standards. These water quality-limited segments included: West Fork, Manning Creek, Big Coldwater Creek, East Fork, Big Juniper Creek, three segments of the Blackwater River, Bucket Branch, and Mare Creek. The 303(d) listed water segments included one segment of the Blackwater River and Mare Creek. The parameters of concern included dissolved oxygen, coliforms, mercury based on Fish Consumption Advisory (FCA), and turbidity. All of the water segments in the Blackwater River Basin are targeted for TMDL development in the year 2011.

7.2.2 Yellow River Basin

The Yellow River Basin encompasses 259 miles of rivers, streams, and creeks; and 640 acres of lakes, ponds, and reservoirs (EPA). According to the 1998 305(b) list, the Yellow River Basin contains five waterbodies that did not meet water quality standards. These water quality-limited segments included: Murder Creek, Turkey Creek, Little Creek, and two segments of the Yellow River. The 303(d) listed water segments included one segment of the Yellow River and Murder Creek. The parameters of concern for these waterbodies include dissolved oxygen, coliforms, mercury based on FCA, and turbidity. All of the water segments in the Yellow River Basin are targeted for TMDL development in the year 2011.

7.2.3 Choctawhatchee Bay Basin

The Choctawhatchee Bay Basin encompasses 118 miles of rivers, streams, and creeks; 11,200 acres of lakes, ponds, and reservoirs; and 146 square miles of bays and estuaries (EPA). According to the 1998 305(b) list, the Choctawhatchee Bay Basin contains seven waterbodies that did not meet water quality standards. These water quality-limited segments included: Lafayette Creek, Boggy Bayou, three segments in Choctawhatchee Bay, Joes Bayou, and Indian Bayou. The 303(d) listed water segments included Boggy Bayou, one segment of the Choctawhatchee Bay, Indian Bayou, and Joes Bayou. The parameters of concern included coliforms, dissolved oxygen, mercury based on FCA, total suspended solids, turbidity, biochemical oxygen demand, and nutrients. Two of the Choctawhatchee Bay segments are targeted for TMDL development in the year 2004, while all other segments in the basin are targeted for TMDL development in 2009.

7.2.4 East Bay Basin

The East Bay Basin lies within the Pensacola Bay Basin which encompasses 62 miles of rivers, streams, and creeks; and 209 square miles of bays and estuaries (EPA). According to the 1998 305(b) list, the Pensacola Bay Basin contains nineteen waterbodies that did not meet water quality standards. These water quality-limited segments included: two segments of the Escambia Bay, three segments identified as Direct Runoff To Bay, Pensacola Bay, Pace Mill Creek, Judges Bayou, Mulatto Bayou, Indian Bayou, Carpenter Creek, Trout Bayou, East River Bay, Texar Bayou, Bayou Grande, Bayou Chico, Jones Creek, Jackson Creek, and Bayou Garcon. The 303(d) listed water segments for Pensacola Bay included the East River Bay. The parameters of concern for this water body included coliforms and turbidity. Nine of the water segments are targeted for TMDL development in 2006, while the other ten are targeted for TMDL development in 2011.

7.3 METHODOLOGY

Scientific literature has repeatedly demonstrated a strong association between land use and water quality. Basins with a predominance of upland forest, wetland cover, and low densities of impervious surface tend to be associated with good water and habitat quality. Those dominated by urban and agricultural land uses or characterized by substantial impervious surface area, however, are likely associated with substantial nonpoint source (NPS) pollutant loading and habitat disturbance. Urban land uses generally cause the most severe environmental impacts associated with NPS pollution, including degraded water and sediment quality and physical degradation of benthic and littoral communities. Agricultural uses can lead to sedimentation, stream and habitat alteration, and the export of nutrients and chemicals into surface and ground waters. Silvicultural activities can also cause sedimentation, habitat loss and alteration, and the export of chemical pollutants.

A variety of Best Management Practices (BMPs) exist to ameliorate the water quality degradation caused by NPS runoff. However, because employing these techniques on a regional scale is both difficult and expensive, BMPs should be directed to those areas that contribute the most to NPS pollution and water quality degradation to obtain the most cost-effective results. To identify the parts of Okaloosa County that contribute the most NPS pollution, this study determined the stormwater pollutant loading potential of sub-basins within the County using a simple land use based pollutant loading model. The model was developed following these steps:

- **Identification of Pollutant Loading Rates –** The most appropriate annual pollutant loading rates (i.e., lbs of pollutant per acre, per year) were identified for each major land use type from review of the scientific literature. By multiplying the rates for each pollutant type by acres of each land use type in a sub-basin, the pollutant loading model estimated the total amount of stormwater runoff pollution for each sub-basin.

- **Identification of BMP Effectiveness –** The scientific literature identified the most appropriate BMP pollutant reduction ratio. The pollutant loading model used these BMP reduction rates for the areas assumed to have BMPs in place.

- **Development of Existing and Future Land Use Maps –** Digital maps of existing and future land use were developed using a GIS. These maps supplied the land use acreage information for each watershed and sub-basin needed for the NPS pollutant loading calculations.

These steps are described in more detail in the sections that follow.

7.3.1 Pollutant Loading Rates and Land Use

Based on a review of previous NPS pollution studies the NPS loadings from the *St. Marks and Wakulla Rivers Resource Assessment & Greenway Protection Plan (St. Marks Plan)* were determined to be the most appropriate for use as the source of the loading rates for this study. The proximity of the *St. Marks Plan* study area to Okaloosa County and its similarity in topography and land use composition suggested that loading rates suitable for the St. Marks and Wakulla River Basins would be appropriate for Okaloosa County. The loading rates used in the *St. Marks Plan* that appear in **Table 7.1** were used to calculate NPS pollutant loadings within the sub-basins for TN, TP, BOD, and TSS.

Table 7.1
Corresponding Land Use and Pollutant Loading Rates

Land Use	Pollutant Loading Rates (lb/ac/yr)			
	TN	TP	BOD	SS
Commercial	21.1	3.14	131	895
Cropland/Pasture	8.89	1.32	14.6	212
Extractive	5.37	0.68	43.7	427
High Density Residential	19.5	4.36	98.3	677
Industrial	17.9	3.1	96.0	936
Institutional	5.55	0.71	73.5	475
Lakes and Streams	7.88	0.69	10.7	19.5
Low Density Residential	5.76	0.74	16.1	55.9
Medium Density Residential	10.1	1.63	37.2	100
Recreation/Open Space	2.76	0.12	3.20	24.5
Silviculture	2.67	0.42	8.89	118
Spoil/Barren	4.06	0.40	23.5	226
Transportation/Utilities	8.00	1.01	67.1	460
Upland Forest	2.67	0.42	8.89	118

7.3.2 Best Management Practice Pollutant Removal Efficiencies

Stormwater runoff is a significant source of NPS pollution, having solids concentrations equal to or greater than untreated sanitary wastewater, and BOD values approximately equal to those of secondary effluent (Florida Greenways Program, 1994). Stormwater BMPs help to control the volume and the speed of runoff before it enters receiving waters and promote the seepage of rainwater into groundwater storage areas. There are two classes of BMPs that are used either individually or in combination to manage urban runoff.

7.3.2.1 Structural Best Management Practices

Structural BMPs involve building an engineered facility to manage water for quality, quantity, or both at either the point of generation or point of discharge to either a storm sewer system or to receiving waters. Most of these involve some type of maintenance. The most common structural BMPs can be categorized as either retention or detention systems.

Table 7.2 shows pollutant removal efficiencies based on *Pollutant Removal Efficiencies for Typical Stormwater Management Systems in Florida* by Dr. Harvey Harper (1985). Because the terms “detention” and “retention” are often used interchangeably, the two terms were defined as follows:

- **Detention -** The collection and temporary storage of stormwater, generally for a period of time ranging from 24-72 hours, in such a manner as to provide for treatment through physical, biological or chemical processes with subsequent gradual release of stormwater to downstream receiving waters

- **Retention -** On-site storage of stormwater with subsequent disposal by infiltration into the ground or evaporation in such a manner as to prevent direct discharge of stormwater runoff into receiving waters

Table 7.2
Pollutant Removal Efficiencies for Stormwater BMPs in Florida

Type of System	Total N (%)	Total P (%)	BOD (%)	TSS (%)
Off-line Retention/Detention	60	**85**	**80**	**90**
Wet Retention	40	50	40	**85**
Wet Detention	25	65	55	**85**
Wet Detention with Filtration	0	60	**99**	**98**
Dry Detention	15	25	40	70
Dry Retention				
0.25-inch retention	60	60	60	60
0.50-inch retention	**80**	**80**	**80**	**80**
0.75-inch retention	**90**	**90**	**90**	**90**
1.00-inch retention	**95**	**95**	**95**	**95**
1.25-inch retention	**98**	**98**	**98**	**98**
Dry Detention with Filtration				
Type A or B soils	0	0	0	75
Type C or D soils	0	0	0	60
Alum Treatment	50	**90**	75	**90**
Source: (Harper 1995)				

According to this study, the bold categories in Table 7.2 meet the State Water Policy Goal of 80 percent reduction for pollutants.

The State of Florida implemented statewide water quality treatment rules with s. 17-25, Florida Administrative Code (now s. 62-25, F.A.C.) in 1983. For this analysis, reductions in pollutant loading due to existing structural controls were deemed insignificant compared to the scale of the analysis. However, for the future land use scenario, all contiguous polygons of the same land use type greater than 10 acres in size were assumed to employ 0.5 inches dry retention, resulting in an 80 percent decrease in predicted stormwater pollutant loadings for those newly developed areas.

7.3.2.2 Non-structural Best Management Practices

Non-structural BMPs do not require construction of a facility, but provide for the development of pollution control programs that may include prevention, education, and regulation. The following are some of the most common elements of non-structural BMPs used today:

- Planning and regulatory tools
- Conservation, recycling, and source control
- Maintenance and operational procedures
- Educational and outreach programs

Removal efficiencies for non-structural BMPs are difficult to identify because they rely on behavioral changes in order to be effective. Furthermore, for the purpose of the pollutant loading model employed in this study, it would have been necessary to predict the geographic extent of each non-structural BMP so that improvements could be applied to the appropriate land use areas. Since neither of these conditions was met in this study, the pollutant reducing effects of non-structural BMPs were not considered in either the existing or future land use scenarios.

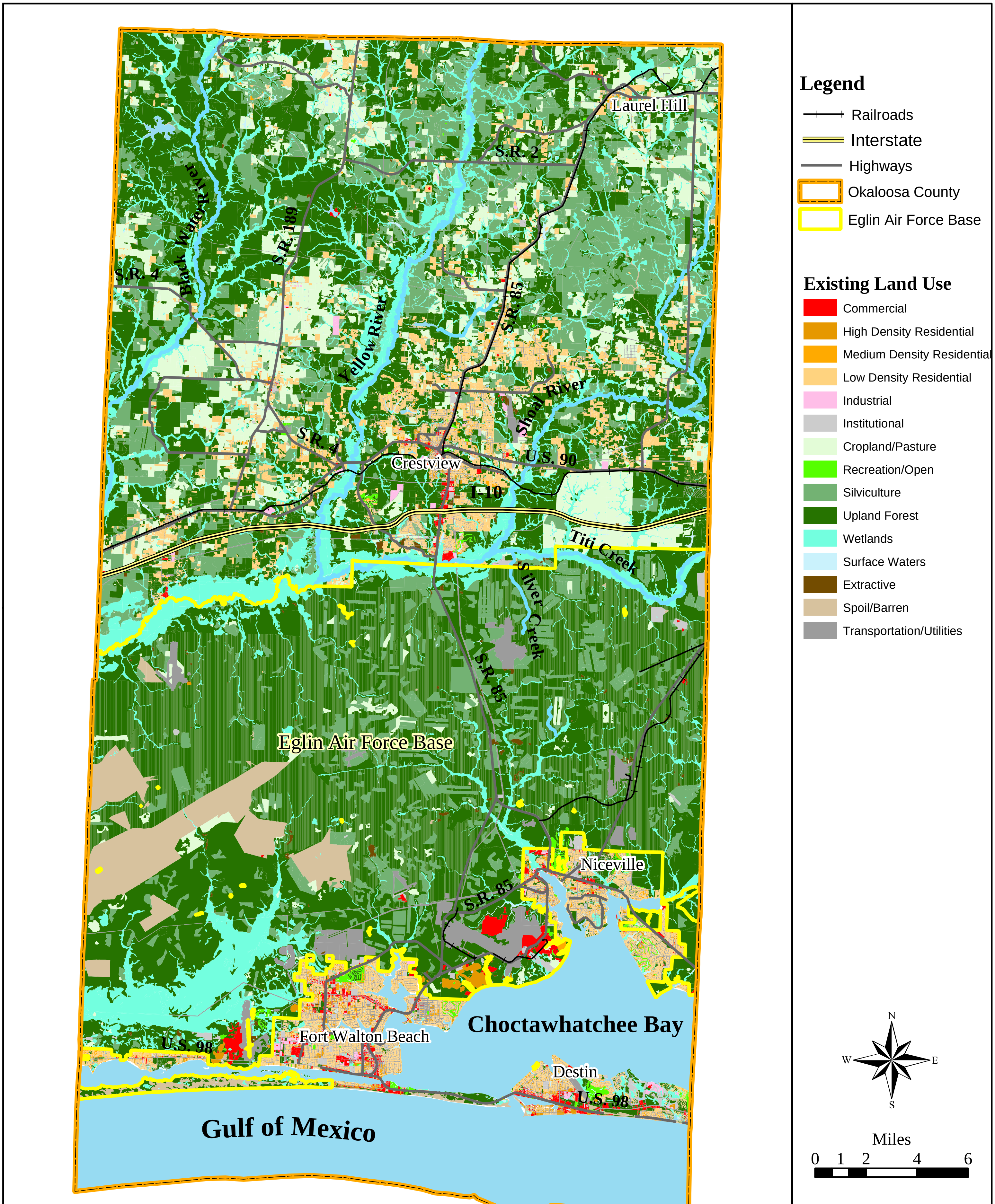
7.3.3 Land Use Scenarios

The pollutant loading model evaluated both existing and future land use scenarios. An existing land use (ELU) database was created from several sources to represent 2002 conditions throughout Okaloosa County. Using the ELU database as a starting point, a future land use (FLU) database was developed to represent the future "build-out" condition of the County based on currently adopted comprehensive plans.

7.3.3.1 Existing Land Use

Generally, the existing land use data used for the pollutant loading analysis was developed the same way as for the H&H models discussed in Section 2.1.2.1. In addition to this data current parcel data obtained from Okaloosa County was overlaid to bring the NWFWMD data up to 2002 conditions.

The final ELU designations used were consistent with the fifteen land use classifications in Table 7.1, so that pollutant loading rates could be clearly matched with each polygon. **Figure 7-2** shows the County's future land use by these 15 classifications.



Master Stormwater Management Plan

Existing Land Use Based on Water Quality Analysis Categories

Figure 7-2

HDR

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7.3.3.2 Future Land Use

The same future land use data discussed in Section 2.1.2.2 was used for the pollutant loading model. **Figure 7-3** shows the County's future land use by the 15 categories used for pollutant loading.

7.3.4 Analysis

Following development of the pollutant loading rates, BMP effectiveness, and land use maps pollutant loadings were calculated for the sub-basins. The sub-basin pollutant loadings were determined by multiplying the various pollutant loading rates for each land use by the area of that land use type within each sub-basin. These loadings were then totaled by type of pollutant loading (TN, TP, BOD, TSS) for each sub-basin.

Detailed results of the pollutant loading model, summarized below, appear in the Water Quality Evaluation Report prepared as part of this Master Plan and delivered under separate cover. The Water Quality Evaluation Report presents loading data in two formats, including total pollutant loading (lb/year) and pollutant loading normalized by area (lb/ac/yr). Because loading normalized by area provides a better understanding of the concentration of pollutants, this summary is limited to a discussion of loading normalized by area.

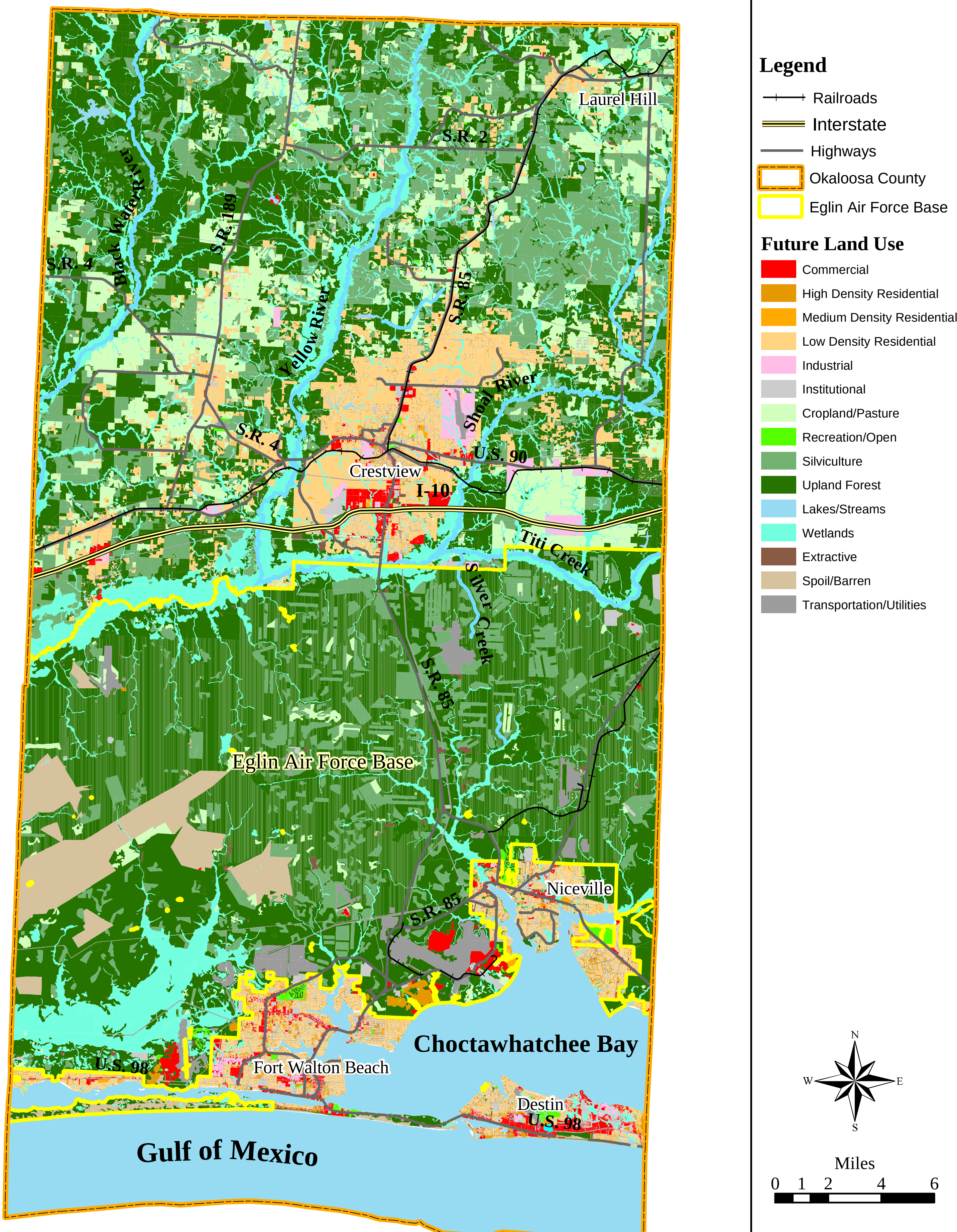
7.4 RESULTS

7.4.1 Existing Land Use

Figure 7-2 shows the County's existing land use based on the 15 classifications used for the pollutant loading analysis. Table 2.2 shows the percentage of each of these classifications within the County.

7.4.2 Annual Pollutant Loadings By Sub-basin, Existing Land Use

Table 7.3 lists the total annual pollutant loadings for each sub-basin (normalized by area) generated by the existing land use in Okaloosa County for each of four pollutants, in pounds of pollutant per acre, per year (lbs/ac/year). The percentile rank of each sub-basin was calculated for each annual pollutant loading value for each sub-basin. The percentile value for a particular sub-basin represents the percentage of the rank-ordered sub-basins that have a lower pollutant loading value. For example, a sub-basin with a percentile value of 80 percent has a pollutant loading greater than that of 80 percent of the other sub-basins in the County.



Master Stormwater Management Plan

Future Land Use Based on Water Quality Analysis Categories

Figure 7-3

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Table 7.3
Existing Land Use Annual Pollutant Loadings By Basin
(normalized by basin area)

BasinID	Basin Name	Acres	Total Nitrogen		Total Phosphorus		Biochemcial Oxygen Demand		Total Suspended Solids	
			lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile
104z	ADAMS MILL CREEK	1,461.9	5.28	77%	0.74	76%	17.12	73%	148.43	72%
102bb	AIRPORT DRAIN	1,976.0	**6.94**	**91%**	**0.92**	**89%**	**51.13**	**99%**	**365.46**	**100%**
102n	ANDERSON BRANCH	946.2	3.17	23%	0.48	28%	10.70	24%	124.42	34%
103mm	BAGGETT CREEK	4,184.4	5.24	75%	0.75	76%	14.19	62%	133.29	54%
103l	BAILY BRANCH	841.5	**6.48**	**89%**	**0.91**	**89%**	14.82	66%	153.86	74%
104l	BARREL BRANCH	686.3	2.94	12%	0.44	10%	9.64	11%	111.12	7%
103x	BEAR BRANCH	1,634.7	3.65	37%	0.52	38%	11.89	37%	117.59	21%
103ll	BEAR CREEK	2,919.7	4.28	56%	0.62	57%	14.57	65%	137.72	61%
104n	BEAVER CREEK	2,454.6	4.17	53%	0.61	53%	11.37	32%	124.10	33%
102p	BEE BRANCH	987.1	2.81	5%	0.43	6%	10.53	23%	123.08	31%
103dd	BENDS CREEK	4,019.9	5.12	73%	0.71	72%	16.32	71%	125.92	38%
104s	BIG BRANCH	375.4	3.12	18%	0.46	18%	9.82	15%	103.89	2%
103d	BIG CREEK	6,310.7	3.97	45%	0.58	45%	11.57	34%	120.78	27%
103qq	BIG FORK	6,801.7	5.30	78%	0.77	78%	13.93	59%	148.86	73%
103a	BIG HORSE CREEK	8,660.7	4.45	58%	0.65	60%	12.63	47%	132.29	50%
104c	BLACKWATER RIVER	23,989.0	4.15	50%	0.60	50%	11.27	30%	118.52	24%
103ggg	BLUE SPRING CREEK	1,933.0	2.76	3%	0.43	4%	9.07	2%	114.61	15%
102x	BOGGY BAYOU	3,903.9	**8.89**	**99%**	**1.28**	**97%**	**51.83**	**100%**	**344.81**	**99%**
104d	BOGGY HOLLOW CREEK	3,005.5	4.71	65%	0.69	67%	12.39	43%	138.29	62%
103ooo	BOILING CREEK	6,537.5	3.79	41%	0.43	3%	19.32	79%	**194.95**	**86%**
104y	BONE CREEK	5,475.4	4.46	58%	0.65	63%	12.04	39%	130.06	45%
103aa	BUCKHANNON BRANCH	1,507.3	5.22	74%	0.74	75%	13.11	54%	128.08	41%
103jjj	BULL CREEK	3,160.0	3.03	13%	0.46	19%	13.43	56%	140.06	63%
104k	BULL PEN BRANCH	4,157.2	3.35	29%	0.49	32%	10.37	21%	111.13	8%
103j	CAMBELLS MILL CREEK	2,707.3	4.12	49%	0.61	54%	12.27	41%	139.90	63%
103bbb	CANOE CREEK	1,033.0	4.24	54%	0.61	54%	14.88	67%	147.65	71%
102hh	CHOCTAWHATCHEE BAY	165.3	**6.09**	**87%**	**0.78**	**81%**	**23.07**	**85%**	136.76	58%
102jj	CINCO BAYOU	3,884.3	**8.77**	**98%**	**1.34**	**98%**	**46.86**	**97%**	**306.88**	**97%**
103ff	CLEAR CREEK	878.3	**5.69**	**84%**	**0.82**	**84%**	14.66	65%	155.15	75%
103bb	COTTON CREEK	2,520.0	**6.36**	**89%**	**0.93**	**90%**	17.37	74%	**174.61**	**83%**
103u	CYPRESS POND BRANCH	1,501.7	3.15	20%	0.47	26%	9.91	16%	112.36	9%
104j	DANLEY BRANCH	313.4	2.92	11%	0.45	14%	9.60	10%	112.87	10%
103gg	DAVIS MILL CREEK	1,978.2	**5.61**	**82%**	**0.78**	**82%**	16.80	72%	137.64	60%
103t	DEADFALL CREEK	6,417.9	3.80	41%	0.55	44%	11.94	38%	123.29	32%
102rr	DESTIN HARBOR	19.1	**7.37**	**93%**	**1.02**	**92%**	**30.70**	**90%**	**206.53**	**88%**
102cc	DIRECT RUNOFF TO BAY 1	40.4	**6.08**	**86%**	**0.87**	**86%**	**25.09**	**87%**	147.20	69%
102dd	DIRECT RUNOFF TO BAY 2	4,331.5	**7.93**	**95%**	**1.36**	**100%**	**40.17**	**93%**	**286.19**	**95%**
102kk	DIRECT RUNOFF TO BAY 3	792.0	**8.47**	**97%**	**1.19**	**95%**	**45.25**	**96%**	**279.93**	**94%**

Table 7.3
Existing Land Use Annual Pollutant Loadings By Basin
(normalized by basin area)

BasinID	Basin Name	Acres	Total Nitrogen		Total Phosphorus		Biochemcial Oxygen Demand		Total Suspended Solids	
			lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile
102mm	DIRECT RUNOFF TO BAY 4	258.5	**7.57**	**94%**	**1.08**	**93%**	**37.76**	**93%**	**221.80**	**90%**
102ll	DIRECT RUNOFF TO BAY 5	2,115.4	**6.20**	**88%**	**0.88**	**86%**	**37.37**	**92%**	**264.96**	**93%**
102ff	DIRECT RUNOFF TO BAY 6	2,340.4	**6.93**	**91%**	**1.01**	**91%**	**31.58**	**91%**	**196.37**	**87%**
105f	DIRECT RUNOFF TO BAY 7	7,973.0	**7.78**	**95%**	**1.17**	**95%**	**42.30**	**95%**	**290.74**	**95%**
105h	DIRECT RUNOFF TO BAY 8	1,397.9	4.85	68%	0.69	67%	**21.72**	**81%**	**170.29**	**82%**
102pp	DIRECT RUNOFF TO BAY 9	477.2	4.01	46%	0.50	34%	19.13	78%	161.39	78%
102qq	DIRECT RUNOFF TO GULF 1	882.5	**8.14**	**96%**	**1.22**	**96%**	**42.76**	**95%**	**296.75**	**96%**
105i	DIRECT RUNOFF TO GULF 2	2,152.7	4.78	67%	0.63	58%	**27.06**	**89%**	**234.63**	**91%**
104i	DOGWOOD BRANCH	38.6	2.67	0%	0.42	0%	8.89	0%	118.00	23%
102ee	EAGLE CREEK	24.0	2.67	0%	0.42	1%	8.89	1%	118.00	22%
105d	EAST RIVER BAY	17,984.5	4.51	59%	0.60	52%	19.06	78%	129.40	44%
102b	EXLINE CREEK	386.9	2.70	2%	0.42	2%	8.95	2%	116.72	17%
102ii	GARNIER BAYOU	3,715.7	**7.39**	**93%**	**1.08**	**94%**	**41.06**	**94%**	**276.55**	**93%**
102s	GARNIER CREEK	6,275.2	3.54	34%	0.52	39%	16.27	70%	160.00	77%
103eee	GOPHER CREEK	1,022.3	2.92	10%	0.44	8%	9.43	8%	106.82	4%
103k	GREEN BRANCH	1,923.8	5.08	72%	0.74	73%	14.15	60%	144.31	68%
103tt	GULLY BRANCH	836.2	3.56	36%	0.49	31%	11.30	31%	111.01	6%
103ee	GUM CREEK 1	4,165.2	5.05	71%	0.74	73%	12.54	44%	140.21	64%
103vv	GUM CREEK 2	660.1	3.14	19%	0.47	22%	12.75	49%	133.00	53%
103ccc	HONEY CREEK	4,129.9	2.76	2%	0.43	4%	9.27	6%	116.83	18%
103f	HORSEHEAD CREEK	9,973.2	4.63	65%	0.67	66%	11.87	36%	133.39	55%
104g	HURRICANE CREEK	3,421.2	3.58	36%	0.49	32%	9.77	13%	105.18	3%
102oo	INDIAN BAYOU	2,746.5	**8.55**	**97%**	**1.26**	**97%**	**45.86**	**97%**	**304.00**	**97%**
102nn	JOES BAYOU	1,043.5	**8.92**	**100%**	**1.34**	**99%**	**49.93**	**98%**	**327.56**	**98%**
103ddd	JULIAN MILL CREEK	246.1	5.26	76%	**0.78**	**80%**	**22.39**	**82%**	**208.36**	**89%**
103n	JUNIPER CREEK 1	7,833.6	3.35	28%	0.49	33%	10.42	21%	117.11	19%
103pp	JUNIPER CREEK 2	2,806.5	**5.93**	**85%**	**0.90**	**88%**	**29.15**	**89%**	**225.09**	**91%**
102c	JUNIPER CREEK 3	6,523.0	3.13	19%	0.46	20%	12.83	50%	134.50	57%
103ww	KING BRANCH	1,369.5	5.04	71%	0.69	68%	**21.10**	**80%**	140.22	65%
103z	KIRKLAND BRANCH	2,123.1	3.52	34%	0.53	40%	9.93	17%	119.77	26%
103oo	LAIRD MILL CREEK	1,014.3	4.89	69%	0.70	70%	12.60	46%	129.09	42%
104w	LIGHTER KNOT CREEK	1.2	4.02	47%	0.51	35%	12.07	39%	53.78	0%
102r	LIGHTWOOD KNOT CREEK	7,649.3	3.68	38%	0.51	36%	18.42	76%	**174.67**	**84%**
103b	LITTLE HORSE CREEK	1,604.0	3.51	33%	0.51	36%	10.85	25%	112.40	10%
102f	LITTLE ROCKY CREEK	7,613.4	2.82	6%	0.43	7%	9.40	8%	115.04	16%
102gg	LITTLE TROUT CREEK	1,533.1	4.15	52%	0.60	51%	16.63	71%	140.85	66%
105b	LIVE OAK CREEK	18,045.8	3.41	31%	0.47	21%	12.21	41%	121.89	30%
104q	LONG BRANCH 1	1,085.0	4.33	56%	0.64	60%	11.47	32%	137.09	60%

Table 7.3
Existing Land Use Annual Pollutant Loadings By Basin
(normalized by basin area)

BasinID	Basin Name	Acres	Total Nitrogen		Total Phosphorus		Biochemcial Oxygen Demand		Total Suspended Solids	
			lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile
102k	LONG BRANCH 2	1,426.1	2.87	7%	0.44	10%	9.21	5%	114.87	15%
103i	LONG CREEK 1	451.8	3.09	16%	0.45	15%	10.45	22%	106.36	4%
103rr	LONG CREEK 2	2,388.7	**7.30**	**92%**	**1.06**	**93%**	15.05	69%	**172.40**	**82%**
102m	LONG CREEK 3	3,033.9	2.88	8%	0.44	9%	9.66	12%	113.41	13%
102z	LONG CREEK 4	71.4	3.27	26%	0.46	19%	10.29	20%	89.55	0%
103nnn	LOST BOY POND OUTLET	898.7	2.97	13%	0.45	16%	9.18	4%	117.42	19%
103q	MACK BRANCH	839.2	4.53	60%	0.65	63%	11.21	30%	129.67	45%
103kkk	MALONE CREEK	5,008.8	2.79	4%	0.43	5%	9.17	3%	113.19	12%
104h	MARE CREEK 1	4,628.6	3.55	35%	0.52	39%	10.53	23%	117.86	21%
103nn	MARE CREEK 2	2,052.8	4.14	50%	0.58	47%	14.00	60%	121.85	29%
103ii	MATHISON CREEK	3,839.2	**5.57**	**80%**	**0.78**	**80%**	**22.60**	**84%**	158.81	76%
103lll	METTS CREEK	4,506.1	3.06	15%	0.47	23%	11.58	34%	129.30	43%
104u	MIDDLE CREEK 1	262.7	3.24	24%	0.46	17%	10.24	19%	90.71	1%
103mmm	MIDDLE CREEK 2	4,144.8	2.82	6%	0.43	8%	9.19	4%	113.61	13%
102g	MIDDLE ROCKY CREEK	1,900.6	2.91	9%	0.44	11%	9.93	17%	116.56	17%
103p	MILL CREEK 1	2,129.2	3.95	44%	0.58	47%	12.28	42%	133.45	56%
103hh	MILL CREEK 2	3,154.3	4.61	64%	0.65	62%	15.02	68%	130.70	46%
102q	MILL CREEK 3	1,125.8	3.74	39%	0.47	26%	14.35	63%	125.72	36%
104m	MINCY BRANCH	0.4	2.67	1%	0.42	0%	8.89	0%	118.00	23%
104p	MUDDY BRANCH	980.4	4.57	63%	0.66	65%	12.15	40%	129.29	43%
103o	MURDER CREEK	10,346.2	4.52	60%	0.65	64%	12.57	45%	130.74	47%
104o	NARROWS CREEK	3,188.2	4.39	57%	0.65	61%	11.81	35%	132.90	52%
102i	NINEMILE CREEK	2,914.5	3.16	21%	0.47	25%	12.42	43%	133.00	52%
104f	OAK CREEK	2,726.2	4.12	48%	0.60	50%	11.87	36%	124.58	34%
104e	PANTHER CREEK	12,764.9	3.98	45%	0.58	46%	11.19	29%	122.84	30%
102e	PARRISH CREEK	4,228.6	3.10	17%	0.47	23%	11.19	28%	127.40	40%
103eee	PEARL CREEK	2,345.1	3.32	28%	0.48	29%	14.38	63%	140.34	65%
104v	PENNY CREEK	8,538.2	**5.54**	**80%**	**0.81**	**82%**	13.40	56%	147.31	71%
102w	PINE LOG CREEK 1	765.4	3.17	22%	0.49	30%	9.45	9%	121.10	28%
103m	PINE LOG CREEK 2	1,914.5	3.78	40%	0.55	43%	11.50	33%	114.20	14%
103jj	PINEY WOODS CREEK	3,021.1	**5.65**	**82%**	0.78	79%	**22.11**	**82%**	158.19	76%
103r	POLLEY CREEK	3,368.6	3.40	30%	0.49	30%	10.94	26%	107.91	5%
103g	POND CREEK	12,043.6	4.84	67%	0.70	69%	12.74	48%	131.43	47%
104x	POPLAR HEAD	2,475.9	3.90	43%	0.58	45%	11.04	27%	125.79	37%
102aa	POQUITO BAYOU	2,831.6	**5.66**	**83%**	**0.86**	**85%**	**32.43**	**91%**	**253.26**	**92%**
103w	POVERTY CREEK	7,681.4	4.28	55%	0.61	55%	12.58	45%	124.91	35%
105c	PRAIRIE CREEK	6,864.9	3.37	30%	0.46	17%	12.71	47%	129.06	41%
104r	PYRON SPRING BRANCH	2,665.1	**6.60**	**90%**	**0.97**	**91%**	14.93	67%	166.71	79%

Table 7.3
Existing Land Use Annual Pollutant Loadings By Basin
(normalized by basin area)

BasinID	Basin Name	Acres	Total Nitrogen		Total Phosphorus		Biochemcial Oxygen Demand		Total Suspended Solids	
			lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile
103iii	RAMER CREEK	1,649.0	2.91	10%	0.45	12%	10.93	26%	124.92	36%
104t	RED WASH BRANCH	1,673.0	4.04	47%	0.60	52%	11.13	28%	131.93	50%
103ss	RESERVOIR OUTLET	1,286.5	**5.60**	**81%**	**0.82**	**83%**	17.07	73%	**168.83**	**80%**
104b	ROCK CREEK	4,185.1	3.48	32%	0.51	37%	10.12	19%	112.05	8%
102y	ROCKY BAYOU	2,465.1	**5.81**	**84%**	**0.85**	**84%**	**25.59**	**88%**	**169.59**	**81%**
102a	ROCKY CREEK	1,560.3	3.09	17%	0.45	13%	9.80	14%	99.12	2%
102l	ROGUE CREEK	4,290.0	2.78	4%	0.43	6%	9.39	7%	119.87	26%
103y	RUM STILL BRANCH	1,842.9	5.18	73%	0.75	77%	13.75	57%	145.73	69%
102u	SANDERS BRANCH	702.2	5.41	79%	0.70	69%	**22.95**	**84%**	142.88	67%
105g	SANTA ROSA SOUND	137.9	4.56	62%	0.63	58%	18.20	76%	136.87	59%
102t	SHAW STILL BRANCH	704.6	4.58	63%	0.62	56%	**25.01**	**86%**	**192.29**	**86%**
103v	SHOAL RIVER	27,342.0	4.72	66%	0.66	65%	17.47	75%	134.94	58%
103cc	SILVER CREEK 1	5,008.0	4.97	70%	0.71	71%	13.86	58%	131.70	49%
103aaa	SILVER CREEK 2	4,816.5	3.73	39%	0.54	41%	**20.14**	**80%**	**183.77**	**84%**
104a	SWEETWATER CREEK	645.5	3.47	32%	0.53	41%	9.82	15%	123.45	32%
102j	SWIFT CREEK	4,521.5	4.15	51%	0.59	48%	**22.47**	**83%**	**186.67**	**85%**
102d	TENMILE CREEK	5,484.7	3.08	15%	0.47	21%	9.79	13%	117.44	20%
103uu	TITI CREEK	11,731.0	4.16	52%	0.59	49%	13.10	53%	126.10	39%
102v	TOMS CREEK	5,123.6	4.85	69%	0.71	71%	**24.80**	**86%**	**211.70**	**89%**
103yy	TRAWICK CREEK	3,397.4	4.24	54%	0.62	56%	16.07	69%	150.48	73%
102h	TURKEY CREEK 1	15,054.2	3.30	27%	0.50	34%	12.76	50%	133.23	54%
102o	TURKEY CREEK 2	2,179.1	3.21	23%	0.47	28%	13.14	54%	134.50	56%
103hhh	TURKEY GOBBLER CREEK	7,548.8	3.06	14%	0.47	24%	9.37	6%	119.33	25%
103fff	TURKEY HEN CREEK	5,942.3	2.90	8%	0.45	15%	9.63	10%	121.00	28%
105a	TURTLE CREEK	17,599.8	3.30	26%	0.45	13%	13.00	52%	141.44	67%
103h	UNNAMED CREEK	1,584.2	3.16	21%	0.47	27%	9.97	18%	113.13	11%
105e	UNNAMED STREAM 1	206.8	3.24	25%	0.42	2%	13.92	58%	147.25	70%
103s	UNNAMED STREAM 2	775.9	**6.09**	**86%**	**0.90**	**87%**	12.98	51%	162.81	78%
103kk	WARD MILL CREEK	1,790.2	4.53	61%	0.63	59%	13.38	55%	127.35	39%
103e	WATSON BAY BRANCH	2,848.2	5.35	78%	0.76	78%	14.52	64%	132.78	51%
103zz	WILKENSON CREEK	2,337.2	3.81	42%	0.55	43%	14.17	61%	131.62	48%
103xx	WILLIAMS BRANCH	1,478.6	5.25	76%	0.74	74%	18.85	77%	**167.51**	**80%**
103c	YELLOW RIVER	58,876.0	3.93	43%	0.54	42%	13.09	52%	109.64	6%
	Total	**600,219.8**	**4.29**	**56%**	**0.62**	**56%**	**15.47**	**69%**	**142.14**	**67%**

7.4.2.1 Total Nitrogen

Nitrogen is a critical nutrient for aquatic plant growth in freshwater environments, and is usually the limiting nutrient in estuarine or marine ecosystems. Excessive amounts of nitrogen contribute to eutrophication and changes in water quality that adversely affect aquatic ecosystems.

The pollutant loading model estimated that over 2.5 million pounds of TN per year was present in stormwater runoff from all existing land uses within Okaloosa County. As shown in **Figure 7-4** TN loadings are primarily associated with agricultural lands in the northern part of the County, the most heavily urbanized areas throughout the County, and spot locations within Eglin AFB.

Table 7.3 lists the 30 sub-basins in the top 20th-percentile group (shown in bold) that generated between 5.3 and 8.4 pounds of TN per acre per year. The sub-basins belonging to the 80th-percentile or greater group appeared in urbanized areas, and in areas with large amounts of agricultural land as previously mentioned. Relatively undisturbed forest lands or silvicultural areas did not generate large amounts of TN per acre.

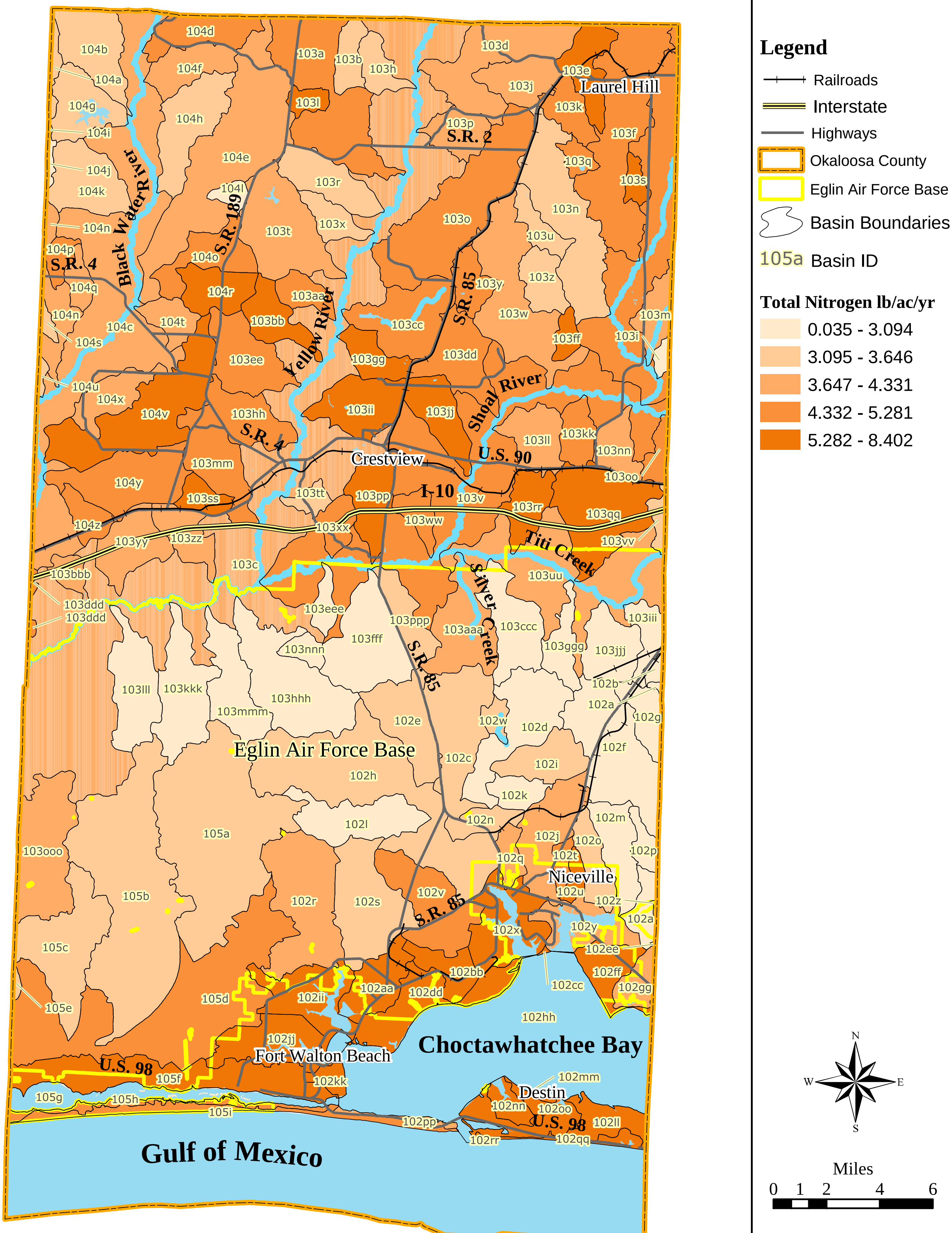
Five or more sub-basins with high annual per acre TN loadings were listed for poor water quality in the 303(d) reports: Boggy Bayou, one or more sub-basins with Direct Runoff to Bay, Indian Bayou, Joes Bayou, and Juniper Creek.

7.4.2.2 Total Phosphorus

Phosphorus is essential to the growth of aquatic plants, and is usually the limiting nutrient in freshwater ecosystems (Wetzel, 1975). Too much phosphorus in the water column stimulates excessive growth of algae and other aquatic plants, contributing to artificially accelerated eutrophication and diminished water quality in lakes and streams.

The pollutant loading model estimated 370,756 pounds of TP per year was present in stormwater runoff from all existing land uses within Okaloosa County. As shown in **Figure 7-5** TP loadings are nearly identical to the TN loadings shown in Figure 7-4. TP loadings are associated with agricultural lands in the northern part of the County, the most heavily urbanized areas throughout the County, and spot locations within Eglin AFB.

Table 7.3 lists the 30 sub-basins in the top 20th-percentile group (shown in bold) that generated between 0.76 and 1.31 pounds of TP per acre per year. The sub-basins belonging to the 80th-percentile or greater group appeared in urbanized areas, and in areas with large amounts of agricultural land as previously mentioned. Relatively undisturbed forest lands or silvicultural areas did not generate large amounts of TP per acre.

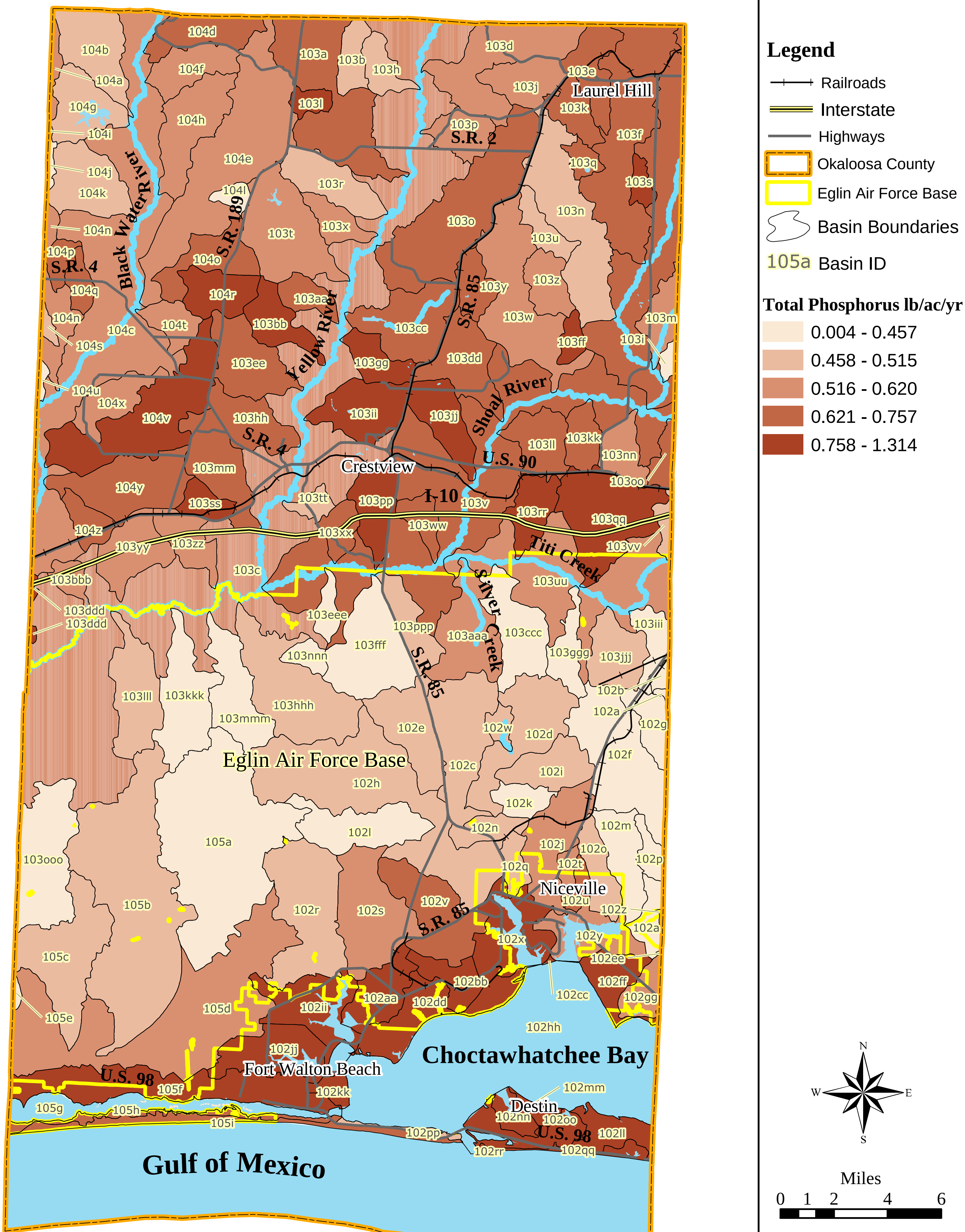


Master Stormwater Management Plan

Pollutant Loading by Basin, Normalized by Area
Existing Land Use
(Total Nitrogen)

Figure 7-4

fig_7-04.mxd - 03/19/2003



Master Stormwater Management Plan	**Pollutant Loading by Basin, Normalized by Area** **Existing Land Use** **(Total Phosphorus)**	**Figure 7-5** HDR

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Five or more sub-basins with high annual per acre TP loadings were listed for poor water quality in the 303(d) reports: Boggy Bayou, one or more sub-basins with Direct Runoff to Bay, Indian Bayou, Joes Bayou, and Juniper Creek.

7.4.2.3 Biochemical Oxygen Demand

BOD is not actually a stormwater pollutant constituent, but a measure of the potential for a variety of pollutants to consume oxygen in surface waters through biological respiration or chemical oxidation. For example, many oils and greases that enter surface waters from roadways and parking lots may be metabolized by bacteria in a receiving water body; as oxygen is consumed by bacteria fed by the hydrocarbons, the dissolved oxygen concentration of the water body will fall. Other runoff constituents may chemically combine with oxygen in water to form new compounds, and thereby remove oxygen from the water column. Reduction in dissolved oxygen adversely affects the desirable aquatic flora and fauna that depend on high oxygen concentrations to maintain an active metabolism, while encouraging nuisance species of bacteria and invertebrates.

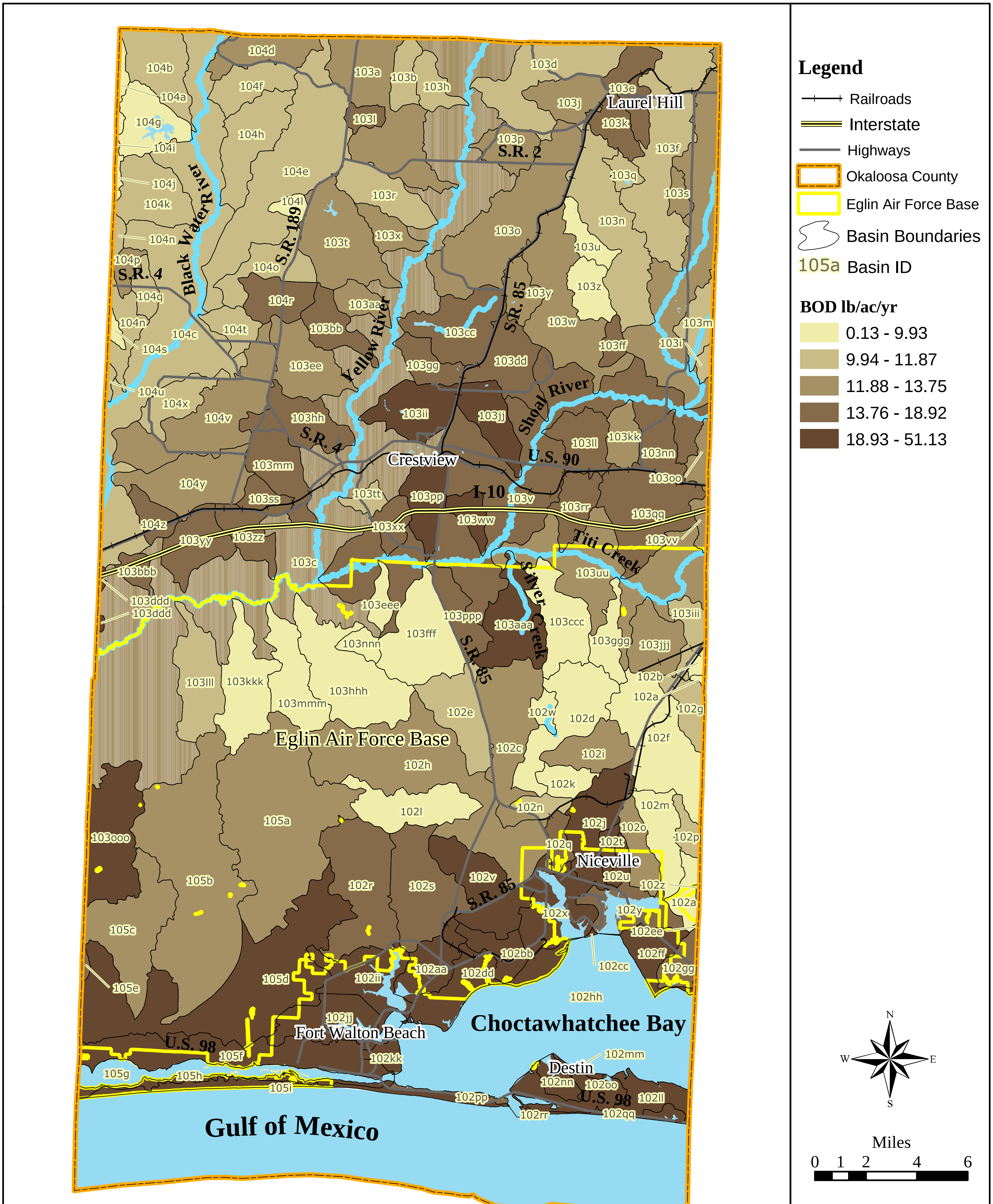
The pollutant loading model estimated nearly 9.2 million pounds of BOD per year loading to surface waters in Okaloosa County. As shown in **Figure 7-6,** unlike the pattern for TN and TP, the agricultural lands in the northern part of the County generated only moderate BOD loadings. However, high concentrations of BOD were most frequently associated with urban land uses and spot locations within Eglin AFB.

Table 7.3 lists the 30 sub-basins in the top 20th-percentile group (shown in bold) that generated between 18.91 and 51.13 pounds of BOD per acre per year. The sub-basins belonging to the 80th-percentile or greater group appeared in urbanized areas, or in areas with large amounts of transportation and utility use.

Five of the sub-basins predicted to have high annual per acre BOD loadings were listed for poor water quality in the 303(d) reports: Boggy Bayou, all the sub-basins with Direct Runoff to Bay, Indian Bayou, Joes Bayou, and Juniper Creek.

7.4.2.4 Total Suspended Solids

TSS is a measure of the material that is carried suspended in the water column, and not chemically dissolved into the water. TSS consists of particles of varying sizes. The larger, heavier particles may fall out of the water column relatively quickly after being introduced from stormwater runoff; these components of TSS contribute to the physical covering of aquatic flora and fauna, and direct destruction of benthic habitats. Smaller particles may stay suspended indefinitely, contributing to the turbidity of the water. High turbidity decreases light penetration



Master Stormwater Management Plan

Pollutant Loading by Basin, Normalized by Area
Existing Land Use
(Biochemical Oxygen Demand)

Figure 7-6

fig_7-06.mxd - 03/19/2003

to the water column, reducing the available light for photosynthesis and thereby contributing to lower dissolved oxygen concentrations.

The pollutant loading model estimated over 85 million pounds of TSS per year loading to surface waters in Okaloosa County. As shown in **Figure 7-7**, unlike the pattern for TN, TP, and BOD, where generally low pollutant loadings throughout most of the County were punctuated with small areas of very high loadings, moderately high TSS pollutant loadings appeared frequently associated with forested, silvicultural and agricultural lands throughout nearly the entire County. Nonetheless, the highest TSS loadings were associated with urban uses, particularly Transportation/Utilities, Industrial, Institutional and High Density Residential areas. Wetland areas, such as those associated with major rivers, were predicted to be very low contributors of TSS pollution.

Table 7.3 lists the 30 sub-basins in the top 20th-percentile group (shown in bold) that generated between 162 and 365 pounds of TSS per acre per year. The sub-basins belonging to the 80th-percentile or greater group appeared in urbanized areas, or in areas with large amounts of transportation and utility use. Relatively undisturbed forest lands or silvicultural areas generated moderate amounts of TSS per acre, while the lowest per acre loadings were associated with wetland dominated areas.

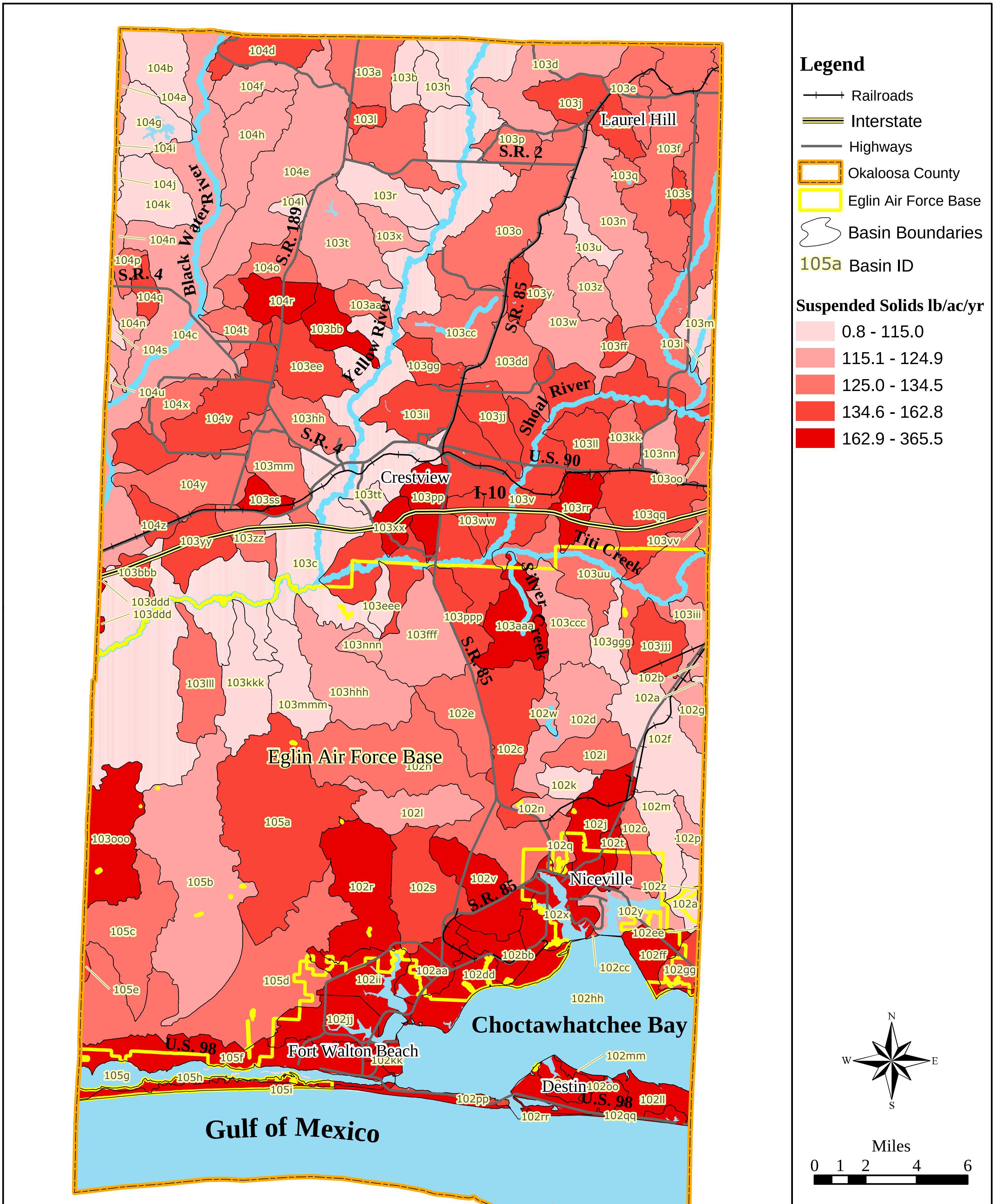
Five of the sub-basins predicted to have high annual per acre TSS loadings were listed for poor water quality in the 303(d) reports: Boggy Bayou, nearly all the sub-basins with Direct Runoff to Bay, Indian Bayou, Joes Bayou and Juniper Creek.

7.4.3 Future Land Use

Figure 7-3 shows the County's future land use based on the 15 classifications used for the pollutant loading analysis. Table 2.2 shows the percentage of each of these classifications within the County.

7.4.4 Annual Pollutant Loadings By Sub-basin, Future Land Use

Table 7.4 lists the total annual pollutant loadings for each sub-basin (normalized by area) generated by future land use estimated for Okaloosa County for each of four pollutants, in pounds of pollutant per acre, per year (lbs/acre/year): As for the existing land use scenario, the percentile rank of each sub-basin was calculated for each annual pollutant loading value for each sub-basin. The percentile value for a particular sub-basin represented the percentage of the rank-ordered sub-basins that had a lower pollutant loading value. For example, a sub-basin with a percentile value of 80 percent had a pollutant loading greater than that of 80 percent of the other sub-basins in the County.



Master Stormwater Management Plan

Pollutant Loading by Basin, Normalized by Area
Existing Land Use
(Total Suspended Solids)

Figure 7-7

fig_7-07.mxd - 03/19/2003

Table 7.4
Future Land Use Annual Pollutant Loadings By Basin
(normalized by basin area)

Basin ID	Basin Name	Acres	Total Nitrogen		Total Phosphorus		Biochemcial Oxygen Demand		Total Suspended Solids	
			lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile
104z	ADAMS MILL CREEK	1,461.9	4.88	71%	0.68	68%	16.63	71%	130.27	50%
102bb	AIRPORT DRAIN	1,976.0	**6.94**	**89%**	**0.92**	**87%**	**51.13**	**97%**	**365.46**	**99%**
102n	ANDERSON BRANCH	946.2	3.17	23%	0.48	28%	10.70	25%	124.42	37%
103mm	BAGGETT CREEK	4,184.4	5.03	76%	0.71	75%	14.02	60%	127.82	43%
103l	BAILY BRANCH	841.5	**6.48**	**87%**	**0.91**	**86%**	14.82	66%	153.83	74%
104l	BARREL BRANCH	686.3	2.94	13%	0.44	10%	9.64	12%	111.12	9%
103x	BEAR BRANCH	1,634.7	3.65	37%	0.52	38%	11.89	37%	117.58	23%
103ll	BEAR CREEK	2,919.7	4.33	55%	0.63	58%	15.05	67%	141.17	67%
104n	BEAVER CREEK	2,454.6	4.17	52%	0.61	54%	11.36	32%	124.05	36%
102p	BEE BRANCH	987.1	2.81	5%	0.43	7%	10.53	24%	123.07	34%
103dd	BENDS CREEK	4,019.9	4.80	67%	0.64	58%	16.40	71%	92.38	2%
104s	BIG BRANCH	375.4	3.12	19%	0.46	19%	9.82	15%	103.89	4%
103d	BIG CREEK	6,310.7	3.97	45%	0.58	45%	11.57	34%	120.78	30%
103qq	BIG FORK	6,801.7	5.11	76%	0.75	79%	14.77	65%	152.17	72%
103a	BIG HORSE CREEK	8,660.7	4.44	57%	0.65	60%	12.62	47%	132.16	54%
104c	BLACKWATER RIVER	23,989.0	4.12	51%	0.60	50%	11.24	31%	117.68	24%
103ggg	BLUE SPRING CREEK	1,933.0	2.76	3%	0.43	5%	9.07	2%	114.61	17%
102x	BOGGY BAYOU	3,903.9	**8.92**	**97%**	**1.28**	**95%**	**51.81**	**98%**	**340.64**	**98%**
104d	BOGGY HOLLOW CREEK	3,005.5	4.71	65%	0.69	69%	12.38	43%	138.22	64%
103ooo	BOILING CREEK	6,537.5	3.79	41%	0.43	4%	19.32	78%	**194.95**	**85%**
104y	BONE CREEK	5,475.4	4.46	58%	0.65	63%	12.04	39%	129.97	49%
103aa	BUCKHANNON BRANCH	1,507.3	5.22	78%	0.74	78%	13.11	52%	128.08	45%
103jjj	BULL CREEK	3,160.0	3.03	14%	0.46	20%	13.43	56%	140.06	65%
104k	BULL PEN BRANCH	4,157.2	3.35	30%	0.49	32%	10.37	21%	111.13	10%
103j	CAMBELLS MILL CREEK	2,707.3	3.97	46%	0.59	50%	12.08	41%	131.94	54%
103bbb	CANOE CREEK	1,033.0	4.28	54%	0.62	55%	15.18	69%	149.35	71%
102hh	CHOCTAWHATCHEE BAY	165.3	**6.55**	**88%**	**0.86**	**83%**	**25.74**	**86%**	153.61	73%
102jj	CINCO BAYOU	3,884.3	**9.01**	**97%**	**1.37**	**98%**	**48.43**	**96%**	**314.13**	**96%**
103ff	CLEAR CREEK	878.3	**5.69**	**82%**	**0.82**	**82%**	14.66	65%	155.15	76%
103bb	COTTON CREEK	2,520.0	**6.36**	**86%**	**0.93**	**88%**	17.37	72%	**174.61**	**81%**
103u	CYPRESS POND BRANCH	1,501.7	3.15	21%	0.47	26%	9.91	17%	112.36	11%
104j	DANLEY BRANCH	313.4	2.92	12%	0.45	15%	9.60	10%	112.87	13%
103gg	DAVIS MILL CREEK	1,978.2	5.30	79%	0.72	76%	16.30	70%	107.82	8%
103t	DEADFALL CREEK	6,417.9	3.80	41%	0.55	44%	11.94	38%	123.28	35%
102rr	DESTIN HARBOR	19.1	**7.37**	**91%**	**1.02**	**90%**	**30.70**	**89%**	**206.51**	**88%**
102cc	DIRECT RUNOFF TO BAY 1	40.4	**6.08**	**84%**	**0.87**	**84%**	**25.09**	**85%**	147.20	69%
102dd	DIRECT RUNOFF TO BAY 2	4,331.5	**7.95**	**93%**	**1.36**	**97%**	**40.44**	**93%**	**287.18**	**94%**
102kk	DIRECT RUNOFF TO BAY 3	792.0	**8.42**	**95%**	**1.17**	**93%**	**44.34**	**95%**	**272.99**	**93%**
102mm	DIRECT RUNOFF TO BAY 4	258.5	**7.76**	**93%**	**1.11**	**93%**	**38.48**	**91%**	**221.35**	**89%**
102ll	DIRECT RUNOFF TO BAY 5	2,115.4	**9.36**	**99%**	**1.41**	**100%**	**59.15**	**100%**	**398.52**	**100%**

Table 7.4
Future Land Use Annual Pollutant Loadings By Basin
(normalized by basin area)

Basin ID	Basin Name	Acres	Total Nitrogen		Total Phosphorus		Biochemcial Oxygen Demand		Total Suspended Solids	
			lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile
102ff	DIRECT RUNOFF TO BAY 6	2,340.4	**8.12**	**95%**	**1.31**	**96%**	**38.53**	**91%**	**240.13**	**91%**
105f	DIRECT RUNOFF TO BAY 7	7,973.0	**8.04**	**94%**	**1.20**	**94%**	**43.98**	**94%**	**300.33**	**95%**
105h	DIRECT RUNOFF TO BAY 8	1,397.9	4.85	69%	0.69	69%	**21.72**	**80%**	**170.29**	**80%**
102pp	DIRECT RUNOFF TO BAY 9	477.2	4.01	47%	0.50	34%	19.13	77%	161.39	77%
102qq	DIRECT RUNOFF TO GULF 1	882.5	**8.52**	**96%**	**1.27**	**95%**	**44.47**	**95%**	**300.21**	**95%**
105i	DIRECT RUNOFF TO GULF 2	2,152.7	4.78	67%	0.63	56%	**27.06**	**87%**	**234.63**	**90%**
104i	DOGWOOD BRANCH	38.6	2.67	1%	0.42	2%	8.89	1%	118.00	27%
102ee	EAGLE CREEK	24.0	2.67	0%	0.42	0%	8.89	0%	117.99	26%
105d	EAST RIVER BAY	17,984.5	4.55	62%	0.61	54%	19.30	78%	130.73	50%
102b	EXLINE CREEK	386.9	2.70	2%	0.42	3%	8.95	2%	116.72	20%
102ii	GARNIER BAYOU	3,715.7	**7.59**	**92%**	**1.11**	**92%**	**42.43**	**93%**	**285.05**	**93%**
102s	GARNIER CREEK	6,275.2	3.54	35%	0.52	39%	16.28	69%	160.00	76%
103eee	GOPHER CREEK	1,022.3	2.92	11%	0.44	9%	9.43	9%	106.82	6%
103k	GREEN BRANCH	1,923.8	4.99	74%	0.72	78%	13.98	60%	139.17	65%
103tt	GULLY BRANCH	836.2	2.89	8%	0.39	0%	9.25	6%	50.09	0%
103ee	GUM CREEK 1	4,165.2	4.90	73%	0.71	76%	12.39	43%	135.43	60%
103vv	GUM CREEK 2	660.1	3.14	20%	0.47	23%	12.75	48%	133.00	56%
103ccc	HONEY CREEK	4,129.9	2.76	2%	0.43	4%	9.27	6%	116.83	21%
103f	HORSEHEAD CREEK	9,973.2	4.63	65%	0.67	67%	11.87	36%	133.39	58%
104g	HURRICANE CREEK	3,421.2	3.58	36%	0.49	32%	9.77	13%	105.18	4%
102oo	INDIAN BAYOU	2,746.5	**9.55**	**100%**	**1.41**	**99%**	**52.70**	**99%**	**339.56**	**97%**
102nn	JOES BAYOU	1,043.5	**9.02**	**98%**	**1.36**	**97%**	**50.45**	**97%**	**329.83**	**97%**
103ddd	JULIAN MILL CREEK	246.1	4.77	66%	0.71	73%	**23.59**	**82%**	**204.94**	**87%**
103n	JUNIPER CREEK 1	7,833.6	3.35	29%	0.49	33%	10.42	22%	117.11	21%
103pp	JUNIPER CREEK 2	2,806.5	**7.21**	**90%**	**1.07**	**91%**	**38.66**	**92%**	**252.35**	**91%**
102c	JUNIPER CREEK 3	6,523.0	3.13	19%	0.46	21%	12.83	50%	134.50	60%
103ww	KING BRANCH	1,369.5	**5.57**	**81%**	**0.76**	**80%**	**25.08**	**84%**	147.53	71%
103z	KIRKLAND BRANCH	2,123.1	3.52	34%	0.53	40%	9.93	17%	119.77	29%
103oo	LAIRD MILL CREEK	1,014.3	4.89	72%	0.70	72%	12.60	46%	129.09	47%
104w	LIGHTER KNOT CREEK	1.2	4.02	47%	0.51	35%	12.07	40%	53.78	0%
102r	LIGHTWOOD KNOT CREEK	7,649.3	3.68	38%	0.51	36%	18.45	74%	**174.89**	**82%**
103b	LITTLE HORSE CREEK	1,604.0	3.51	34%	0.51	36%	10.85	26%	112.38	12%
102f	LITTLE ROCKY CREEK	7,613.4	2.82	6%	0.43	8%	9.40	8%	115.04	18%
102gg	LITTLE TROUT CREEK	1,533.1	4.56	63%	0.68	67%	19.08	76%	152.88	73%
105b	LIVE OAK CREEK	18,045.8	3.41	32%	0.47	21%	12.21	41%	121.89	33%
104q	LONG BRANCH 1	1,085.0	4.33	54%	0.64	59%	11.47	32%	137.09	63%
102k	LONG BRANCH 2	1,426.1	2.87	7%	0.44	11%	9.21	5%	114.87	17%
103i	LONG CREEK 1	451.8	3.09	17%	0.45	16%	10.45	23%	106.36	5%
103rr	LONG CREEK 2	2,388.7	**7.31**	**91%**	**1.06**	**91%**	15.09	68%	**172.66**	**80%**
102m	LONG CREEK 3	3,033.9	2.88	8%	0.44	10%	9.66	13%	113.41	15%

Table 7.4
Future Land Use Annual Pollutant Loadings By Basin
(normalized by basin area)

Basin ID	Basin Name	Acres	Total Nitrogen		Total Phosphorus		Biochemcial Oxygen Demand		Total Suspended Solids	
			lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile	lbs/ac/year	Percentile
102z	LONG CREEK 4	71.4	3.27	26%	0.46	19%	10.29	21%	89.55	1%
103nnn	LOST BOY POND OUTLET	898.7	2.97	13%	0.45	17%	9.18	4%	117.42	22%
103q	MACK BRANCH	839.2	4.53	60%	0.65	63%	11.21	30%	129.67	48%
103kkk	MALONE CREEK	5,008.8	2.79	4%	0.43	6%	9.17	3%	113.19	14%
104h	MARE CREEK 1	4,628.6	3.55	36%	0.52	39%	10.52	23%	117.86	25%
103nn	MARE CREEK 2	2,052.8	4.03	48%	0.57	45%	13.77	58%	116.55	19%
103ii	MATHISON CREEK	3,839.2	**5.31**	**80%**	0.72	77%	**22.83**	**80%**	128.07	44%
103lll	METTS CREEK	4,506.1	3.06	15%	0.47	23%	11.58	34%	129.30	47%
104u	MIDDLE CREEK 1	262.7	3.24	25%	0.46	17%	10.24	20%	90.71	2%
103mmm	MIDDLE CREEK 2	4,144.8	2.82	6%	0.43	8%	9.19	4%	113.61	15%
102g	MIDDLE ROCKY CREEK	1,900.6	2.91	10%	0.44	12%	9.93	18%	116.56	19%
103p	MILL CREEK 1	2,129.2	3.95	44%	0.58	48%	12.28	42%	133.45	58%
103hh	MILL CREEK 2	3,154.3	4.23	53%	0.58	49%	14.38	64%	109.48	8%
102q	MILL CREEK 3	1,125.8	3.74	39%	0.47	27%	14.35	63%	125.74	39%
104m	MINCY BRANCH	0.4	2.67	0%	0.42	1%	8.89	0%	117.99	26%
104p	MUDDY BRANCH	980.4	4.57	63%	0.66	65%	12.01	39%	128.60	45%
103o	MURDER CREEK	10,346.2	4.52	60%	0.65	64%	12.57	45%	130.74	51%
104o	NARROWS CREEK	3,188.2	4.39	56%	0.65	61%	11.81	35%	132.90	55%
102i	NINEMILE CREEK	2,914.5	3.16	21%	0.47	26%	12.42	44%	133.00	56%
104f	OAK CREEK	2,726.2	4.12	50%	0.60	52%	11.87	36%	124.57	38%
104e	PANTHER CREEK	12,764.9	3.96	45%	0.58	46%	11.17	29%	121.94	34%
102e	PARRISH CREEK	4,228.6	3.10	18%	0.47	24%	11.19	30%	127.40	43%
103eee	PEARL CREEK	2,345.1	3.32	28%	0.48	30%	14.38	63%	140.34	67%
104v	PENNY CREEK	8,538.2	**5.31**	**80%**	**0.77**	**81%**	13.25	54%	140.11	66%
102w	PINE LOG CREEK 1	765.4	3.17	23%	0.49	30%	9.45	10%	121.10	32%
103m	PINE LOG CREEK 2	1,914.5	3.78	40%	0.55	43%	11.50	33%	114.20	16%
103jj	PINEY WOODS CREEK	3,021.1	**6.30**	**86%**	**0.89**	**85%**	**29.37**	**89%**	**195.68**	**86%**
103r	POLLEY CREEK	3,368.6	3.39	31%	0.49	31%	10.90	26%	107.26	6%
103g	POND CREEK	12,043.6	4.85	69%	0.70	70%	12.80	50%	131.79	52%
104x	POPLAR HEAD	2,475.9	3.90	43%	0.58	47%	11.04	28%	125.79	40%
102aa	POQUITO BAYOU	2,831.6	**5.61**	**82%**	**0.85**	**82%**	**32.73**	**90%**	**252.69**	**92%**
103w	POVERTY CREEK	7,681.4	4.22	52%	0.60	53%	12.51	45%	120.79	31%
105c	PRAIRIE CREEK	6,864.9	3.37	30%	0.46	18%	12.71	47%	129.06	46%
104r	PYRON SPRING BRANCH	2,665.1	**6.60**	**89%**	**0.97**	**89%**	14.93	67%	166.71	79%
103iii	RAMER CREEK	1,649.0	2.91	10%	0.45	13%	10.93	27%	124.92	39%
104t	RED WASH BRANCH	1,673.0	4.04	50%	0.60	52%	11.13	28%	131.93	53%
103ss	RESERVOIR OUTLET	1,286.5	**5.92**	**83%**	**0.87**	**84%**	18.95	76%	**182.48**	**83%**
104b	ROCK CREEK	4,185.1	3.48	33%	0.51	37%	10.12	19%	112.05	10%
102y	ROCKY BAYOU	2,465.1	**6.24**	**85%**	**0.95**	**89%**	**28.59**	**88%**	**175.12**	**82%**
102a	ROCKY CREEK	1,560.3	3.09	17%	0.45	13%	9.80	15%	99.12	3%

Table 7.4
Future Land Use Annual Pollutant Loadings By Basin
(normalized by basin area)

Basin ID	Basin Name	Acres	Total Nitrogen lbs/ac/year	Total Nitrogen Percentile	Total Phosphorus lbs/ac/year	Total Phosphorus Percentile	Biochemcial Oxygen Demand lbs/ac/year	Biochemcial Oxygen Demand Percentile	Total Suspended Solids lbs/ac/year	Total Suspended Solids Percentile
102l	ROGUE CREEK	4,290.0	2.78	4%	0.43	6%	9.39	8%	119.87	30%
103y	RUM STILL BRANCH	1,842.9	5.22	77%	**0.76**	**80%**	13.83	58%	144.78	69%
102u	SANDERS BRANCH	702.2	5.27	78%	0.71	73%	**23.45**	**82%**	136.63	63%
105g	SANTA ROSA SOUND	137.9	4.57	64%	0.63	57%	18.22	73%	136.20	62%
102t	SHAW STILL BRANCH	704.6	4.86	70%	0.65	62%	**26.71**	**86%**	**185.98**	**84%**
103v	SHOAL RIVER	27,342.0	4.83	68%	0.67	66%	18.68	75%	136.02	61%
103cc	SILVER CREEK 1	5,008.0	4.91	73%	0.70	71%	13.73	57%	127.01	42%
103aaa	SILVER CREEK 2	4,816.5	3.74	39%	0.54	41%	20.13	79%	**183.57**	**84%**
104a	SWEETWATER CREEK	645.5	3.47	32%	0.53	41%	9.82	16%	123.45	36%
102j	SWIFT CREEK	4,521.5	4.54	61%	0.64	60%	**24.57**	**83%**	**195.07**	**86%**
102d	TENMILE CREEK	5,484.7	3.08	16%	0.47	22%	9.79	14%	117.44	23%
103uu	TITI CREEK	11,731.0	4.04	49%	0.58	47%	13.34	55%	126.33	41%
102v	TOMS CREEK	5,123.6	4.86	71%	0.71	74%	**24.82**	**84%**	**211.85**	**89%**
103yy	TRAWICK CREEK	3,397.4	4.50	58%	0.66	65%	18.10	73%	163.91	78%
102h	TURKEY CREEK 1	15,054.2	3.31	28%	0.50	34%	12.77	49%	133.38	57%
102o	TURKEY CREEK 2	2,179.1	3.24	24%	0.48	29%	13.24	54%	133.80	59%
103hhh	TURKEY GOBBLER CREEK	7,548.8	3.06	15%	0.47	25%	9.37	7%	119.33	28%
103fff	TURKEY HEN CREEK	5,942.3	2.90	9%	0.45	15%	9.63	11%	121.00	32%
105a	TURTLE CREEK	17,599.8	3.30	27%	0.45	14%	13.00	52%	141.44	68%
103h	UNNAMED CREEK	1,584.2	3.16	22%	0.47	28%	9.96	19%	113.07	13%
105e	UNNAMED STREAM 1	206.8	3.24	26%	0.42	2%	13.92	59%	147.24	70%
103s	UNNAMED STREAM 2	775.9	**6.09**	**84%**	**0.90**	**86%**	12.98	51%	162.81	78%
103kk	WARD MILL CREEK	1,790.2	4.51	59%	0.63	56%	13.35	56%	126.45	41%
103e	WATSON BAY BRANCH	2,848.2	5.01	75%	0.70	71%	14.05	61%	118.22	28%
103zz	WILKENSON CREEK	2,337.2	3.81	42%	0.55	43%	14.17	62%	131.62	52%
103xx	WILLIAMS BRANCH	1,478.6	4.40	56%	0.60	51%	**23.12**	**81%**	154.36	75%
103c	YELLOW RIVER	58,876.0	3.92	43%	0.54	42%	13.21	53%	107.41	7%
	Grand Total	**600,219.8**	**4.31**	**54%**	**0.62**	**55%**	**15.88**	**69%**	**142.37**	**68%**

For the future land use scenario an additional BMP assumption was made. All contiguous areas of future urban land use that were 10 acres or greater in size were assumed to employ 0.5 inches of retention, with a pollutant removal of 80 percent. Consequently, the relatively small amount of additional urbanization proposed for the County under the future land use scenario resulted in only minor increases in pollutant loadings. The total pollutant loading increases from the ELU scenario to the FLU scenario for TN, TP, BOD, and TSS were estimated to be 0.58%, 0.60%, 2.65% and 0.16%, respectively.

7.4.4.1 Total Nitrogen

The pollutant loading model estimated that nearly 2.6 million pounds of TN per year will be generated in the stormwater runoff from all future land uses within Okaloosa County. As shown in **Figure 7-8** the FLU scenario produced similar results to the ELU scenario attributing TN loadings to agricultural land in the northern part of the County, the most heavily urbanized areas throughout the County, and spot locations within Eglin AFB. It should be noted that some sub-basins showed future decreases in TN loadings. For example, the pollutant loading model estimated that Adams Mill Creek (Sub-basin 104z) generated 7,721 lbs of TN under the ELU condition, but only 7,139 lbs of TN for the FLU scenario. The reason for this decrease is:

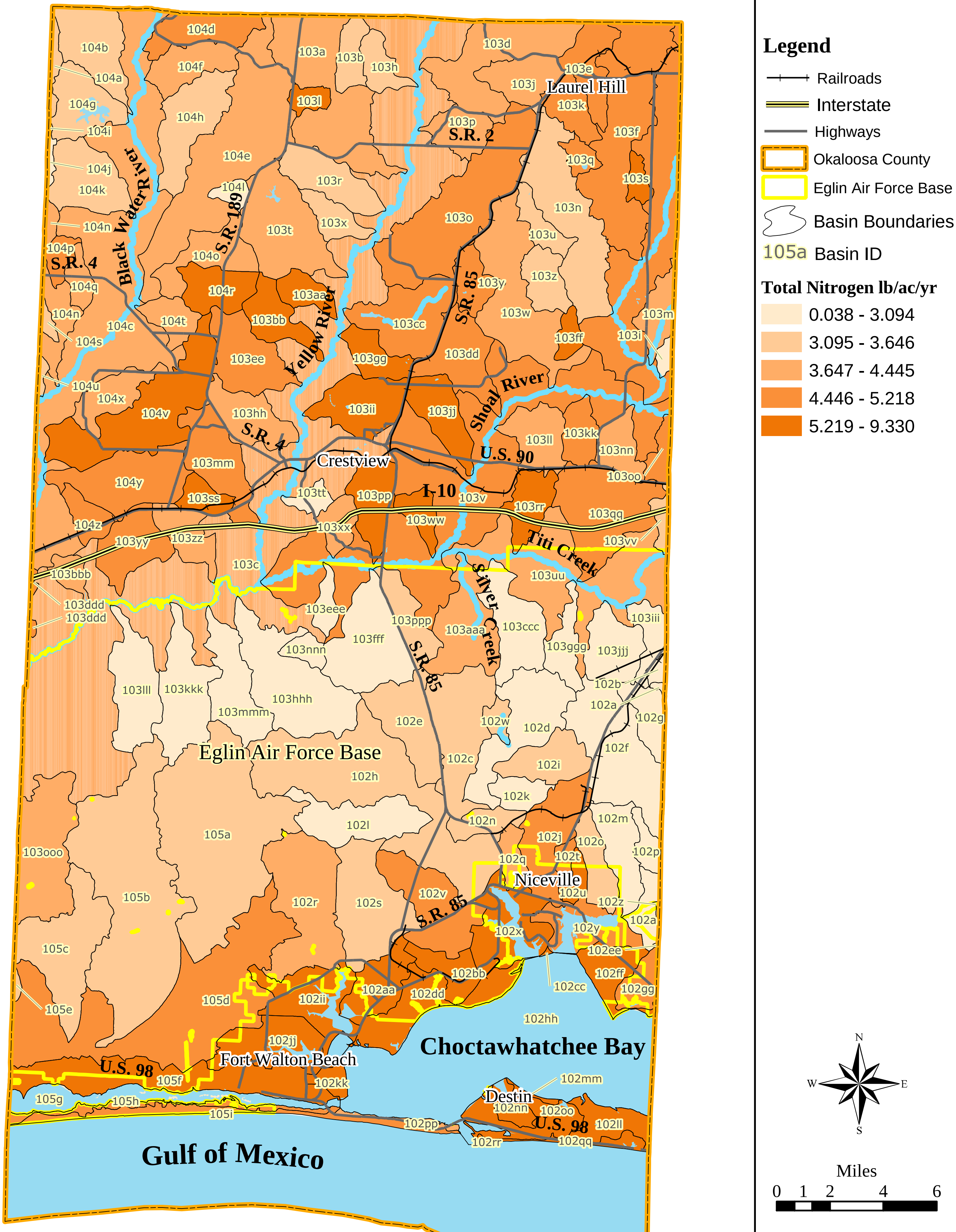
- Most of the urbanization planned for the sub-basin involves the development of low density residences in areas that are currently cropland/pasture. Because cropland/pasture has higher TN loadings than low density residences, the proposed conversion contributed to lower overall TN loadings for the sub-basin.

- All new urban land uses measuring 10 acres or larger in size were assumed to include stormwater treatment that would remove 80 percent of TN from the predicted runoff.

Table 7.4 lists the 30 sub-basins in the top 20th-percentile group (shown in bold) that generated between 5.2 and 9.3 pounds of TN per acre per year. Similar to the ELU scenario, the sub-basins belonging to the 80th-percentile or greater group for the FLU scenario appeared in urbanized areas, and in areas with large amounts of agricultural land as previously mentioned. Relatively undisturbed forest lands or silvicultural areas did not generate large amounts of TN per acre.

7.4.4.2 Total Phosphorus

The pollutant loading model estimated 372,984 pounds of TP per year will be generated in the stormwater runoff from all future land uses within Okaloosa County. As shown in **Figure 7-9** the FLU scenario produced nearly identical results to the ELU scenario attributing TP loadings to agricultural land in the northern part of the County, the most heavily urbanized areas throughout



Master Stormwater Management Plan	**Pollutant Loading by Basin, Normalized by Area** **Future Land Use** **(Total Nitrogen)**	**Figure 7-8**

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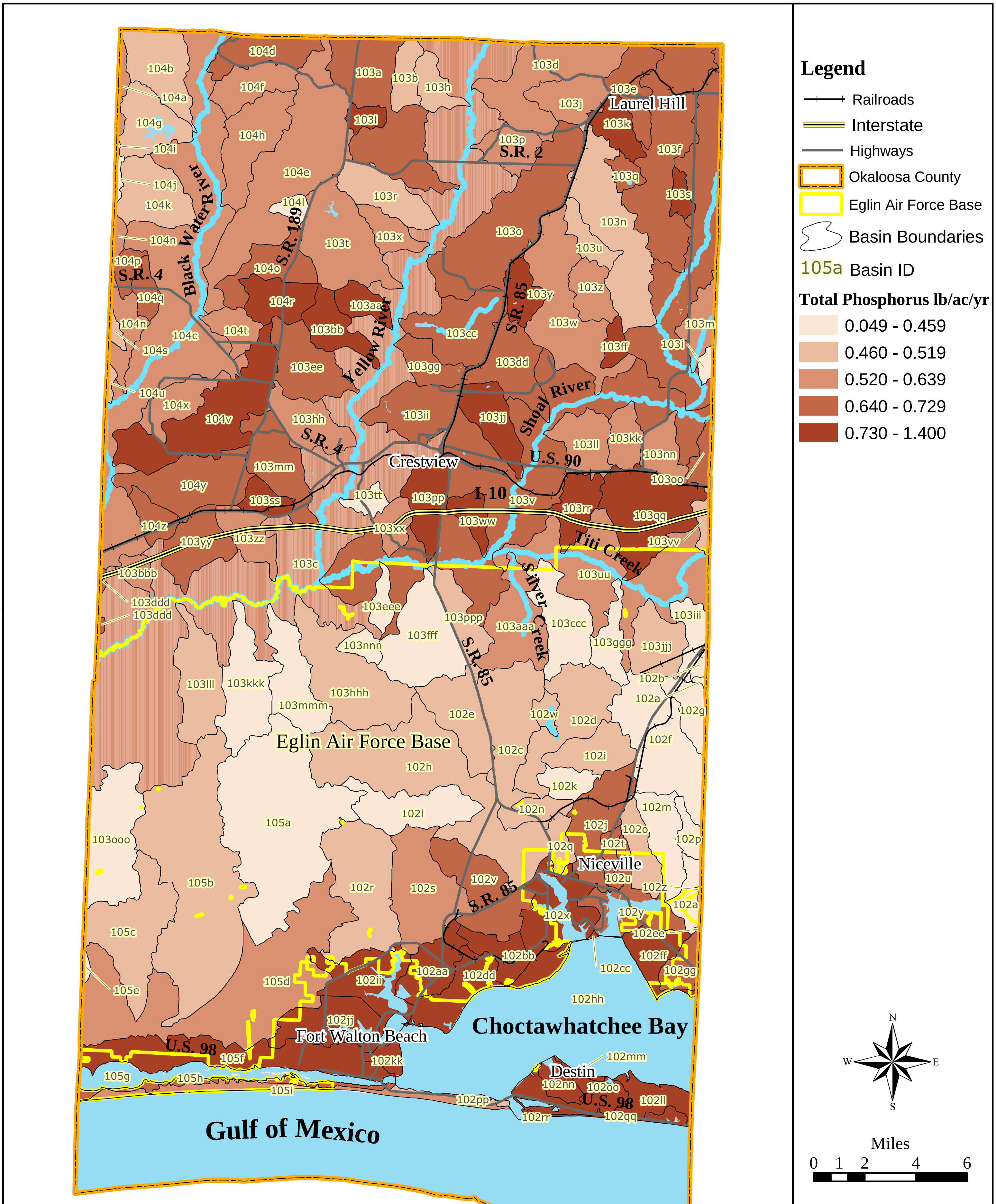


Master Stormwater Management Plan

Pollutant Loading by Basin, Normalized by Area
Future Land Use
(Total Phosphorus)

Figure 7-9

fig_7-09.mxd - 03/19/2003

the County, and spot locations within Eglin AFB. As with TN, some sub-sub-basins showed reductions in TP loadings between ELU and FLU scenarios.

Table 7.4 lists the 30 sub-basins in the top 20th-percentile group (shown in bold) that generated between 0.72 and 1.41 pounds of TP per acre per year. Similar to the ELU scenario, the sub-basins belonging to the 80th-percentile or greater group appeared in urbanized areas, and in areas with large amounts of agricultural land as previously mentioned. Relatively undisturbed forest lands or silvicultural areas did not generate large amounts of TP per acre.

7.4.4.3 Biochemical Oxygen Demand

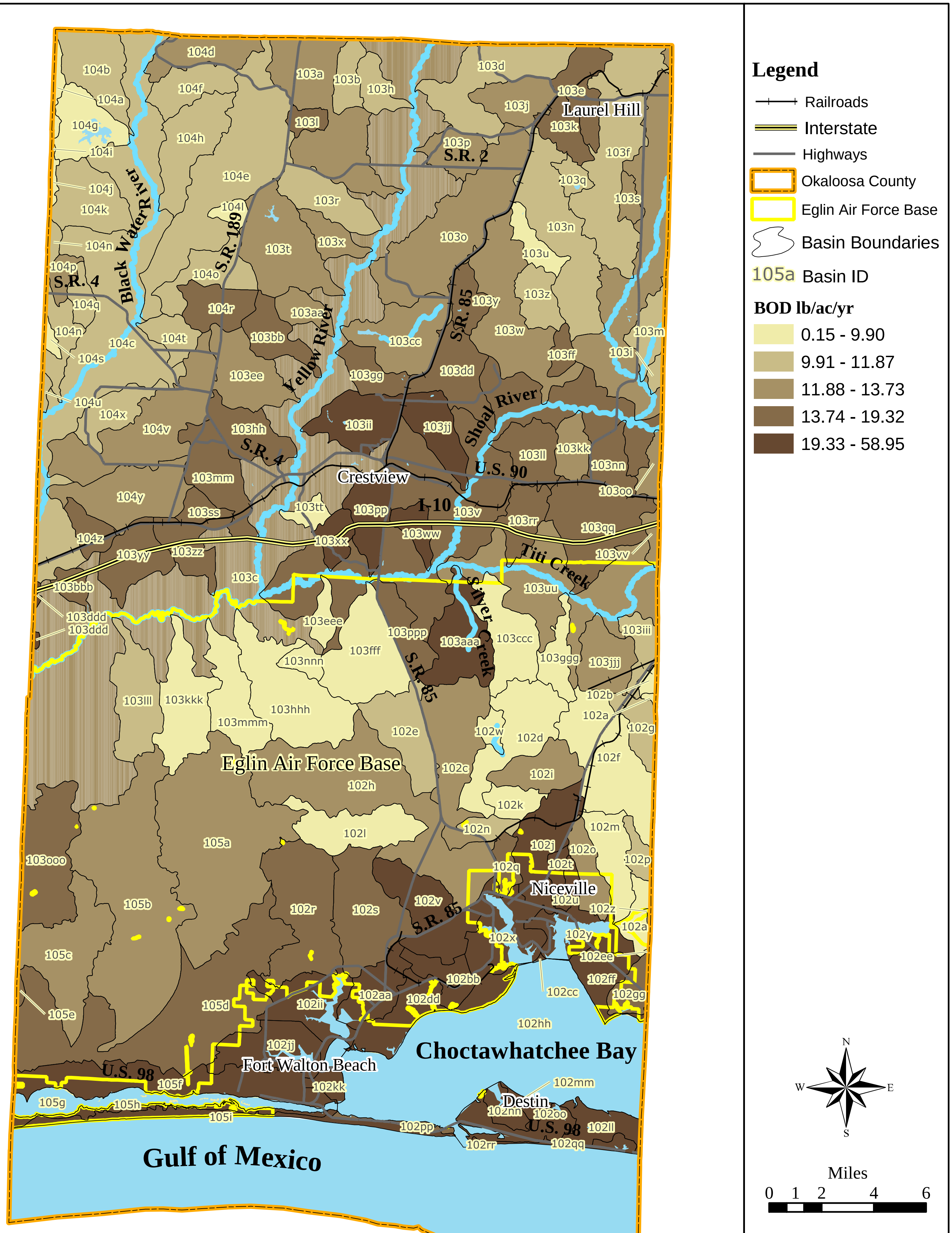
The pollutant loading model estimated nearly 9.5 million pounds of BOD per year loading to surface waters in Okaloosa County, an increase of 2.65% over the ELU scenario. As shown in **Figure 7-10**, the FLU scenario produced similar results to the ELU scenario attributing BOD loading to urban land use and spot locations within Eglin AFB. As with TN and TP some sub-basins showed reductions in total BOD loadings between the ELU and FLU scenarios.

Table 7.4 lists the 30 sub-basins in the top 20th-percentile group (shown in bold) that generated between 19.31 and 58.95 pounds of BOD per acre per year under the FLU scenario. The sub-basins belonging to the 80th-percentile or greater group appeared in urbanized areas, or in areas with large amounts of transportation and utility use. Relatively undisturbed forest lands or silvicultural areas did not generate large amounts of BOD per acre.

7.4.4.4 Total Suspended Solids

The pollutant loading model estimated 85.454 million pounds of TSS per year loading to surface waters in Okaloosa County, an increase of only 0.16 percent over the ELU scenario. As shown in **Figure 7-11**, the FLU scenario produced similar results to the ELU scenario attributing TSS loading primarily to urban land uses and secondarily to forested, silvicultural, and agricultural land uses. As with TN, TP, and BOD, some sub-basins showed reductions in TSS loadings between the ELU and FLU scenarios.

Table 7.4 lists the 30 sub-basins in the top 20th-percentile group (shown in bold) that generated between 164 and 397 pounds of TSS per acre per year. The sub-basins belonging to the 80th-percentile or greater group appeared in urbanized areas, or in areas with large amounts of transportation and utility use.



Master Stormwater Management Plan

Pollutant Loading by Basin, Normalized by Area
Future Land Use
(Biochemical Oxygen Demand)

Figure 7-10

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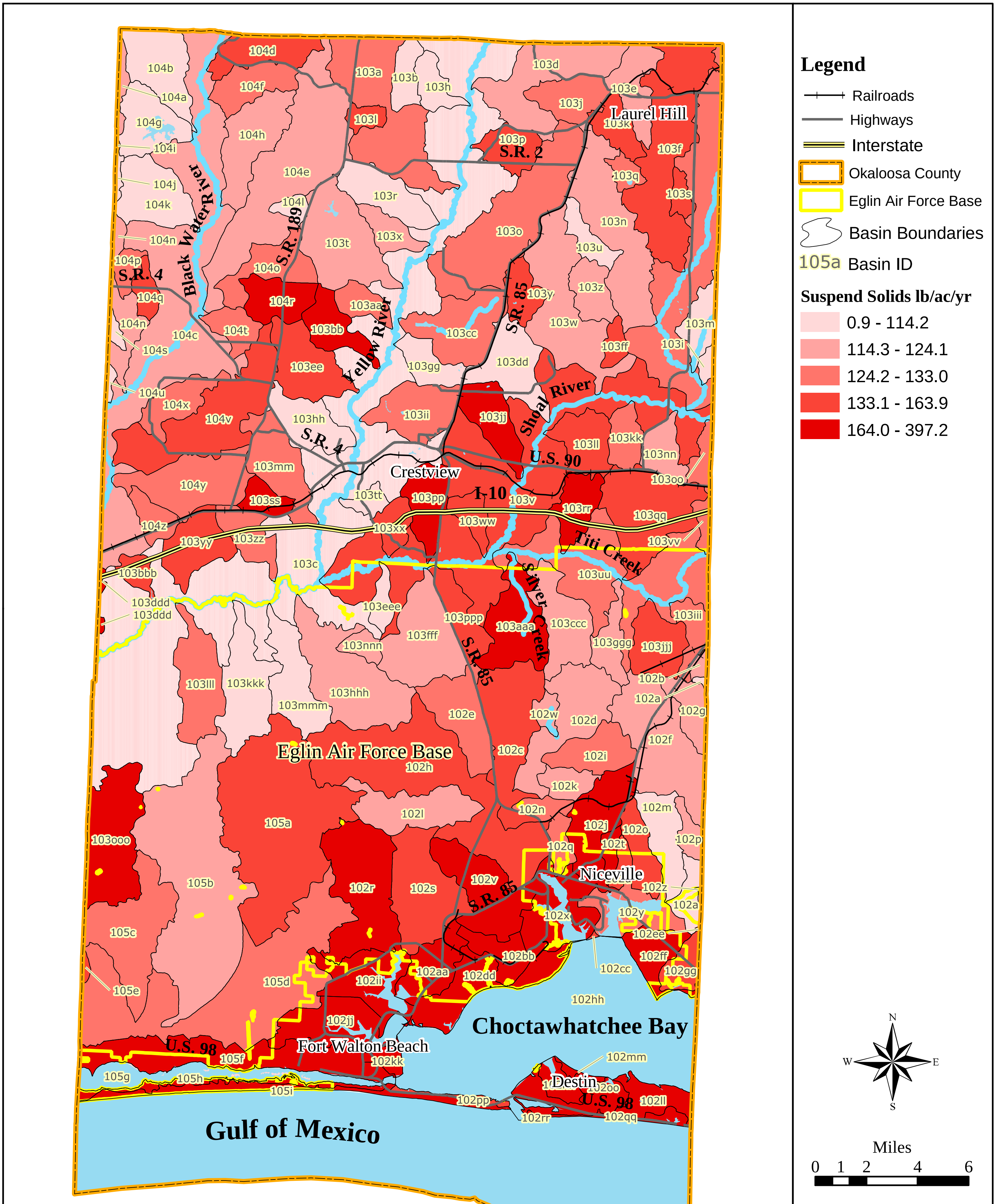


Master Stormwater Management Plan

Pollutant Loading by Basin, Normalized by Area
Future Land Use
(Total Suspended Solids)

Figure 7-11

fig_7-11.mxd - 03/19/2003

7.5 RECOMMENDATIONS

Table 7.5 lists the sub-basins contained in the 80th-percentile for all four pollutants to identify those sub-basins most in need of water quality treatment. The sub-basins that generated a large amount of pollutants per acre are listed with an “X” in the lbs/acre/year column. Sub-basins that were listed in the 303(d) report as not meeting water quality standards are listed in bold type. Although not discussed in this report, the sub-basins that generated the greatest total loading of pollutants are listed with an “X” in the lbs/year column. Additional information related to total loading can be found in the *Water Quality Analysis Report*.

The sub-basins that generated large pollutant loads per acre should be evaluated to identify the specific land uses that contribute to their high pollutant loading. These sub-basins might be good candidates for regional stormwater treatment systems, but might also benefit from other BMP implementation, such as more frequent street cleaning. Sub-basins listed in the 303(d) reports as having poor water quality conditions (shown in bold type) should be given priority for BMP evaluation and development. The sub-basins with an “X” in the lbs/year column that generate large, absolute amounts of stormwater pollutants should be evaluated for potential development of regional stormwater treatment systems if in urban areas, or the establishment of aggressive stormwater BMPs for silvicultural and agricultural lands in rural areas.

Table 7.5
Basins Recommended for Stormwater BMPs

Basin ID	Basin	80th-Percentile Pollutant Loading Listing	
		lbs/ac/yr	lbs/yr
102bb	Airport Drain	X	
103qq	Big Fork		X
103a	Big Horse Creek		X
104c	**Blackwater River**		**X**
102x	**Boggy Bayou**	**X**	**X**
102jj	Cinco Bayou	X	X
102rr	Destin Harbor	X	
102dd	**Direct Runoff to Bay 2**	**X**	**X**
102kk	**Direct Runoff to Bay 3**	**X**	
102mm	**Direct Runoff to Bay 4**	**X**	
102ll	**Direct Runoff to Bay 5**	**X**	
102ff	**Direct Runoff to Bay 6**	**X**	
105f	**Direct Runoff to Bay 7**	**X**	**X**
102qq	Direct Runoff to Gulf 1	X	
105d	**East River Bay**		**X**
102ii	Garnier Bayou	X	X
103f	Horsehead Creek		X
102oo	**Indian Bayou**	**X**	**X**
102nn	Joes Bayou	X	
103pp	**Juniper Creek 2**	**X**	
102r	Lightwood Knot Creek		X
105b	Live Oak Creek		X
103o	**Murder Creek**		**X**
104e	Panther Creek		X
104v	Penny Creek		X
103jj	Piney Woods Creek	X	
103g	Pond Creek		X
102aa	Poquito Bayou	X	
103w	Poverty Creek		X
102y	Rocky Bayou	X	
103v	Shoal River		X
103uu	Titi Creek		X
102v	Toms Creek		X
102h	**Turkey Creek 1**		**X**
105a	Turtle Creek		X
103c	**Yellow River**		**X**

8.0 REPAIR AND REPLACEMENT PROJECTS

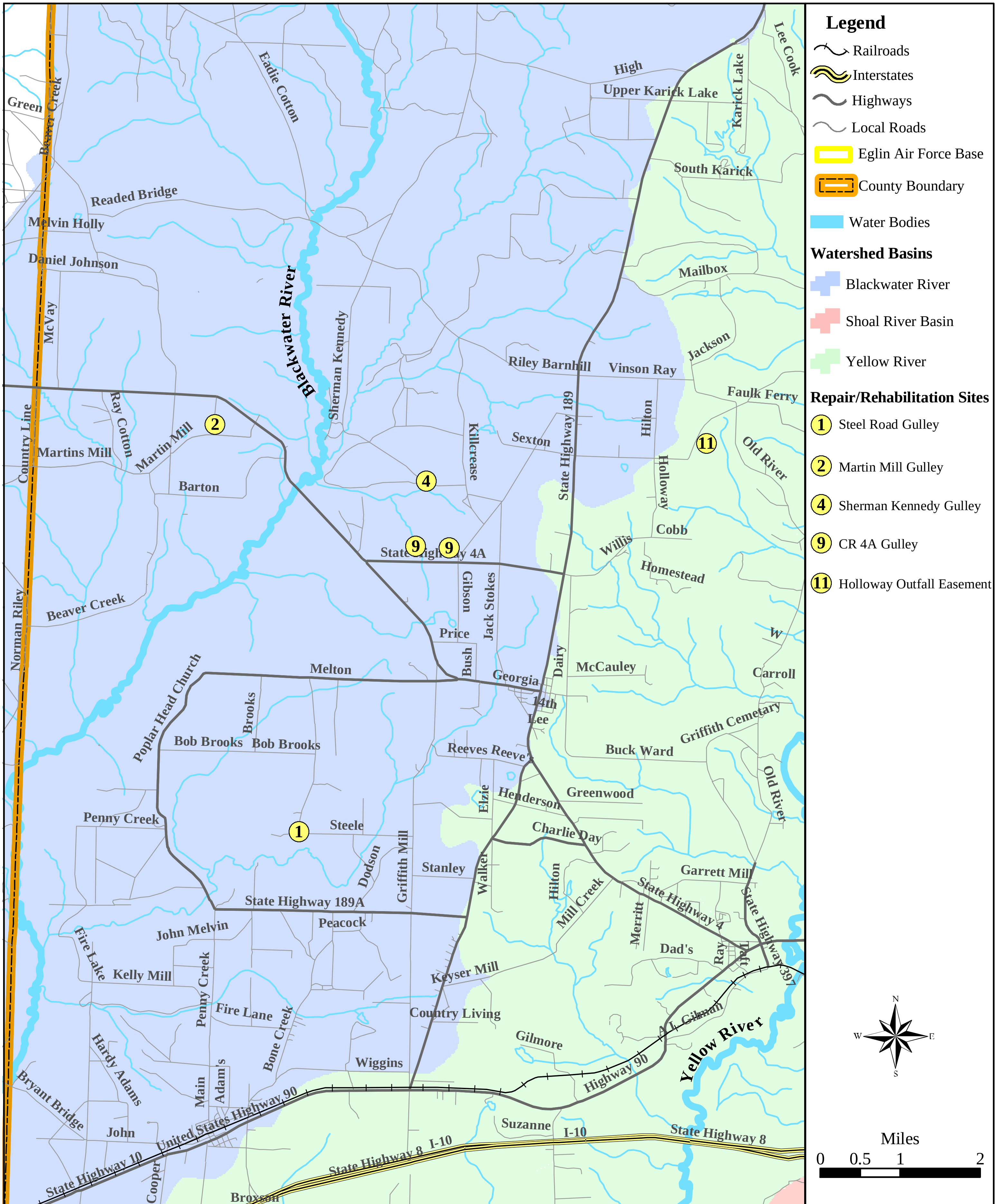
8.1 DATA COLLECTION AND RANKING

A team was formed consisting of Okaloosa County Engineering and Maintenance personnel and HDR personnel to discuss existing locations where repair projects are currently needed beyond the capability of the County. Projects were brought to HDR's attention throughout the entire County, and a ranked project list was generated. Reliance was placed upon the knowledge of County Staff who are actually involved in maintaining the various areas of the County and were able to provide accurate descriptions of actual field conditions. Ranking was based on the degree of potential for loss of embankment or roadway, with consideration also being given for driver safety.

8.2 SITE EVALUATIONS AND COST ESTIMATION

Once the list of locations was compiled, HDR accompanied County Staff to each site to develop an understanding of each situation and troubleshoot repair solutions. Decisions as to the required repair of each site were developed collectively among the team members, and planning level scopes of work and cost estimates were developed utilizing "as-built" plans where available. The total construction cost was supplemented with an additional amount for required permitting and engineering.

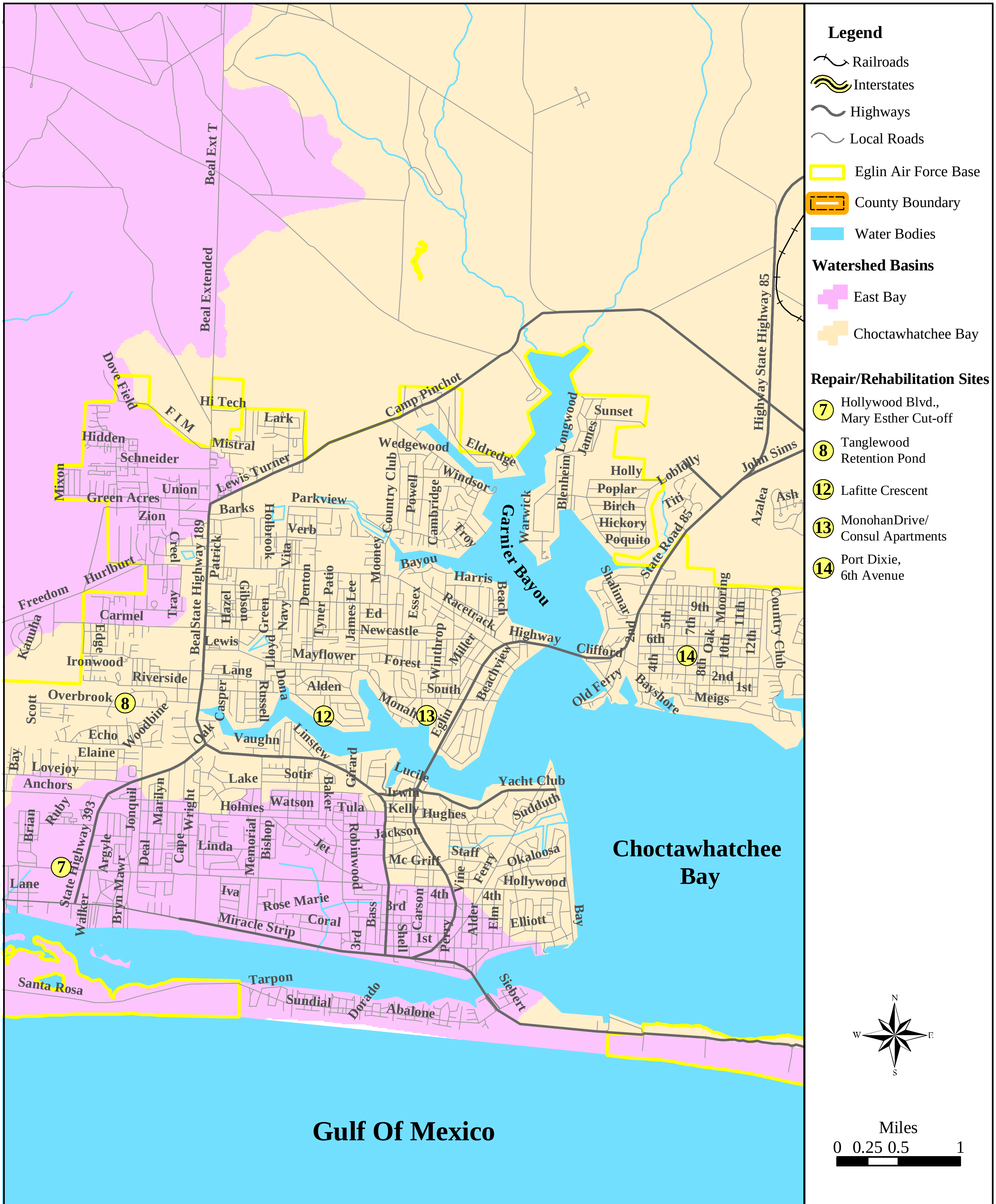
Table 8-1 contains a list of repair and replacement projects and their costs prioritized by Okaloosa County for inclusion in the CIP. **Figures 8-1** through **8-4** show the locations of the 14 projects.



Okaloosa Stormwater Management Plan

Repair and Rehabilitation Projects
Northwest Area of the County

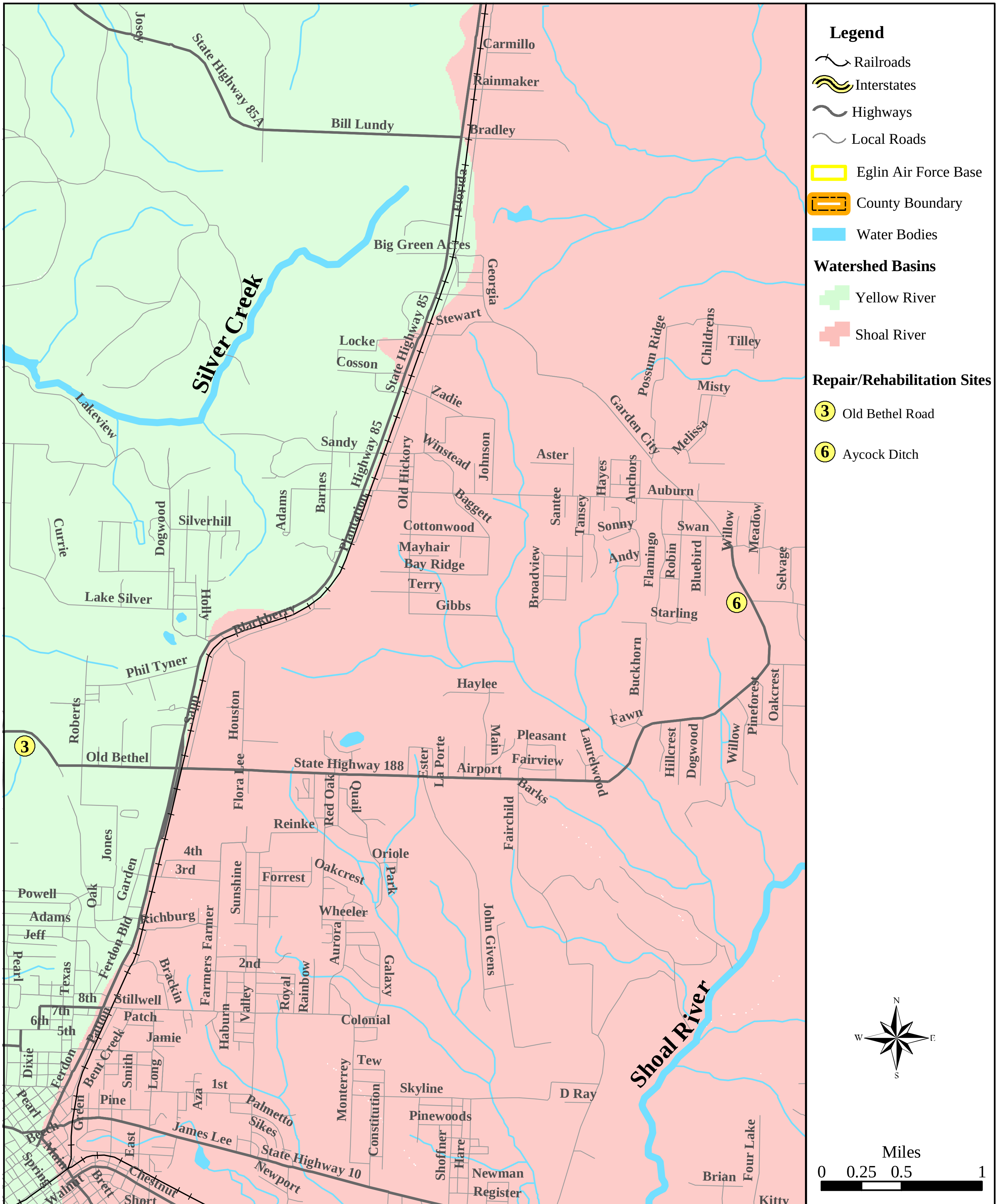
Figure 8-1



Okaloosa Stormwater Management Plan

Repair and Rehabilitation Projects
Fort Walton Area

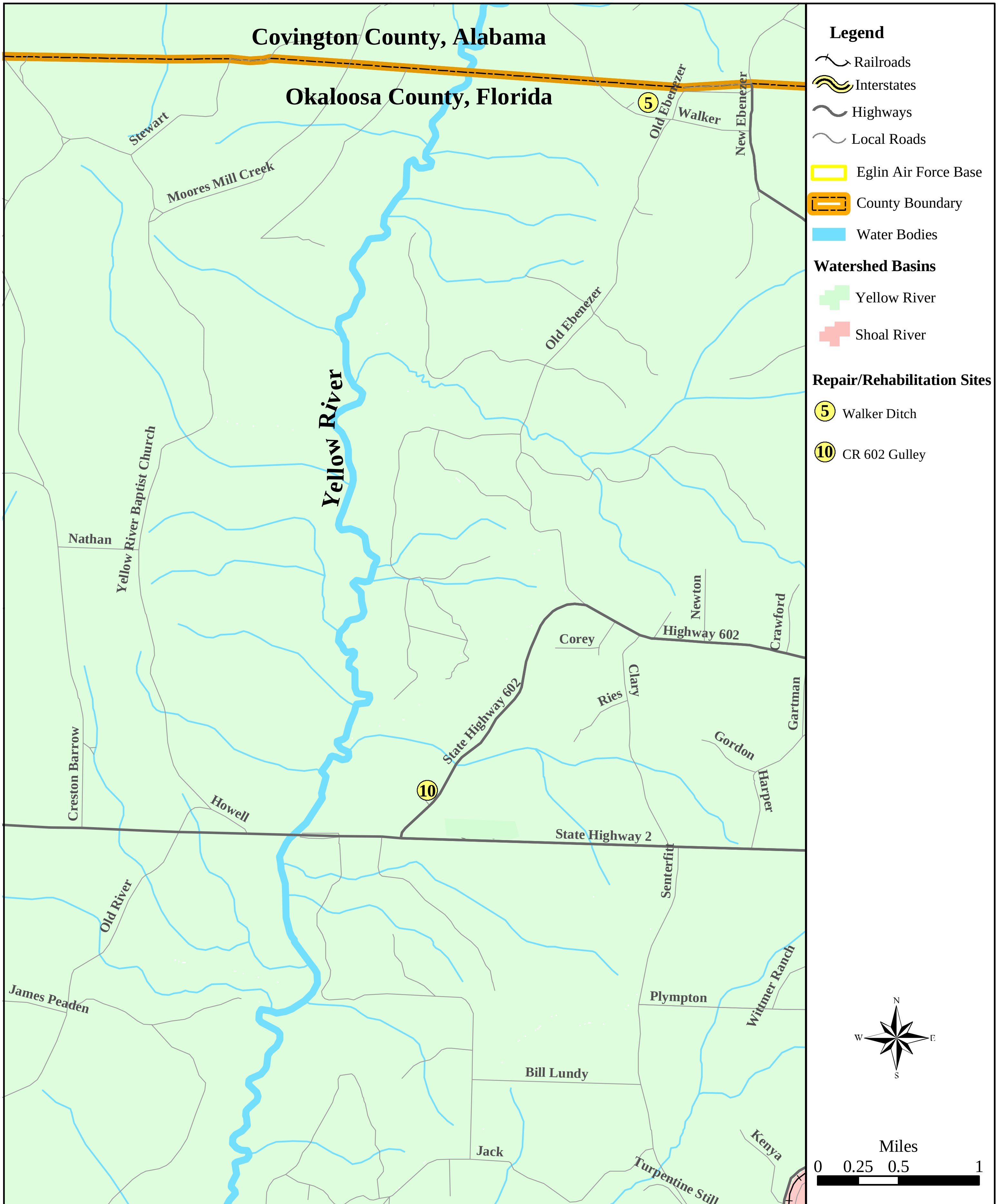
Figure 8-2



Okaloosa Stormwater Management Plan

Repair and Rehabilitation Projects
Crestview Area

Figure 8-3



Okaloosa Stormwater Management Plan

Repair and Rehabilitation Projects
North Central Area of the County

Figure 8-4

Table 8.1
Repair and Replacement Project List

Rank	Project Description	Estimated Construction Cost[1]
1	**Steel Road Gulley** Replace pipe and Junction box, bank stabilization – 2 locations	$125,000
2	**Martin Mill Gulley** Replace inlet, drop structure, pipe, bank stabilization – 2 locations	$75,000
3	**Old Bethel Road Outfall Easement** Clean-out/Replace pipe, bank stabilization, paved ditch, structures	$60,000
4	**Sherman Kennedy Gulley** Clean swale and rip-rap	$25,000
5	**Walker Ditch** Raise inlet with pop-off pipe, bank stabilization	$50,000
6	**Aycock Ditch** Clean and Grade ditch, rip-rap at ends	$10,000
7	**Hollywood Boulevard, Mary Esther Cut-off to Ready Avenue** Re-line approximately 1,000 feet of 24/36 inch pipe, repair inlets	$70,000
8	**Tanglewood Retention Pond System under power lines** Three ponds – reshape/stabilize slopes, replace pop-off structures and outfall pipe	$90,000
9	**CR 4A Gulley** Replace ditch pavement, rip-rap ends, grout voids under structure – 2 locations	$65,000
10	**CR 602 Gulley** Clearing, bank stabilization, rip-rap at outfall pipe	$40,000
11	**Holloway Outfall Easement** Rip-rap approximately 650 feet of ditch, new outfall structures	$50,000
12	**Lafitte Crescent** Re-line 670 feet of 36 inch pipe	$60,000
13	**Monohan Drive/Consul Apartments outfall** Re-line approximately 400 feet of 15/24/48 inch pipe	$50,000
14	**Port Dixie, 6th Avenue from 5th to 9th** Re-line approximately 2,000 feet of 36/48 inch pipe, repair inlets	$160,000

[1] Estimated cost reflects cost of construction and does not include engineering fees

9.0 RECOMMENDATIONS

Okaloosa County remains almost 80 percent undeveloped forest land, with all significant development concentrated near the coast, around Niceville, or in the vicinity of Crestview. In addition, development has not encroached significantly on historical flood plains. As a result, with the exception of those structures documented by the report, the County's stormwater conveyance systems generally operate at an acceptable level of service.

The fact that Okaloosa County has not yet developed to its full potential provides an opportunity to avoid future flood control and water quality issues through effective watershed maintenance.

9.1 REGIONAL STORMWATER PLANNING

Regional stormwater management facilities provide an opportunity to reduce pollutants while streamlining the cost of future projects in the selected basins. To provide a regulatory framework for regional planning, a documented understanding was reached with FDEP establishing a stormwater banking program. A summary of the operation of Okaloosa County's approved banking program follows:

- The County has five major watersheds, including the Yellow River, Shoal River, Blackwater River, East Bay, and Choctawhatchee Bay. A separate bank will be established for each watershed. A map showing these watersheds was delivered to FDEP for discussion and documentation.

- When the County builds a regional facility, a sub-basin will be delineated describing the area directly served by the facility. To be eligible for the program, the regional facility must treat, at a minimum, the entire sub-basin (sub-basin treatment volume). If banking credits are desired, the facility may provide additional treatment (excess treatment volume) above that required to fully treat the sub-basin.

- Once the facility is constructed, certified, and inspected by the FDEP, any excess treatment volume will be tabulated and banked for future consideration.

- All projects constructed within the sub-basin are covered by the sub-basin treatment volume and may be constructed under the permit issued for the regional facility upon notice to FDEP and concurrence, without the need for additional permitting.

- For linear projects (i.e. roadway projects) where treatment cannot be provided within the existing right-of-way, treatment can be accomplished by debit from the bank, provided the project is an eligible project and is located in a watershed with available banking credit. Examples of eligible projects include the addition of paved shoulders, new turn lanes, and dirt road paving projects.

To receive credits and account for debits, project submittals shall include a tabulation detailing the required treatment volume and any involved credits or debits. A spreadsheet format is preferred by FDEP.

Because the construction of a regional facility requires substantial capital expenditure, the facility should meet both quality and quantity goals, and provide an opportunity for cost benefits. In this regard, the pursuit of regional facilities is recommended where at least two of the following criteria apply:

- The facility will reduce pollutant loading in an area discharging to an impaired water body, or contributing sufficient pollutants for inclusion on the Pollutant Loading 80th Percentile Listing.

- The facility is located in an area with identified future County projects that could receive stormwater treatment by compensation in the facility.

- The facility is located in an area that is expected to develop in the near future, increasing environmental impacts.

Applying these criteria, **Table 9.1** presents sub-basins that have been identified for potential regional facilities.

Table 9.1
Sub-Basins Identified as Priority Candidates for Regional Stormwater Management

Sub-Basin ID[1]	Sub-Basin Name	Reasons For Inclusion
103dd	Bends Creek	80th Percentile List, Future Projects
102jj	Cinco Bayou	80th Percentile List, Future Projects
103ff	Clear Creek	80th Percentile List, Future Projects
102dd	Direct Runoff to Bay 2	Directly discharges to 303d stream, 80th Percentile List, Future Projects
102ll	Direct Runoff to Bay 5	Directly discharges to 303d stream, 80th Percentile List, Future Projects
102ff	Direct Runoff to Bay 6	Directly discharges to 303d stream, 80th Percentile List
102ii	Garnier Bayou	80th Percentile List, Future Development
103pp	Juniper Creek 2	Directly discharges to 303d stream, 80th Percentile List
103o	Murder Creek	Directly discharges to 303d stream, 80th Percentile List
103jj	Piney Woods Creek	80th Percentile List, Future Projects, Future Development

1. See Figure 7-1 for location of sub-basin ID.

It should be noted that additional sub-basins within the County meet these criteria. However, these basins are not included because they are located either on Eglin AFB or within incorporated areas of the County.

9.2 NON-STRUCTURAL IMPROVEMENTS

In addition to meeting the EPA and DEP regulations, the County's NPDES Phase II NOI contained in Appendix A provides a summary of the County's non-structural program. This program is separated into the following six minimum control measures:

- Public Education and Outreach
- Public Involvement/Public Participation
- Illicit Discharge Detection and Elimination
- Construction Site Stormwater Runoff Control
- Post-construction Stormwater Management in New Development and Redevelopment
- Pollution Prevention/Good Housekeeping

Details regarding the programs recommended to meet each minimum control measure appear in the attached NOI document. Note that portions of the non-structural program have been initiated as part of this project, such as the development of an inventory system, revisions to the Land Development Code, and defining maintenance needs.

9.3 STRUCTURAL IMPROVEMENT SUMMARY BY BASIN

Table 9.2 summarizes the project recommendations made in Chapters 3 through 8 by basin.

Table 9.2
Structural Improvement Recommendations by Basin

Project Description
Blackwater River Basin
Steel Road Gulley
Martin Mill Gulley
Sherman Kennedy Gulley
CR 4A Gulley
Yellow River Basin
Old Bethel Road Outfall Easement
Walker Ditch
CR 602 Gulley
Holloway Outfall Easement
Culvert Desilting – 65
Culvert Replacement - 90
Foxwood Subdivision (Option 2)
Shoal River Basin
Aycock Ditch
Culvert Replacements - 92, 93, 94
Antioch Road
Coastal Basins
Hollywood Boulevard, Mary Esther Cut-off to Ready Avenue
Tanglewood Retention Pond System under power lines
Lafitte Crescent
Monohan Drive/Consul Apartments outfall
Port Dixie, 6th Avenue from 5th to 9th
Culvert Replacements - 13, 14, 201, 202, 203, 207, and 210-213
Meigs Drive Improvements
Commons Drive Improvements
Gap Creek Recommendations
Cimarron Outfall Improvements
Lake Blake Outfall Improvements

9.4 PROJECT RANKING (CIP)

As shown below, the total cost of all of the proposed structural improvements presents a large financial burden. In this regard, the improvements will have to be undertaken by the County as funding becomes available. Detailed cost estimates are contained in **Appendix D**.

To aid in the establishment of priorities, all capital improvement projects identified by the study were ranked. Although an objective analytical approach was not followed due to the diversity of projects addressed, the projects were evaluated for feasibility and effectiveness. Emphasis was given to projects that will reduce the risk of flood damage, protect existing infrastructure, or provide public health and safety benefits. Repair and replacement projects were generally given a high priority as these projects present immediate needs, and could become aggravated with time. The LOS culvert replacements were typically considered a low priority, as the LOS replacement recommendations are based on a systematic analysis, and not reported problems.

Using these criteria the County's CIP was ranked in the order shown in **Table 9.3.**

Table 9.3
Ranked CIP List

Rank	Basin	Project Description	Analysis Category	Estimated Cost[1]
1	Blackwater	Steel Road Gulley – Replace pipe and junction box, stabilize bank	R&R	$125,000
2	Blackwater	Martin Mill Gulley – Replace inlet, drop structure, and pipe; stabilize bank	R&R	$75,000
3	Yellow	Old Bethel Road Outfall Easement – Replace pipe, stabilize bank, pave ditch, miscellaneous structures	R&R	$60,000
4	Blackwater	Sherman Kennedy Gulley – Regrade swale and add rip-rap	R&R	$25,000
5	Yellow	Walker Ditch – Raise inlet with pop-off pipe, stabilize bank	R&R	$50,000
6	Shoal	Aycock Ditch – Clean and regrade ditch, add rip-rap	R&R	$10,000
7	Coastal	Hollywood Boulevard, Mary Esther Cutoff to Ready Avenue – Reline approximately 1000' of 24"/36" pipe, repair inlets	R&R	$70,000
8	Coastal	Tanglewood ponds under power lines – Reshape/stabilize slopes for 3 ponds, replace weir structures and outfall pipe	R&R	$90,000

Table 9.3
Ranked CIP List

Rank	Basin	Project Description	Analysis Category	Estimated Cost[1]
9	Blackwater	CR 4A Gulley – Replace ditch pavement, add rip-rap at ends, grout voids under structure	R&R	$65,000
10	Yellow	CR 602 Gulley – Clear, stabilize band, add rip-rap at outfall	R&R	$40,000
11	Yellow	Holloway Outfall Easement – Add rip-rap to approximately 650' of ditch, provide new outfall structures	R&R	$50,000
12	Coastal	Cimarron Outfall - Regrade Ditch, Replace 4 culverts	Detailed Study	$290,000
13	Shoal	Antioch Road – Raise roadway profile, replace 5 culverts	Detailed Study	$610,000
14	Yellow	Foxwood Subdivision – Add underdrains	Detailed Study	$85,000
15	Coastal	Install Gage Site #2 – Cinco Bayou	Data Collection Sites Report	$5,000
16	Coastal	Lafitte Crescent – Reline 670' of 36" pipe	R&R	$60,000
17	Coastal	Monohan Drive/Consul Apartments Outfall – Reline approximately 400' of 15"/24"/48" pipe	R&R	$50,000
18	Coastal	Port Dixie, 6th Avenue from 5th to 9th – Reline approximately 2000' of 36"/48" pipe, repair inlets	R&R	$160,000
19	Coastal	Meigs Drive – Replace culvert, raise roadway profile	Detailed Study	$135,000
20	Coastal	Install Gage Site #1 – East Bay River	Data Collection Sites Report	$5,000
21	Shoal	Install Gage Site #6 – Pond Creek	Data Collection Sites Report	$5,000
22	Coastal	Replace Culvert 203 under US 98 east of Hurlburt gate	LOS	$110,000
23	Shoal	Replace Culvert 93 under Highway 90 at Mill Creek	LOS	$60,000
24	Coastal	Replace Culvert 207 under US 98 west of Leisure Time RV Center	LOS	$20,000

Table 9.3 Ranked CIP List				
Rank	**Basin**	**Project Description**	**Analysis Category**	**Estimated Cost[1]**
25	Coastal	Replace Culvert 14 under SR 189 at Garnier Creek	LOS	$450,000
26	Coastal	Replace Culvert 213	LOS	$20,000
27	Coastal	Replace Culvert 201 under US 98 near Magnolia Shores	LOS	$25,000
28	Shoal	Replace Culvert 94 under Okaloosa Lane at Mill Creek	LOS	$60,000
29	Yellow	Replace Culvert 90 under Pandora Drive	LOS	$40,000
30	Coastal	Replace Culvert 210 under US 98 near Hurlburt Field Housing	LOS	$20,000
31	Coastal	Replace Culvert 13 under SR 189 at Lightwood Knot Creek	LOS	$180,000
32	Coastal	Replace Culvert 212 east of 98 West Liquor Store	LOS	$20,000
33	Coastal	Replace Culvert 202 under US 98 east of Hulburt pedestrian overpass	LOS	$20,000
34	Shoal	Replace Culvert 92 under Highway 90 at Toms Creek	LOS	$50,000
35	Coastal	Replace Culvert 211 under US 98 west of Hurlburt pedestrian overpass	LOS	$20,000
36	Coastal	Blake Lake – Upgrade storm drain	Detailed Study	$150,000
37	All	Install all remaining gage sites	Data Collection Sites Report	$35,000
38	All	Regional Stormwater Facilty – Build one each 5 year cycle	None	$300,000
SUBTOTAL				$3,645,000
ENGINEERING AND PERMITING @ 20%				$729,000
TOTAL				**$4,374,000**

9.5 FEMA MAP REVISIONS

At this time no FEMA map revisions are recommended. However, as federal funds become available under the map modernization program, the following is recommended:

- Extend the coverage of the County TINs to completely encompass the computed flood plains
- Truth the TINs to benchmark survey
- Supplement the TINs with limited surveyed cross-sections to better define the channel of the main stems
- Focus map revision efforts on tributaries in developing areas such as the South County and Crestview.

10.0 FINANCING STORMWATER IMPROVEMENTS AND OPERATIONS

10.1 INTRODUCTION

This chapter identifies a recommended course of organizational and funding action for Okaloosa County that, when implemented, will allow it to meet its federal regulatory requirements, address "catch-up" and future stormwater infrastructure needs, and provide for a level of operations and maintenance that will assure that stormwater facilities are performing according to expectations and that stormwater runoff meets or exceeds desired water quality goals.

10.2 STUDY LIMITATIONS

Currently, there is no formal stormwater department or division in Okaloosa County. Existing County stormwater efforts are primarily associated with the construction and maintenance of road projects. As such, it was necessary to estimate the current level of County stormwater spending among the various departments and programs and present it as a "virtual" program that is occurring, but has no separate identity within the current County organizational structure.

Forecasts expenses for the next five years were made under a series of assumptions about the potential capital and NPDES Phase II programs. The forecast of possible revenue sources was made using existing available information, such as parcel data provided by the County GIS Department. The implementation of any new funding tools would require additional data not currently compiled, such as the amount of impervious area covering each parcel.

As the future stormwater program takes shape and improved information becomes available, program expenses, revenue requirements, and amount of funding levies will change to some extent. Therefore, the program descriptions and financial analyses addressed in this report should be viewed as a conceptual feasibility study of alternative program feature - a level of analysis sufficient to guide the County in its decision-making process, but one that needs further development during implementation.

10.3 CURRENT STORMWATER PROGRAM LEVELS AND FUNDING

The historical stormwater and drainage activities of Okaloosa County government have been closely associated with the County's road construction and maintenance programs, so much so that the stormwater activities do not have any noticeable separate identity in the County budgeting process. The County's FY2003 CIP identifies a number of drainage improvements, but all are associated with road construction and rehabilitation improvements.

Staff currently performing or supporting stormwater functions are dispersed around the County in various road districts, bridge units, construction, engineering, and administrative office locations. Most equipment used in stormwater maintenance is shared with other County services. In general, the current stormwater activities of the County have no separate or distinct organization, staffing, or resource identity.

For purposes of establishing a baseline or current-day level of County stormwater spending, it was necessary estimate what portions of various County Road Department expenses are attributable to stormwater efforts. Estimates of stormwater spending from the Road Department's Personnel Services, Overtime, Contractual Services, Repair and Maintenance Services, Fuels, Materials, and new Construction accounts were provided by County Public Works staff. Based on those estimates, current annual spending related to County stormwater activities totals about $1.1 million and yields the sum of about 16-18 full-time equivalent (FTE) staff from part-time support efforts of various City departments and other programs within the Public Works Department.

Revenue to support these roads and drainage services currently originates from the County Transportation Trust Fund. Primary sources of revenue to the Transportation Trust Fund include gas/fuel tax, a half-cent sales tax, and toll bridge proceeds. In fiscal year 2003, these revenues were budgeted to cover, in full, the anticipated expenses of the County Road Department.

10.4 SOURCES OF FUNDING FOR STORMWATER SERVICES

10.4.1 Historical Funding in the U.S.

Around the U.S., drainage services have historically been a periodic, sometimes visible issue for local government. When it rains and floods, it becomes a public priority. When it is dry, public interest wanes. Funding support and desire for a focused, continuing drainage effort have typically followed this same cyclical path. Further, funding for drainage must compete for limited public funds with other program services (roads, police, fire, EMS, etc.), many of which sustain a high-priority funding status.

As a result, particularly where significant growth has or is being experienced, local governments are typically "behind the curve," trying to catch-up and remediate existing drainage problems and maintain an ever-growing drainage system. The historically unattainable goal for most local governments is to get out in front of the drainage issues and keep future problems from growing.

Over time, even the terminology used to reference these desired governmental functions has evolved from a focus on providing for simple drainage of floodwater to supplying effective stormwater management services that can promote multi-purpose goals of health and safety, water quality, environmental, and recreational/aesthetic values for the community.

Probably the biggest impetus for the innovation of multi-objective stormwater thinking is the current and prospective regulatory requirements of the federal NPDES Phase II permitting process, which affects large urban areas and selected mid-size urbanized areas (i.e., MS-4 cities). NPDES Phase II identifies urban runoff as a "point source" discharge of pollutants to the nation's water that requires a permit from the Federal or State government (where states have assumed designation as the permitting authority). The permits are being conditioned to require the permit holders, at their own expense, to perform a variety of stormwater management activities (in six general program areas) that will directly or indirectly address the quality of urban stormwater runoff.

Because of these federal/state requirements, many local governments are faced with not only increased expenditures for stormwater, but also continuing annual expenditures for stormwater. This has helped transform the old, sometimes important, drainage function into continuing multi-purpose stormwater program requirements with annual reporting responsibilities to the regulatory authority.

However, the need for effective stormwater management should not be simply viewed as another unfunded federal mandate. A recent workshop of ten managers of prominent stormwater utilities from around the U.S. all echoed a common theme, the leading-edge stormwater programs have achieved their success and support, not from basing the need for action on unfunded federal mandates, but instead by involving the public and helping to transform their waterways from sometimes hazardous, perhaps unhealthy, streams and rivers into community assets that are used and valued by their citizens.

These changing regulatory requirements, public preferences, development and environmental impact issues, and improved science are forcing a reevaluation of what and how these services are provided. Okaloosa County will be a Phase II permit holder, and its proposed programs will also elevate the priority of continuing funding needs for the County stormwater services.

This advent of the Phase II requirements is also coming at an inopportune time for many state and local governments caught in the grips of recessionary impacts on government revenues. Property and sales tax revenues have been diminished, while at the same time, many communities are forced to make increased infrastructure investments and operational spending to remedy the effects of existing development and to provide for anticipated growth.

Because of the increased funding needs for stormwater, underlined by the fact that many general or dedicated funds cannot afford additional spending, many regional, county, and local governments are turning to the option of creating stormwater utilities.

As related by the Florida Association of Stormwater Utilities (now the Florida Stormwater Association), a stormwater utility (SWU) is an enterprise fund structured utility service program that has a focused, mission-oriented goal of improved stormwater management and sustainable revenues, generally from "user" or rate charges (FASU, 1997). Stormwater utilities were first created in Colorado and Washington in the 1970's with a focus on funding drainage. Tallahassee was the first city in Florida to establish one in 1986, and the period of the late 1980's and early 1990's saw rapid growth in the creation of these new government programs in Florida, many with multi-purpose stormwater missions.

As of the FASU's 1997 survey, there were 91 established SWUs in Florida, but only comprising about 20 percent of the Florida entities with stormwater responsibilities. Of the 91 SWUs, 93 percent (85 utilities) were established by municipal governments and 7 percent (6 utilities) were created by urban counties to serve residents of unincorporated areas (FASU, 1997).

10.4.2 Alternative Sources of Funding for Stormwater in Okaloosa County

Historically, most drainage services for municipalities have been funded out of general revenues, comprised mainly of property and sales tax revenues. Some county governments use similar sources of funds or are able to enjoy special dedicated funding sources that provide specifically for drainage or roads and related drainage. The primary source of funding for drainage services by Okaloosa County is its Transportation Trust Fund that includes gas/fuel taxes, a half-cent sales tax, and toll bridge proceeds as the significant sources of revenue. Within this dedicated fund, monies spent on drainage compete with funds available for roads.

Table 10.1 presents a list of various taxes, rates, and fees typically used by local government to fund stormwater services. Various characteristics of these levies are described, including whether or not:

- The levy can provide sufficient funds for an adequate stormwater program (funding adequacy),
- A dependable amount of revenue can be counted on from year-to-year (revenue stability),
- The local government has some broader discretion in how the funds are used (flexibility),
- The levy provides for ease and efficiency in managing the revenue program (cost of administration),

Table 10.1
Characteristics of Alternative Stormwater Funding Mechanisms

Type of Funding	Funding Adequacy	Revenue Stability	Flexibility in Use of Funds	Cost of Admin.	Needed Legal Authority	Fairness and Equity
Property Tax	**Usually insufficient** Given periodic nature of flooding problems and typical higher spending priorities given to other general government programs.	**Relatively stable** But can vary with economic cycle.	**Flexible** Funds can be used for a variety of stormwater purposes, but may be limited to the authorized purposes of the tax.	**Low**	**Present** General Obligation bonding would require voter approval.	**Not equitable** Property value is not highly correlated with contribution to flooding or water quality problems.
Sales Tax	**Usually sufficient** Sufficiency depends upon portion of sales tax allocated for stormwater and amount of sales tax base.	**Relatively stable** But can vary with economic cycle.	**Flexible** Funds can be used for a variety of stormwater purposes.	**Low**	**May require voter approval**	**Not equitable** Economic activity is not correlated with contribution to flooding or water quality problems.
User Rates	**Usually sufficient** Sufficiency depends on political acceptability of rates.	**Stable**	**Flexible** Funds can be used for a variety of stormwater purposes.	**Medium**	**Present** Revenue bonding does not require voter approval.	**Equitable** Relates user charge to a measure of contribution to flooding and water quality problems.
Impact Fee	**Partially sufficient** Usually helps pay portion of new capital. Sufficiency usually depends on political acceptability of growth-related fees.	**Variable** Can vary with degree of growth.	**Less Flexible** Funds should only be used for capital improvements providing for growth.	**Medium**	**Present**	**Equitable** Relates user charge to a growth-related contribution to flooding and water quality problems.
Grants	**Partially sufficient** Usually helps cover portion of project-related cost, many times with local cost-share match.	**Variable** Can vary given uncertainty of awards.	**Less Flexible** Funds only used for purposes identified in the grant.	**Medium**	**Present**	**Not applicable**

- Current statutory authority exists for the county government to authorize the levy (needed legal authority), and
- The levy generally treats customers fairly for the services rendered (fairness and equity).

10.4.2.1 Property Tax

Property tax revenues are generally stable, local governments have broad flexibility in the use of funds, and the cost of administration is relatively low. However, property taxes for most local governments have not proven historically sufficient to maintain an adequate, on-going drainage program, nor do they readily provide for debt funding of large drainage projects, nor are they equitable in charging the populace for problems caused or services rendered. Property value has little relationship to the property's contribution to flooding, and some states (such as Texas) specifically prohibit the use of property value as a means of designing the funding levies for municipal stormwater utilities. For instance, a high-value, multi-story building may have a small building "footprint" that sheds relatively little stormwater runoff, but a nearby paved parking lot with a low property value may produce a great deal of rainfall runoff.

10.4.2.2 Sales Tax

Dedicated sales tax or other special tax revenues are also generally stable over time, have a low cost of administration, and can be sufficient in funding a variety of stormwater purposes, unless the tax authorization more narrowly limits the size of the levy or use of funds. However, gaining the sales tax funding tool typically requires statutory authorization and voter approval, and the level of economic (retail sales) activity behind the sales tax does not highly relate to the contribution to flooding, so its fairness and equity considerations are low.

10.4.2.3 User Rates

User rates (sometimes inappropriately called a stormwater fee) can provide for sufficient, stable revenue for a variety of stormwater programs, and the legal authority is present for Florida municipalities and counties. User rates, properly designed, can fairly relate the contribution to flooding and areawide services rendered to the levy charged. However, the cost of developing and administering a user charge system can be higher than other types of levies.

10.4.2.4 Impact Fees

An impact fee (also called capital recovery fee or a system development charge) is a one-time, up-front fee designed for a specific, limited purpose, namely, to make growth help pay for the "impacts" or costs caused by new development and to reduce the amount of funds paid by existing customers. Impact fees are equitable in their nature and can produce a noticeable amount of revenue over time, but in most cases, the amount of revenue from this source is not fully sufficient to offset the costs related to growth. The amount of impact fee revenues can vary dramatically from year-to-year with development/business cycles. Also if the need for growth-related projects is near-term, the costs to get infrastructure in place may precede the collection of the impact fees, so that other funds or debt financing is required.

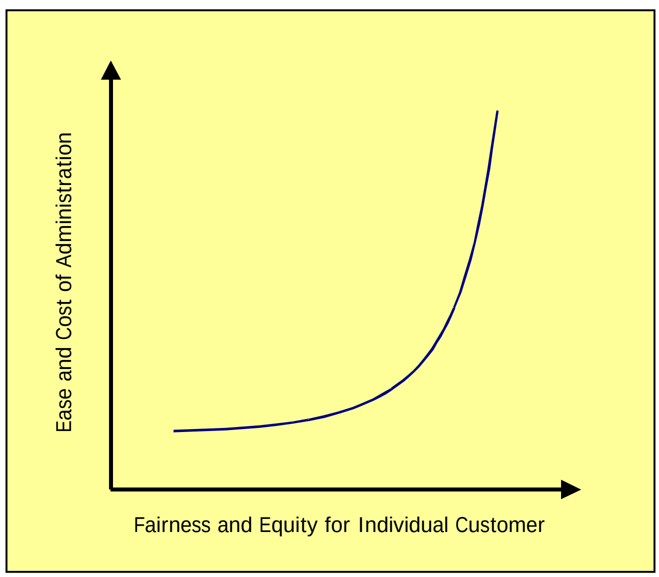
10.4.2.5 Grants

Proceeds from Federal or state grants can be a valuable source of funds in that they do not directly originate from local coffers. However, grant funding is many times targeted towards a special project and is not flexible for a variety of uses. Typically, grants will also require a local cost share and provide only a portion of the project funding. Further, grants are usually not a dependable source of funding and may entail higher costs of administration and reporting.

10.4.3 Stormwater Rate Design Issues

Most user charge systems have an implicit trade-off between the degree of fairness and equity for the individual customer and the complexity and ease/cost of administration of the user charge system as shown in **Figure 10-1**. Within a utility system, whether it be an electric, water, wastewater or stormwater utility, every individual customer has a unique cost of service that it imposes on the system. For example, a residential customer located next to a wastewater treatment plant imposes a lesser degree of cost on the utility system than does a residential or commercial customer located on the far side of town whose effluent is conveyed over long distances or may be of higher wastewater strength. Changes in location, topography, soils, and service use characteristics can all affect the degree of customer costs.

Figure 10-1
Conceptual Tradeoff Between Fairness and Equity and Costs of Administering a User Charge System



Utility customers also benefit differently from the services received. Water used in an office restroom may have a much lower implicit value than water used in high-value electronics manufacturing, yet both may be charged the same water rate. The benefit-side aspects of a stormwater user charge can be more complex. While some customers may benefit more directly from stormwater improvements or services (such as alleviating flooding to that property), all area residents and businesses, even those on the “top of the hill,” benefit indirectly from improved safety, and transportation and emergency access to their homes, schools, hospitals, business districts, etc., as well as other possible advantages of improved water quality, aesthetic or recreational opportunities.

It is impractical to have unique individual rates for every single customer in a utility system that accurately reflects a true cost and benefits picture. Not only are there analytical problems in deriving such complex rates, the administrative costs of gathering and maintaining this level of information is cost-prohibitive.

As a means of striking a balance between fairness/equity and a manageable, affordable rate system, the concept of customer classes is often used. With a customer class approach, “like” customers are grouped together for purposes of being charged a common user rate. In this manner, customer similarities and differences, such as service use characteristics, can be broadly acknowledged. Everyone within a customer class does not achieve perfect equity, but a generally, fair, equitable, and manageable user charge system can result. Case law has upheld the right of utilities to levy customer class and areawide charges where unique cost of service and benefit issues to individual properties are not over-riding considerations.

Another key issue in developing a user charge system is the basis for which the service is charged. Is it the quantity of service used? Is it the amount of time or time of day in which it is used? Or is it some other measure? In water rates, it is usually the volume of metered water use. In electric rates, it may include both quantity of use and time of day.

With respect to stormwater rates, the most widely used basis for this type of levy is a square foot measure of impervious cover. Impervious cover is usually defined as hardened, relatively impermeable, ground cover that rejects the absorption of rainfall and yields stormwater runoff. In this manner, the amount of impervious cover present on a property (typically rooftop, deck, driveway/parking area, and sidewalks) can identify a measure of contribution to the flooding problem and relate the cause of the problem to the amount of levy imposed on the customer.

Since many communities do not collect data on impervious cover and are faced with the cost of developing and maintaining this information, some entities have chosen to use more indirect measures as the basis of their stormwater rate design, such as building square footage or lot size. However, neither building square footage (especially in multi-story buildings) or lot size (developed or undeveloped lot?) is as appropriate a measure of likely stormwater runoff as the impervious cover statistic.

However, even the impervious cover measure by itself is not a perfect indicator of stormwater runoff. Some properties may have flat or highly absorbent soils while other properties may be rocky or sloping. Two properties with the same amount of impervious cover may have different rainfall runoff characteristics due to the density of development on the lot (e.g., fully developed versus grassy buffers) or the presence or lack of on-site drainage controls. Some properties may be located adjacent to waterbodies where there are no substantial downstream development at risk from the increased runoff of the property.

Once again, there are trade-offs in how complex a stormwater rate system can get and still be administratively manageable and affordable. The public must bear in mind that the costs of administering the rate system are paid by the customers.

The most common characteristics of stormwater rate systems across the U.S. include:

- Impervious cover as the preferred basic measure of stormwater "service use,"
- A single rate (charge) per square foot of impervious cover,
- Two customer classes whereby:
    - All single family residential properties receive an equivalent bill per month reflecting an average amount of impervious cover per single-family residential property,
    - Non-residential (apartments, commercial business, industry, and institutional land uses) receive unique monthly bills based the specific amount of impervious cover determined for these individual properties, and
- A possible credit against paying the full rate that is based on the degree of on-site drainage improvements funded and maintained by someone other than the governing entity and where affected customers must file for a consideration of the credit.

There are many variations to this approach. Some entities will bill stormwater monthly using an existing (water, wastewater, etc.) utility billing system, while others may use an assessment that is presented on the annual or semi-annual property tax statement. Some may levy different user rates for residential versus non-residential properties (although this isn't recommended as the run-off impacts of residential impervious cover can't be easily distinguished from the effects of non-residential impervious cover).

Some entities, particularly in the southeast U.S., provide a tiered pricing block in their residential rate that is intended to provide “lifeline” rate relief to low- or fixed-income customers (if this is based purely on income levels, there may be questionable legality to this approach from a cost of service point of view). While other entities may enact tiered pricing blocks or rates based on the density of development on the parcel (impervious cover as a percent of total lot size).

Another variation from entity to entity is whether exemptions are granted and to whom. Probably, the most common exemption is for the entity making the stormwater levy to exempt itself. This is rationalized as not having to move money from one of their pockets (departments) to another. However, good enterprise-fund cost accounting practices would be for all government departments to pay their full costs and for the payment of stormwater rates to be explicitly included as a budgeted expense for all departments. There is often community pressure to also exempt other types of land uses, such as federal or state property, schools, charitable organizations, etc. which may be tax exempt and feel they should also continue to benefit in any new user charge system. However, few of these entities can effectively argue that they shouldn’t pay for electric, water, wastewater, or garbage services provided to them, and the provision of general stormwater services should not be viewed differently, unless there are valid cost of service reasons to mitigate charging the stormwater rates. There is case law upholding the right of utilities to charge for service to these types of customers.

10.5 INTEGRATED FINANCIAL PLANNING MODEL

As a part of the stormwater master planning effort, HDR has developed an integrated financial planning model (IPFM) for the Okaloosa County stormwater program. It was developed on an Excel spreadsheet and provides for a five-year forecast of stormwater revenues and expenses, given an array of “what-if” assumptions of future program conditions. It integrates various planning, engineering, financial, and management/organizational issues into a coherent forecast of future program possibilities. The model was designed for flexibility and can be used for future program and capital planning, developing annual budgets, assessing alternative revenue sources, and providing a multi-year perspective on rate and fee-setting.

The model is, of course, based on various assumptions and the availability of existing data. Over time as improved information becomes available better inputs to the model can be specified and the forecasts made ever more relevant. Current limitations in the modeling or data include having to estimate current stormwater expenses, the lack of availability of reliable impervious cover data for the entire unincorporated area of the county, conceptual design and costing of new operational programs, conceptual engineering costing of new infrastructure projects, and some other factors to be discussed in the following sections.

10.5.1 Identification of Future Program Scenarios to be Modeled

In assessing future program alternatives, it is important to have, as a basic reference point, a picture of what might happen with continuing the status quo method of program organization and funding. In this way, the impacts of continuing to "do business" the same way can be identified, and the potential effects and costs of any new alternative courses of action can be judged against the current program approach. An array of new program alternatives should also be defined that span a range of meaningful future options and provide some "sensitivity" information on changes in key variables. As described in **Table 10.2**, a series of six scenarios were identified for modeling, evaluation, and reporting purposes. The financial planning model will be provided to the County at completion of this effort, so revised or updated scenarios can be modeled at a later date.

Table 10.2
Organizational, Program, and Funding Scenarios to be Modeled

1.1	**Scenario 1 – Continue Status Quo**
	County drainage service continues organizationally as an adjunct to its road and bridge program, and it funding sources would remain the same. No stormwater projects identified in the Master Plan would be implemented and limited capital spending would continue to be tied to drainage for road projects. In this status quo future, internal and outsourced activities for NPDES Phase II compliance would be funded as a matter of regulatory compliance, including adequate maintenance of drainage facilities.
1.2	**Scenario 2 – Modified Status Quo with Moderately-Paced Master Plan CIP**
	Same as Scenario 1, but stormwater projects identified in the Master Plan are funded at a moderate pace.
1.3	**Scenario 3 – Stormwater Utility with User Rates only and Moderately-Paced CIP**
	Same as Scenario 2 except an enterprise fund county stormwater utility would be formed and funded with dedicated stromwater rate revenues.
1.4	**Scenario 4 – Stormwater Utility with User Rates, Impact Fees, and Moderately-Paced CIP**
	Same as Scenario 3, but with the addition of impact fees as an extra funding source.
1.5	**Scenario 5 – Stormwater Utility with User Rates only and Aggressively-Paced CIP**
	Same as Scenario 3, but the stormwater projects are funded at a more aggressive pace.
1.6	**Scenario 6 – Stormwater Utility with User Rates, Impact Fees, and Aggressively-Paced CIP**
	Same as Scenario 5, but with the addition of impact fees as extra funding source and stormwater projects are funded at a more aggressive pace.

Differences between current spending levels and Scenario 1 will highlight the effects of implementing the NPDES Phase II program. The differences between Scenario 1 and 2 highlight the effects of implementing the Master Plan capital program. Scenario 3 indicates what an alternative funding levy would entail to provide for the same program expenses as in Scenario 2. Then, Scenario 5 illustrates the effects of more quickly funding capital improvements with the same funding levy. Scenarios 4 and 6 show the effects of adding an additional funding tool (impact fees) to the different speeds of the capital program.

It should be noted that many of Okaloosa's future stormwater program choices are not discretionary in the long-run, it is just a consideration of how to pay for efforts in a way that is fair and will help mitigate financial impacts.

10.5.2 Assumptions Common to All Future Program Scenarios

10.5.2.1 General Assumptions

All scenarios begin with estimates of current Okaloosa County stormwater spending from the FY2003 budget. All future scenarios are modeled over the prospective five-year period, FY2004 to FY2008. Rates of inflation for most expenditures are assumed at a annual rate of two percent with the exception of salaries and group health and life insurance increasing at annual rates of three percent and five percent, respectively. The current classified job descriptions and salary schedules of the County are assumed to continue with exception of the aforementioned inflation adjustment.

For all scenarios, except the status quo, a target of providing sufficient revenue to allow for a three-month operating reserve was also assumed. This operating reserve would provide for unexpected changes in projected expenses, such as unanticipated program expenses or additional expenses that are incurred during severe weather conditions.

10.5.2.2 Costing of NPDES Phase II Program

Various new NPDES Phase II activities anticipated for Okaloosa County are described in Appendix A. The projected level of additional effort varies from community to community depending upon what is submitted in the permit and what existing programs may already address NPDES Phase II issues. The level of effort and expense will also vary from year-to-year as the program develops and as larger studies or activities are initiated. There is also a consideration of whether to achieve these program requirements through internal efforts or outsourcing. Most entities are considering the outsourcing of the one-time-type efforts, but gaining internal capabilities for policy-related evaluations or for efforts that will continue from year-to-year.

In the case of Okaloosa County, a total of about 3,140 person-hours was estimated for NPDES Phase II programs in the first year, increasing to about 13,760 hours in Year 2, and then to about 22,560 hours of effort as the program reaches the mature stage. This includes both internal and outsourced efforts and activities across an array of County departments.

Within Public Works, it was estimated that NPDES Phase II would entail about 1,450 to 11,300 hours annually. It is assumed that about 15 percent of this effort would be outsourced with 85 percent done internally. Of this internal effort, about 20 percent of that can be accomplished with existing staff resources, thus leaving between 1,160 hours (Year 1) to 8,000 hours (Year 5) to be accomplished with new internal resources over the five-year implementation period. This equates to about 0.6 to eventually 4.3 full-time equivalent staff (FTEs) positions working on the NPDES Phase II program in Public Works. One position is already hired, and another can be filled through an existing staffing vacancy, thus leaving two new positions yet to be created and filled to meet program staffing requirements.

It is also estimated that two new positions will be needed to address NPDES Phase II activities in other County departments, most likely in the Growth Management and Water and Sewer Departments. Given the additional salary-related and non-labor expenses associated with this staffing, it is estimated that Public Works will require from about $300,000 to $350,000 of annual spending related to the NPDES Phase II programs, while other departments will need about $200,000 on a continuing basis for supporting these activities. All together, these NPDES Phase II non-maintenance activities will total about $300,000 to $550,000 annually during the first five years of the regulatory program.

10.5.2.3 Costing of Adequate Maintenance Program

Also part of these future NPDES Phase II efforts is a comprehensive inventory of the County drainage infrastructure. For purposes of this Master Plan, estimates of the miles of drainage ditches, drainage outfalls, ponds, curbed streets, and other drainage structure were obtained from County staff. Using these facility inventory estimates, the *Florida Department of Transportation Maintenance Rating Handbook*, and a survey of Okaloosa County entities, other entities' maintenance experience and costs, and discussions with Okaloosa County staff, HDR has estimated a level of stormwater maintenance efforts and equipment requirements for Okaloosa County that would generally meet good industry practices (FDOT, 2002; FDOT, 2003; Leon County (Florida) Public Works, 2003; and Universal City, Texas, 2003)

Table 10.3 presents the inventory estimate of current drainage facilities for which Okaloosa County has maintenance responsibilities, as well as a recommended frequency of maintenance and average duration of maintenance efforts per facility type. This leads to an identification of

Table 10.3
Field Operations Department – Estimated Crew and Equipment Needs
Okaloosa County Stormwater Management Program

Item	2002	2003	2004	2005	2006	2007	2008
Inventory of County-Maintained Drainage Facilities							
Curbs/gutters (mi/2sides/3 passes)	156	156	156	156	156	156	156
Storm Drains (mi)	52	52	52	52	52	52	52
Ditches (mi)	1,436	1,436	1,436	1,436	1,436	1,436	1,436
Outfalls (mi)	15	15	15	15	15	15	15
Ponds (#)	140	140	140	140	140	140	140
Recommended Maintainance Frequency (times per year)							
Streets w/curbs & gutters		4.00	4.00	4.00	4.00	4.00	4.00
Storm Drains		0.20	0.20	0.20	0.20	0.20	0.20
Mowing of Ditches and Outfalls		4.00	4.00	4.00	4.00	4.00	4.00
Excavation of Drainageways		0.20	0.20	0.20	0.20	0.20	0.20
Excavation of Ponds		0.10	0.10	0.10	0.10	0.10	0.10
Assumed Avg. Maintainance Duration, including mobilization/demobilization							
Streets w/curbs & gutters (hrs/mi)		1.00	1.00	1.00	1.00	1.00	1.00
Storm Drains (hrs/mi)		140.00	140.00	140.00	140.00	140.00	140.00
Mowing of Ditches and Outfalls (hrs/mi)		0.50	0.50	0.50	0.50	0.50	0.50
Excavation of Drainageways (hrs/mi)		20.00	20.00	20.00	20.00	20.00	20.00
Excavation of Ponds (hrs/pond)		8.00	8.00	8.00	8.00	8.00	8.00
Assumed Crew Size by Maintenance Activity							
Streets w/curbs & gutters		2	2	2	2	2	2
Storm Drains		1	1	1	1	1	1
Mowing of Ditches and Outfalls		1	1	1	1	1	1
Excavation of Drainageways		3	3	3	3	3	3
Excavation of Ponds		3	3	3	3	3	3
Estimated Maintenance Effort (FTEs/year)*							
Streets w/curbs & gutters		0.67	0.67	0.67	0.67	0.67	0.67
Storm Drains		0.78	0.78	0.78	0.78	0.78	0.78
Mowing of Ditches and Outfalls		1.56	1.56	1.56	1.56	1.56	1.56
Excavation of Drainageways		9.38	9.38	9.38	9.38	9.38	9.38
Excavation of Ponds		0.18	0.18	0.18	0.18	0.18	0.18
Total		12.6	12.6	12.6	12.6	12.6	12.6
Estimated Staffing Requirement (crew chief & members)							
Streets w/curbs & gutters	1 crew		1.00	1.00	1.00	1.00	1.00
Storm Drains	2 crews		2.00	2.00	2.00	2.00	2.00
Mowing of Ditches and Outfalls							
Excavation of Drainageways	3 crews		4.00	4.00	8.00	8.00	8.00
Excavation of Ponds							
Total		-	7.0	7.0	11.0	11.0	11.0
Related Equipment Needs							
Street Sweepers				1	1	1	1
Tractor/Mower			1	1	2	2	2
Transport					1	1	1
Vacuum Trucks					1	1	1
Excavator					1	1	1
Back Hoes					1	1	1
Dump Trucks			1	1	1	1	1
Pick-Up Trucks			2	2	2	2	2
Chain Saws			3	3	5	5	6
New Equipment Needs							
Street Sweepers		-	-	1	-	-	-
Tractor/Mower		-	1	-	1	-	-
Transport		-	-	-	1	-	-
Vacuum Trucks		-	-	-	1	-	-
Excavator		-	-	-	1	-	-
Back Hoes		-	-	-	1	-	-
Dump Trucks		-	1	-	-	-	-
Pick-Up Trucks		-	2	-	-	-	-
Chain Saws		-	3	-	2	-	1
Unit Equipment Costs ($/unit)							
Street Sweeper		$ 120,000	$ 122,400	$ 124,848	$ 127,345	$ 129,892	$ 132,490
Tractor/Mower		$ 35,000	$ 35,700	$ 36,414	$ 37,142	$ 37,885	$ 38,643
Transport		$ 90,000	$ 91,800	$ 93,636	$ 95,509	$ 97,419	$ 99,367
Vacuum Truck		$ 100,000	$ 102,000	$ 104,040	$ 106,121	$ 108,243	$ 110,408
Excavator		$ 90,000	$ 91,800	$ 93,636	$ 95,509	$ 97,419	$ 99,367
Back Hoe		$ 55,000	$ 56,100	$ 57,222	$ 58,366	$ 59,534	$ 60,724
Heavy Dump Truck		$ 65,000	$ 66,300	$ 67,626	$ 68,979	$ 70,358	$ 71,765
Pick-Up Truck		$ 22,000	$ 22,440	$ 22,889	$ 23,347	$ 23,814	$ 24,290
Chain Saw		$ 150	$ 153	$ 156	$ 159	$ 162	$ 166
Total New Equipment Costs							
Street Sweepers		$ -	$ -	$ 124,848	$ -	$ -	$ -
Tractor/Mower		$ -	$ 35,700	$ -	$ 37,142	$ -	$ -
Transport		$ -	$ -	$ -	$ 95,509	$ -	$ -
Vacuum Trucks		$ -	$ -	$ -	$ 106,121	$ -	$ -
Excavator		$ -	$ -	$ -	$ 95,509	$ -	$ -
Back Hoes		$ -	$ -	$ -	$ 58,366	$ -	$ -
Dump Trucks		$ -	$ 66,300	$ -	$ -	$ -	$ -
Pick-Up Trucks		$ -	$ 44,880	$ -	$ -	$ -	$ -
Chain Saws		$ -	$ 459	$ -	$ 318	$ -	$ 166
Total		$ -	$ 147,339	$ 124,848	$ 392,965	$ -	$ 166

* Assumes labor-hours per year of 1,856

the amount of crews and equipments needed to better address the County stormwater maintenance needs.

While northeast Florida experience was utilized in these parameters, it should be emphasized that these are representative averages and the actual frequency and duration of maintenance efforts and requisite equipment needs will vary from project to project. In some cases, special equipment such as dredges or drag line excavators are not specified as part of the County's equipment fleet, but could be contracted for services if needed.

This effort identified an need for six full-time crews who would address street sweeping (1 crew), vacuuming of storm sewers and mowing of ditches and outfalls (2 crews) and routine excavation of drainageways and ponds (3 crews). While the stormwater utility is envisioned funding all of these maintenance positions, four of the positions would be new, yielding an increase in County salary-related costs of about $192,000 per year above current spending. Overall salary-related costs associated with the improved maintenance activity totals around $500,000 to $600,000 annually. An array of additional equipment needs (totaling $665,000 over five years) was also identified and scheduled for purchase over a three-year period.

In all scenarios modeled, these enhanced maintenance efforts would be phased in over time to help minimize cost impacts. During Year 3 of the improved program, annual maintenance-related expenditures are expected to peak at about $1.150 million as major equipment purchases are made and then decrease to a continuing level of expenditure of about $820,000 annually after that.

These improved maintenance efforts would allow Okaloosa County drainageways and structures to function at their intended level of service and also provide for activities needed for NPDES Phase II compliance.

Note that County Public Works is currently conducting an inventory of all County stormwater facilities. When complete, this information should be substituted for the estimates provided, and will increase the accuracy of the presented maintenance projections.

10.5.2.4 Costing of CIP

Chapter 9 identifies a series of major and minor capital improvements projects for stormwater. These projects total over $3.8 million with about $1 million of that targeted at culvert replacements where drainage is currently impaired. An amount of $30,000 has been identified for the construction of five gaging sites to monitor water flows and/or precipitation.

This array of capital projects and costs are modeled in Scenarios 2 through 5 with only the speed of implementation varied between the scenarios.

10.6 FUTURE PROGRAM SCENARIO RESULTS

10.6.1 Scenario 1 – Continue Status Quo

In the status quo scenario, County stormwater services would continue organizationally as an adjunct to its road and bridge program, and its funding source would remain the same. No stormwater projects identified in this Master Plan would be implemented and limited capital spending would continue to be tied to drainage for road projects. In this status quo future, internal and outsourced activities for NPDES Phase II compliance would be funded as a matter of regulatory compliance, including adequate maintenance of drainage facilities.

Even without the implementation of the stormwater CIP, the total program expenses under the status quo scenario would total from about $2.0 million in the initial years, increasing to about $2.3 million per year as the new NPDES Phase II activities and related, improved maintenance programs are initiated and new equipment is purchased.

This would imply an increase in spending of about $1 million above current Transportation Trust Fund, road-related drainage funding. If the additional funds were to come from this source, it would mean that funding for existing roads programs would be reduced by that amount. It is not likely that the Transportation Trust Fund can provide the additional funding for stormwater programs not directly related to roads and bridges. Therefore, in this scenario, the incremental funding might need to originate from the County General Fund. If all of the stormwater program spending were to originate from general revenue, this would result in an implicit ad valorem tax rate of $0.40 to $0.52 per $100 assessed valuation. If the additional $1 million in stormwater spending (over the current spending) were to come from tax revenues, the incremental implicit tax rate would be about one-half of that.

It is unlikely that either: (a) existing General Fund programs would be reduced sufficiently to pay for these new stormwater program initiatives with no new taxes, or (b) the economic and political pressures would allow for the requisite tax increase.

10.6.2 Scenario 2 – Modified Status Quo with Moderately-Paced CIP

This scenario is similar to Scenario 1 with the addition of a moderately-paced capital improvements program. In this scenario, stormwater spending would range from about $2.8 to $3.3 million annually as the capital program is implemented on a cash-funded basis. Cash funding of the projects is assumed as long-term debt funding with this scenario and would involve General Obligation bonds and voter approval.

This yields an implicit tax rate of $0.60 to $0.67 per $100 valuation if the entire stormwater program were funded from this source or one-half of that if the Transportation trust Fund can be used to a considerable extent.

10.6.3 Scenario 3 –Stormwater Utility with User Rates Only & Moderately-Paced CIP

In this program scenario, a stormwater utility would be formed as a separate enterprise fund with funding arising from stormwater utility rates, and the stormwater CIP identified in this Master Plan would be implemented over a five-year period. The stormwater utility vehicle can provide for focused, measurable efforts and funding dedicated to stormwater services.

In this scenario, the CIP could be funded in a series of two revenue bond issues with repayment of the debt pledged from rate revenues. Program expenses would range from $2.1 to $2.6 million per year as the new programs are implemented and the two debt fundings of the capital projects are issued.

A stable stormwater rate structure that would generate this level of revenue during the 5-year planning period is $3.85 per month for each single-family revenue dwelling and $0.0023 per month square foot of impervious cover for non-residential land uses.

10.6.4 Scenario 4 –Stormwater Utility with User Rates Only & Aggressively-Paced CIP

In this program scenario, County stormwater services would again be solely supported from stormwater utility rates, but the stormwater CIP identified in this Master Plan would be implemented over a more rapid three-year period. If all of the capital projects were to be funded in one bond issue in the year 2005, the required stormwater rates would increase to about $3.95 per month per single-family customer and $0.0023 per month per square foot of impervious cover for non-residential customers.

10.6.5 Scenario 5 –Stormwater Utility with User Rates, Impact Fees & Moderately-Paced CIP

In this scenario, County stormwater services would be supported from stormwater utility rates and from impact fees imposed on new development, and the stormwater CIP identified in this Master Plan would be implemented over a five-year period.

This scenario is similar to Scenario 3 with the addition of stormwater impact fees applied to new development. With the addition of a stormwater impact fee of $250 per new residential dwelling and a non-residential fee of $0.1429 per square foot of impervious cover for new non-residential development, the monthly stormwater rates could be reduced to about $2.96 per single-family customer and $0.0017 per square foot of impervious cover for non-residential customers.

10.6.6 Scenario 6 – Stormwater Utility with User Rates, Impact Fees & Aggressively-Paced CIP

In this scenario, County stormwater services would be supported from stormwater utility rates and from impact fees imposed on new development, and the stormwater CIP identified in this Master Plan would be implemented over a three-year period. If all of the capital projects were to be funded in one bond issue in the year 2005 and both stormwater utility rate and impact fees were to be levied, the required stormwater rates would total about $3.05 per month per single-family customer and $0.0018 per month per square foot of impervious cover for non-residential customers. The impact fees are again assumed at $250 per new residential dwelling and a non-residential fee of $0.1429 per square foot of impervious cover for new non-residential development.

10.6.7 Comparison and Contrast of Modeling Scenarios

Table 10.4 presents a comparison of the key characteristics of the six program scenarios that were modeled. In Scenarios 1 and 2, State statutory or policy limits related to local sales tax options or gasoline/fuel tax revenue sharing may limit any significant additional funding from these sources to pay for additional stormwater funding needs, especially if such new stormwater funding is not directly related to road construction. Further, the option of reducing spending on road construction to fund additional stormwater programs may also not be a viable option, given the increased need for transportation facilities with a growing population. For Scenario 1 and 2, Table 10.4 indicates the extent of additional stormwater funding needs over current levels of expenditures.

All of the scenarios involving a stormwater utility reflect potential utility rates in the range of other stormwater utilities. A prior survey of 206 stormwater utilities in the U.S. found that the mean monthly rate per equivalent residential unit was $3.80 and the median monthly rate was $3.00. These rate levels are now somewhat low compared to today’s cost of stormwater services. The survey is now almost four years old, and these rates were surveyed prior to the implementation of NPDES.

Table 10.4
Comparison of Financial Effects of Alternative Program Scenarios

Scenario	Description	Organizational Identity	Pace of Capital Spending	Annual Expenditures (mill. $) 2003	Annual Expenditures (mill. $) 2005	Annual Expenditures (mill. $) 2008	Needed Funding Levies 2003	Needed Funding Levies 2005	Needed Funding Levies 2008	Comment
1	**Continue Status Quo**	Adjunct to Roads & Bridges								
	Expenditures									
	Operations and Maintenance			$ 0.886	$ 1.869	$ 2.106				If increases in various Transportation Trust Fund levies are not possible, then either spending on county roads would need to be reduced, tax support would be needed, or additional stormwater needs would go unfunded.
	Cash Funded Capital plus Debt Sevice		do not fund major CIP	$ 0.074	$ -	$ -				
	Transfer Out to Other Funds			$ -	$ 0.101	$ 0.202				
	Total			$ 0.960	$ 1.970	$ 2.308				
	Funding Levies									
	Additional Funding Needs over FY03 (mill. $)				$ 1.010	$ 1.348				
2	**Modified Status Quo with Moderately Paced CIP**	Adjunct to Roads & Bridges								
	Expenditures									
	Operations and Maintenance			$ 0.886	$ 1.870	$ 2.107				If increases in various Transportation Trust Fund levies are not possible, then either spending on county roads would need to be reduced, tax support would be needed, or additional stormwater needs would go unfunded.
	Cash Funded Capital		complete CIP in 5 yrs	$ 0.074	$ 0.852	$ 1.029				
	Transfer Out to Other Funds			$ -	$ 0.101	$ 0.202				
	Total			$ 0.960	$ 2.823	$ 3.338				
	Funding Levies									
	Additional Funding Needs over FY03 (mill. $)				$ 1.863	$ 2.378				
3	**Stormwater Utility with User Rates only & Moderately Paced CIP**	Enterprise Fund								
	Expenditures									
	Operations and Maintenance			$ 0.886	$ 1.870	$ 2.107				
	Cash Funded Capital plus Debt Sevice		complete CIP in 5 yrs	$ 0.074	$ 0.194	$ 0.316				
	Transfer Out to Other Funds			$ -	$ 0.101	$ 0.202				
	Total			$ 0.960	$ 2.165	$ 2.625				
	Funding Levies									
	Monthly User Rates									
	Single family Residential (per DU)						$ -	$ 3.85	$ 3.85	
	Non-residential ($ per sq ft)						$ -	$ 0.0023	$ 0.0023	
4	**Stormwater Utility with User Rates only & Aggressively Paced CIP**	Enterprise Fund								
	Expenditures									
	Operations and Maintenance			$ 0.886	$ 1.870	$ 2.107				
	Cash Funded Capital plus Debt Sevice		complete CIP in 3 yrs	$ 0.074	$ 0.316	$ 0.316				
	Transfer Out to Other Funds			$ -	$ 0.101	$ 0.202				
	Total			$ 0.960	$ 2.287	$ 2.625				
	Funding Levies									
	Monthly User Rates									
	Single family Residential (per DU)						$ -	$ 3.95	$ 3.95	
	Non-residential ($ per sq ft)						$ -	$ 0.0023	$ 0.0023	
5	**Stormwater Utility with Users Rates, Impact Fees, & Moderately Paced CIP**	Enterprise Fund								
	Expenditures									
	Operations and Maintenance			$ 0.886	$ 1.870	$ 2.107				
	Cash Funded Capital plus Debt Sevice		complete CIP in 5 yrs	$ 0.074	$ 0.194	$ 0.316				
	Transfer Out to Other Funds			$ -	$ 0.101	$ 0.202				
	Total			$ 0.960	$ 2.165	$ 2.625				
	Funding Levies									
	Monthly User Rates									
	Single family Residential (per DU)						$ -	$ 2.96	$ 2.96	
	Non-residential ($ per sq ft)						$ -	$ 0.0017	$ 0.0017	
	One-time Impact Fee for New Development									
	Single family Residential (per DU)						$ -	$ 250	$ 250	
	Non-residential ($ per sq ft)						$ -	$ 0.1429	$ 0.1429	
6	**Stormwater Utility with Users Rates, Impact Fees, & Aggressively Paced CIP**	Enterprise Fund								
	Expenditures									
	Operations and Maintenance			$ 0.886	$ 1.616	$ 1.792				
	Cash Funded Capital plus Debt Sevice		complete CIP in 3 yrs	$ 0.074	$ 0.316	$ 0.316				
	Transfer Out to Other Funds			$ -	$ 0.101	$ 0.202				
	Total			$ 0.960	$ 2.033	$ 2.310				
	Funding Levies									
	Monthly User Rates									
	Single family Residential (per DU)						$ -	$ 3.05	$ 3.05	
	Non-residential ($ per sq ft)						$ -	$ 0.0018	$ 0.0018	
	One-time Impact Fee for New Development									
	Single family Residential (per DU)						$ -	$ 260	$ 260	
	Non-residential ($ per sq ft)						$ -	$ 0.1429	$ 0.1429	

10.7 Recommended Organizational and Funding Plan

Given the regulatory and funding pressures facing Okaloosa County, the current method of road project related funding of drainage projects will no longer be sufficient to meet future stormwater needs. Further, it is not likely that additional use can be made of Transportation Trust Fund or County General Fund revenues, and neither source of funding is very equitable in terms of making users (causing the problem or benefiting from the solutions) pay for service rendered.

Many local governments facing similar program needs and funding limitations have already or are in the process of turning to the stormwater utility (SWU) organization as the most viable method for addressing future program needs. The stormwater utility focuses the program efforts and provides for accountability. The stormwater user rates and impact fees are equitable and are levied at a primary cause of stormwater problems, impervious cover runoff. The dedicated source of rate revenue also allows for other new possibilities, such as the use of revenue bonds that avoid the political difficulties associated with General Obligation bonds.

10.7.1 SWU Organizational Concept

In consultation with County staff as to the most practical means of accomplishing various County stormwater activities, it is recommended that the possible new stormwater program incorporate appropriate elements of existing County departments, but also centralize certain stormwater efforts under the management umbrella of the stormwater utility.

In this scenario, a new stormwater division would be created in Public Works with three key underlying programs: (a) administration of the stormwater utility, planning, and water quality (NPDES Phase II) programs, (b) stormwater engineering and project management, and (c) plan review and project inspection. Current stormwater maintenance efforts would be enhanced through funding transfers to the Roads Department with accountability to a field supervisor located within the stormwater utility.

With the recent hire of an employee that could serve as the Stormwater Utility Manager and an existing staffing vacancy, the identified need for four full-time equivalent positions within the possible Admin, Planning & Water Quality Division of the utility would be reduced to two additional unbudgeted positions. As mentioned earlier, transfers to other departments were also modeled to provide for two new NPDES Phase II support positions and for four additional maintenance positions in the Roads Department. The funding transfer to the Roads Department also assumes that existing maintenance staff will focus their efforts full-time in stormwater maintenance needs.

10.7.2 Stormwater Funding

It is recommended at this time that the Okaloosa County Commissioners consider developing Scenario 3. In this scenario, the CIP would be funded in a series of two revenue bond issues with repayment of the debt pledged from rate revenues. Program expenses would range from $2.1 to $2.6 million per year as the new programs are implemented, equipment is purchased, and the two debt fundings of the capital projects are issued. While not relied upon in the modeling given their uncertainty, receipt of grant proceeds could help to address certain program or capital expenses and would help reduce the projected annual outlay.

A stable stormwater rate structure that would generate this level of revenue during the five-year planning period is estimated at $3.85 per month for each single-family revenue dwelling and $0.0023 per month square foot of impervious cover for non-residential land uses.

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